Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost

Editors
Paulo J.S. Cruz, Dan M. Frangopol & Luis C. Neves

International Association for
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Edited by

Paulo J.S. Cruz
University of Minho, Guimarães, Portugal

Dan M. Frangopol
University of Colorado, Boulder, Colorado, USA

Luis C. Neves
University of Minho, Guimarães, Portugal

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Table of Contents

Application of structural system identification methods

Long-term signal analysis for the existing bridge health monitoring systems

Damage assessment – strength and durability

Assessment, monitoring and control of bridge vibrations

Seismic and dynamic analysis

Assessment of bridge repair and strengthening

Handling uncertainty in analysis design

Status and findings of current BHM applications in the world

Inspection and prediction of structural performance

Bridge inspection and monitoring

Fatigue analysis

Bridge owners benefits from probability-based assessment and maintenance management

Reliability and risk management

Seismic design and retrofitting strategies for bridges

Bridge testing and assessment

Research needs for BHM systems of the future and benchmark studies

Integration of bridge management and bridge monitoring

Innovative developments towards improving bridge seismic safety

Soft computing in bridge engineering

Loads and testing

Bridge evaluation using field testing

Business intelligence and asset management

Load and resistance assessment of railway bridges

Design and analysis

Measurement and monitoring

Civil structural health monitoring

Repair and strengthening

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Preface

The number of deteriorating bridges is increasing worldwide. Costs of maintenance, repair and rehabilitation of these bridges far exceed available budgets. Maintaining the safety of existing bridges by making better use of available resources is a major concern for bridge management. Internationally, the bridge engineering profession has taken positive steps to develop more comprehensive bridge performance measures, conduct better maintenance activities, and provide new forms of bridge management. It was therefore considered appropriate to bring together all of the very best work that has been done in the field of bridge maintenance, safety, management, life-cycle performance and cost at the Third International Conference on Bridge Maintenance, Safety and Management (IABMAS'06), held in Porto, Portugal from July 16 to 19, 2006. The First (IABMAS'02) and Second (IABMAS'04) International Conferences on Bridge Maintenance, Safety and Management were held in Barcelona, Spain, July 14–17, 2002, and Kyoto, Japan, October 18–22, 2004, respectively.

The International Association for Bridge Maintenance and Safety (IABMAS), which serves as the organizing association of IABMAS'06, encompasses all aspects of bridge maintenance, safety, and management. Specifically, it deals with: health monitoring and inspection of bridges; bridge repair and rehabilitation issues; bridge management systems; needs of bridge owners, financial planning, whole life costing and investment for the future; bridge related safety and risk issues and economic and other implications. The objective of IABMAS is to promote international cooperation in the fields of bridge maintenance, safety, management, life-cycle performance and cost for the purpose of enhancing the welfare of society. The interest of the international bridge community in all these fields has been confirmed by the high response to the call for papers. In fact, more than 500 abstracts were received at the Conference Secretariat. About 80% of them were selected for final publication as full-papers and presentation at the Conference within four plenary sessions and 72 technical sessions. Compared to IABMAS'04 the total of number of papers scheduled for presentation has increased from 347 to 421.

IABMAS'06 covered all major aspects of bridge maintenance, safety, management, life-cycle performance and cost including advanced materials, ageing of bridges, assessment and evaluation, bridge codes, bridge diagnostics, bridge management systems, composites, design for durability, deterioration modelling, emerging technologies, fatigue, field testing, financial planning, health monitoring, high performance materials, innovations, inspection, life-cycle performance, load capacity assessment, loads, maintenance strategies, new technical and material concepts, non-destructive testing, optimization strategies, prediction of future traffic demands, rehabilitation, reliability and risk management, repair, replacement, residual service life, safety and serviceability, service life prediction, strengthening, sustainable materials for bridges, sustainable bridges, and whole-life costing, among others.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost contains the lectures and papers presented at IABMAS'06. It consists of a book of abstracts and a CD-ROM containing the full texts of the lectures and papers presented at IABMAS'06, including the T.Y. Lin Lecture, nine Keynote Lectures, and 411 technical papers from 35 countries. This set provides both an up-to-date overview of the field of bridge engineering and significant contributions to the process of making more rational decisions in bridge maintenance, safety, management, life-cycle performance, and cost for the purpose of enhancing the welfare of society.

On behalf of IABMAS, the chairs of the Conference would like to take this opportunity to express their sincere thanks to the authors, organizers of special sessions and mini-symposia, and participants for their contributions, to the members of the Conference Scientific Committee for their dedicated work, and to the members of the Local Advisory and Organizing Committees for the time and effort they have dedicated to make of IABMAS'06 a successful event. Finally, we would like to register our sincere thanks to all the sponsors of IABMAS'06.

Paulo J.S. Cruz and *Dan M. Frangopol*
Chairs, IABMAS'06
Guimarães and Boulder, April 2006

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T.Y. Lin Lecture

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Bridge forms and aesthetics

Man-Chung Tang
Chairman of the Board and Technical Director, T.Y. Lin International, USA

ABSTRACT: A successful bridge design must be natural, simple, original and harmonious with its surroundings. Aesthetics is not an addition to a bridge. It is an integral part of a bridge design. Both structural configuration and aesthetics must be considered together during the conceptual stage of a bridge. To achieve such a task, the bridge engineer must have a good understanding of structural theory and bridge aesthetics.

1 INTRODUCTION

About two thousand years ago, in "De Architectura", Marcus Vitruvius Pollio proclaimed:

"Structures shall be safe, functional and beautiful!"

Today, we will modify this slightly to: "A bridge must be safe, functional, economical and beautiful!"

Cost was not a priority for important structures in the Roman Empire because Marcus Vitruvius Pollio worked for the Emperor. We do not have that luxury. Besides that, not much has changed in how we define a successful bridge.

2 STARTING FROM THE BASICS –THE A-B-C OF STRUCTURES

A bridge can have various forms. It can be a girder, a truss, an arch, cable-stayed, or suspended. Even though they may all look very different from each other, each of these bridges is comprised of only three basic structural elements: axial, bending and curved elements. I call this the ABC of structures. By combining and mixing of these three types of basic structural elements, we can create all different types of bridge configurations.

For example, a truss bridge consists of mainly axially loaded elements. Bending may exist, but it is secondary. So is a cable-stayed bridge. An arch carries its load with a change of curvature. This is also true in the suspension cable of a suspension bridge. A girder bridge is mainly a bending element Fig. 1.

Even though some engineers tend to specify the desired bridge type in relation to bridge spans, in reality there are no rules dictating what is the best form to satisfy a given function of a bridge. It depends on many variables such as the cost and availability of construction materials, transportation, skill of labor, aesthetics, and many other reasons.

3 ENGINEERING IS AN ART, NOT A SCIENCE

Engineering is not applied science. The aim of science is to discover truth. Truth is unique. Therefore, science is rigid because we can not change the truth. An engineer creates things based on his/her experiences and his/her preferences. Experience is not unique. Preference is subjective. Therefore, engineering is flexible. This flexibility affords engineers the possibility to select, or even to create, new forms of structures to satisfy the given function of a bridge, within certain limitations.

We already know that all of the possible forms of bridges are comprised of three basic structural elements: the axial, bending and curved elements. The art of bridge engineering is knowing how to

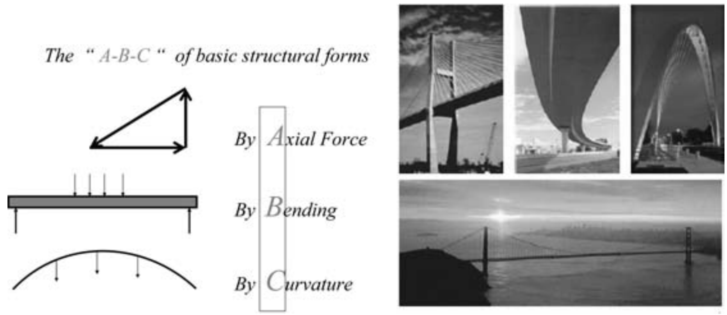

Figure 1. A-B-C of structures.

mix these elements together to arrive at the best structure that satisfies the fundamental requirements of "safe, functional, economical and beautiful." This is similar to a cook mixing various ingredients in a certain way so the food will taste best.

But, in practice, the task of coming up with the best combination requires experience and training. Just like playing piano or golf. The basics are simple, but to play well requires many years of hard work and practice.

Pointing out that bridge engineering is not a science, but an art, should remind many engineers, especially the younger engineers, that dabbling in extensive analysis and structural theories is not really bridge engineering. These exercises only serve as some basic tools a bridge engineer utilizes to design bridges. To be able to design a bridge successfully, a bridge engineer must be versatile and knowledgeable in many other areas.

4 HISTORIC EVOLUTION OF BRIDGE FORMS

All of the major bridge forms of today, girder, arch, truss, cable-stayed and suspension, are actually almost as old as human civilization. In some primitive forms, they had all been built many, many centuries ago, Fig. 2. However, the evolution of today's more sophisticated forms can be traced to the introduction of various onstruction materials of various times.

In the very old days, say 4000 years ago, the only construction materials available were wood and stones. Wood from tree trunks could be used as girders. So people chopped down trees to create bridges. For larger crossings, they put up stone piers so the tree trunk could span between them. But these bridges were not sturdy. They did not last long and could not have large spans.

Stone is much stronger than wood, but only in compression and not in tension. Therefore, all early Egyptian and Greek buildings have very closely spaced columns. The roofs of these buildings were also made of stones. But most of them did not stand the test of time, because stone can not sustain high tensile stresses from bending.

The arch was the most ingenious invention by the Chinese and the Romans to fully utilize the compressive strength of stone. The Romans built the barrel vault by connecting successive arches together to create a large inner space. The dome was created by rotating the arch around its vertical axis. Using the dome, the arch and the vault, the Romans built many spectacular structures. The Pantheon, with its 43.3 m diameter dome, was the largest dome in the world for about 1800 years.

The arch allowed the Romans to build many bridges and viaducts, some of which are still in use today.

Iron, with its greater strength, gave us longer and more slender arch spans. But iron also lacks tension capacity so it did not really usher in any new form of bridge beyond arches.

It was steel that revolutionized bridge construction. This is because steel can take both tension and compression. After mass production of steel was made possible, we were able to build long span girder bridges, truss bridges and large and slender arch bridges.

Combining steel and concrete, we also built many long span reinforced concrete bridges. High-strength wires were produced by a cold drawn process. The great strength of the wires makes

Figure 2. Ancient structures of stone and wood.

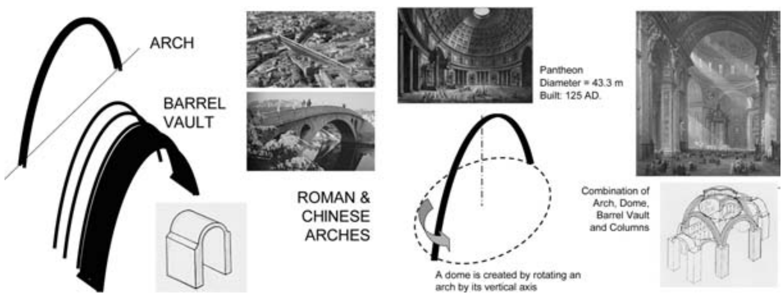

Figure 3. The arch was a great invention.

long span suspension bridges possible. Before high-strength wires were commercially available, suspension bridges were built using steel chains and eye bars.

The introduction of high-strength wires also stimulated the development of cable-stayed bridges. A cable is not effective if the sag due to its own weight is too large. The sag is a function of the tension in the cable. Therefore, a cable can only be effective if we can stress it to a very high force, which is possible only if the cable is made out of high strength material.

High-strength wires also improved the economy and versatility of prestressed concrete. As a result, we have built many prestressed concrete bridges.

It is evident that the availability of specific materials affects the selection of bridge forms. Thus we can create a simple flow chart to represent this process, Fig. 5.

Looking to the future, there are new materials such as carbon fibers, ultra high performance concrete and nano material that may be useable in the development of new bridge forms. But they are not yet ready for large scale application.

5 SPAN LIMITATIONS

When we need to cross a big river or a bay, the question always comes up is: How long of a span we can build? Before we estimate the largest spans we can construct, let us look back at the history of various bridge forms. Fig. 6 shows how the maximum span of each type of bridge has evolved. I have also included in Fig. 6 the longest spans that are under construction. They fit surprisingly well into the historic curves.

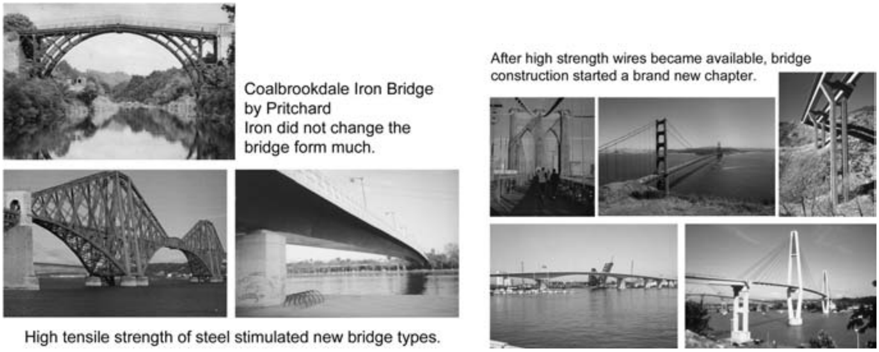

Figure 4. Introduction of new materials stimulate development of new bridge forms.

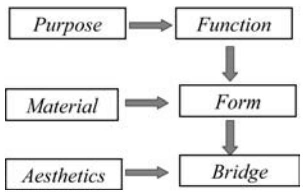

Figure 5. Flow chart.

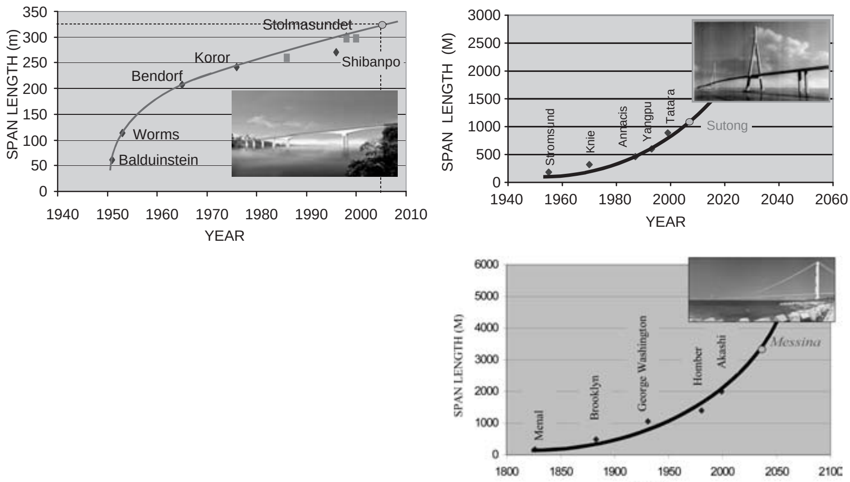

Figure 6. Milestone maximum spans of various bridge forms. a. concrete segmental bridges b. cable-stayed bridges c. suspension bridges.

All of these curves show a trend: the maximum span length is increasing. But, the maximum possible spans are way beyond what we are building nowadays. The maximum span depends mainly on the allowable stresses we can have in the construction materials we use. In [1], based on the construction materials we have today, steel and concrete, the maximum span of a cable-stayed bridge and suspension bridge is estimated to be 4,000 m and 7,000 m respectively. Compared with the longest spans we are building today, the 1,088 m Sutong cable stayed bridge and the 3,300 m span Messina suspension bridge, there is no need to worry about reaching the limit.

However, being able to build a very long span technically does not mean we can freely choose longer spans even if they are not necessary. Longer span usually cost more. Since economy is one of the basic considerations for the success of a bridge project, we must keep this in proper perspective.

6 VALUE VERSUS COST

A bridge is economically feasible if its value is equal to or higher than its cost. The value of a bridge is the sum of many components – generally, its functional value, its social value and its aesthetic value.

The value of function is obvious. Because of the bridge, a user can reduce the time required to reach from Point A to Point B. This convenience saves money and time, which can be represented by a dollar value. As the standard of living in a society inches higher, the value of this convenience also increases. A bridge project that was economically not feasible 20 years ago may become feasible today.

A bridge not only connects places, it also connects people. Very often a bridge is built to symbolize a unification, a friendship or simply a bond between two communities. The value of this symbolism is hard to determine numerically. But under certain circumstances, it can be very significant. It can even be the sole reason for the construction of a structure.

A famous saying states, "Beauty is in the eyes of the beholders!" Beauty is very subjective. But beauty does have value. With very few exceptions, people do not wear simply the cheapest clothing because we want to look good, or at least to look appropriate. We decorate our homes because making them more beautiful makes us enjoy more living in there. We travel to places just to enjoy looking at the beautiful scenery and surroundings there. There is no doubt that we are willing to pay for aesthetics all of the time, knowingly or unknowingly.

A nice-looking building in a nice neighborhood can demand a higher rent. Consequently, aesthetics considerations have always been important in the design of commercial buildings, both exterior and interior. Webster Dictionary defines "civilization" as "*modern comforts and conveniences, as made possible by science and technology*." Here, convenience is achieved through functionality. Comfort, however, can only be achieved by having quality in our environment, which includes the form and aesthetics of our structures. Without quality in our environment, we cannot have quality in our life.

Most bridges are paid for by tax dollars. They are indirectly paid for by the taxpayers, or citizens just like you and me. Consequently, it is the responsibility of the bridge engineer to understand what the taxpayers would do if they were to design the bridge. Considering that most taxpayers are willing to pay for aesthetics in their daily life, there is no reason we engineers should impose on them just the cheapest bridge possible.

Besides, aesthetics does not always cost money. Just paying more attention to details can bring significant improvement in the appearance of the structure.

Coming back from Paris, we remember the Eiffel Tower, the paintings in the museums, the Louvre, the Arc de Triumph, the Alexander III Bridge, and other beautiful structures. Most of them actually have no functional value, like the famous painting of Mona Lisa. We remember them, and love them because they are beautiful. And you do not need to be a bridge engineer to notice that the bridges across the Seine are beautiful structures, with thoughtful attention to aesthetics.

It would be unthinkable if all the bridges over the Seine would have been built the cheapest way possible. Paris would not be the Paris we know today!

7 BRIDGE AESTHETICS

We should treat bridge design as an art. But a bridge is not just an art object. The basic purpose of a bridge is still to carry traffic. We may create a sculpture just because it looks good. We never build a bridge just because it looks good. Combining these requirements, a successful bridge design must be natural, simple, original and harmonious with its surroundings.

A bridge is usually a large and very visible structure in its vicinity. It should look natural and fit well into the landscape. It should also be simple and not look superficial. A structure looks more natural if it can convey an understandable impression to the general public about how it works.

Uniqueness is an important factor in any piece of art. Likewise, each bridge should be unique in itself. Each structure has its own requirements and unique surroundings. Each bridge should be

Figure 7. Memories of paris.

Figure 8. Natural, simple, original and harmonious.

designed based on its own conditions. Therefore, each bridge should be original and have its own style, its own characters and its own design. Just like a painting, only the original is valuable.

Being harmonious with its surroundings does not necessary mean that the bridge must just blend well with its environment. It means that the structure should be configured such that it “fits” well in its position. Very often that may mean that the bridge stands out in its surroundings, if that is more appropriate.

At the same time, it should be noted that a bridge will be viewed from all different angles. Looking good only from one view point is not sufficient.

8 DECORATIONS

A bridge is just like any other large structure. In most cases, the magnitude and the form of the bridge itself is so powerful that decoration only makes it look less impressive. However, if a bridge is built to symbolize certain events or a relationship, like the Alexander III Bridge in Paris, which was built to celebrate the friendship between the Russian Empire and France, decoration can convey certain meanings and impressions. In general, bridges in cities tend to be more suitable for decorations, which can create more harmony with the bridge’s surroundings. They tend to be smaller in size too. A bridge in a natural setting should be more natural and simple and decoration is usually not desirable.

9 AESTHETIC LIGHTING

Aesthetic lighting is an art in itself. It makes the bridges not only visible but gives them a life at night. But it is important to differentiate between illumination and aesthetic lighting. Illumination just makes a bridge visible but aesthetic lighting makes use of the interaction of light and the structure to create special effects and impressions Fig. 9.

It is also important that aesthetic lighting be considered early in the design so that all physical facets of the lighting can be properly accommodated in the structure. Some lighting elements can appear undesirable in the daytime if they are not properly located.

Figure 9. Aesthetic lighting.

10 WHO DESIGNS THE BRIDGE – ENGINEER OR ARCHITECT?

A bridge is not a sculpture. A bridge has to be safe, functional and economical too. These are matters that are best dealt with by engineers. It is a bad idea to leave the aesthetics of a bridge only to architects. Architects can not conceptualize a complete bridge. They do not have the sufficient training in structural engineering to perform this task. It is also wrong, although practiced by many engineers, to ask the architect to beautify the bridge after the engineer has finished the structural design. By that time, all an architect can do is add decorations, which is not always appropriate.

Aesthetics is not an addition to a bridge. It is an integral part of a bridge design. Both structural configuration and aesthetics must be considered together during the conceptual stage of a bridge. To achieve such a task, the bridge engineer must have a good understanding of structural theory and bridge aesthetics. It is unfortunate that most engineering schools do not teach aesthetics. A bridge engineer must learn aesthetics inside or outside of the school. And this is not difficult, but rather, very exciting!

As a rule, the engineer must be the prime designer of a bridge. He must conceptualize the bridge to satisfy all structural and functional requirements with due considerations of aesthetics. As a compromise, he can also work together with an architect during the conceptual stage of the design.

11 AESTHETICS

What is aesthetics?

Most engineers are used to following rules in the books. When it comes to aesthetics, there are no rules. Even though many scholars have tried to establish rules for structural beauty, no one has ever succeeded. The "golden section" concept, which is supposed to prescribe the proportion of the best looking rectangle, for instance, has been studied and promoted by various architects for many years. But even today, no one can really say that such a rectangle that has a proportion of the two sides of Ô (=1.6318..) definitely looks better than a square, under all circumstances.

In general, aesthetics are just about proportion, balance, and harmony. When we look at an object, we do not go through any logical derivation to determine whether it is beautiful or not. The reaction is more spontaneous. Even though not everyone agrees, most people have come to the conclusion that there are no rules for aesthetics.

Human perception often changes with time. But real beauty transcends time and style. A beautiful bridge can be dramatic and daring. It can also be graceful and poetic. The basic idea is to attract an emotional response from the viewers, and a kind of surprise. How we achieve this is the art.

Nature endorses simplicity. All important rules of the nature are simple. Even the most important equations of nature in physics are extremely simple, like $F = ma$, or $E = mc2$. The human mind, being immersed in nature most of the time, is accustomed to simplicity. It has been proven repeatedly that the simplest configuration is usually the best-looking solution. It has been said that to arrive at the most beautiful structure, the best method is to try to take away any component that can be taken away – a process of simplification! Obviously, this requires experience and a good understanding of structure and aesthetics.

Figure 10. Multiple alternatives.

Figure 11. Elements of surprise.

We teach our children to walk properly, talk properly and move properly. A person with a good posture automatically emits a certain charisma. Clothing style and accessories are secondary. A good posture does not cost additional money. Once a person is well trained in manner and attitude, it comes naturally. This also applies to bridge design. It actually applies to any kind of art. If we disregard the skill of the artist or the designer, the actual cost of a good painting versus a bad one is basically the same: the canvas, the paint, the brushes, etc. The material required for a $100 dress is not necessarily less expensive than a $5,000 dress. The difference is how the materials are put together by the designer. In the same way, the basic cost of a beautiful bridge and a mediocre bridge can be negligible. In reality, a well designed, good looking bridge is usually more economical because it is more natural and simple. It follows the intentions of nature.

It is obvious that there are three distinct differences between bridge design and other forms of art. First, a bad painting will just end up gathering dust in a basement some place and never bother anybody again, while a bridge, once it is complete, will be prominently displayed in the public eye for hundreds of years. The community can not escape being effected by it. Second, and this is unfortunate, the fruits of success are rewarded to the painter or the dress designer, while in our present system, there is little incentive for a bridge engineer to spend additional effort to strive for the best looking bridge form because his fee will just be the same regardless.

In addition, most other artists work more or less alone while an engineer works in a group and must deal with many related parties, including economists and politicians, who may elect to impose their own idea on the bridge engineer.

Making the effort to search for the best-looking bridge alternate does require time and effort. But, once this becomes a routine in design, this additional effort is not significant. Despite the above factors, it is still a responsibility of a bridge engineer to pay attention to aesthetics. A bad-looking bridge is a kind of pollution to the community. That pollution will be there for a long, long time.

12 SUMMARY

With the flexibility of engineering and by utilizing the three basic elements, the A-B-C's of structures, we should be able to create bridge forms to fit any given occasion. It is not difficult. Just keep it simple, natural, original and harmonious with the landscape. A bridge engineer can not neglect this responsibility.

If we want quality in our life, we must have quality in our environment. If we want quality in our environment, we must pay close attention to the beauty of the structures we build, including all bridges. We must keep our world beautiful for the enjoyment of all citizens.

This is the only world we have.

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Protection of our bridge infrastructure against manmade and natural hazards

F. Seible, G. Hegemier, J. Wolfson, R. Conway & K. Arnett
Department of Structural Engineering, Jacobs School of Engineering, University of California, San Diego, La Jolla, California, USA

J.D. Baum
Science Applications International Corporation, Center for Applied Computational Sciences, McLean Virginia, USA

1 INTRODUCTION

Recent and continuing terrorist attacks on all of our nations' building and transportation infrastructure have clearly demonstrated the need to develop and implement blast mitigation and hardening optimization methodologies to protect our built infrastructure, both at home and overseas. In many cases similar structural response for both the seismic and the blast load case can be observed. For example, both can result in progressive structural collapse and mitigation measures, such as design for redundancy and ductility, as well as retrofit measures to increase concrete confinement apply to both seismic and blast conditions. The vulnerability of these structures under a range of potential threat scenarios must be quantified. New and existing retrofit technologies must be verified. Computational seismic and blast physics models must be improved and validated for damage prediction and to determine effective mitigation strategies.

To address these issues, the Powell Structural Research Laboratories at the University of California, San Diego have been investigating earthquake and blast mitigation strategies by performing a wide range of field tests, laboratory tests and computational analyses on critical bridge infrastructure. While seismic hazard mitigation for bridges has developed over the years into an established science, blast mitigation research for bridge structures is just commencing. The first series of bridge blast performance characterization was done by Caltrans (California Department of Transportation) on reinforced concrete bridge decks. The second test series was performed on an orthotropic steel bridge deck. The test articles in these two test series are representative of large, heavily utilized bridge structures in California as well as many other locations around the world. The decks in both of these series were subject to a variety of explosive loads in field tests performed at the Naval Air Warfare Center Weapons Division at China Lake. Computational bridge response analyses and damage predictions were also performed.

In addition to the bridge deck test program, testing is being performed on steel cellular tower bridge components. These bridges are highly visible and have been identified as potential targets. The test program for these components includes live explosive field testing, the performance of sophisticated computational analyses, and laboratory testing using the world's first hydraulically driven blast simulator that simulates explosive loads without the use of explosive materials. The blast simulator program is funded by the Technical Support Working Group (TSWG) as part of their international Blast Mitigation program. The blast simulator is located at the Englekirk Structural Engineering Center of UCSD, a unique multi-hazard structural engineering test facility that also includes the largest outdoor shake table in the world, funded by the National Science Foundation, and the Caltrans Soil Foundation Structure Interaction facility.

Testing has now shown that retrofit technologies developed for seismic mitigation have proven very effective in mitigating blast damage, in particular those using carbon reinforced fiber polymer wrap. Comparisons between seismic and blast loading is providing insight into what other technologies may be successful in mitigating multiple structural hazards. Further testing of these methodologies is underway.

2 BRIDGE DECK FIELD TESTING

2.1 *Blast testing of a reinforced concrete deck*

Blast testing was conducted at Naval Air Warfare Center Weapons Division at China Lake on a full scale, reinforced concrete box girder test specimen. Since the primary objective of the test was to determine the localized damage to a bridge superstructure as a result of a car bomb, the specimen consisted only of the top reinforced concrete deck slab.

The construction of the specimen was done at UCSD and then transported to the test site at China Lake. Multiple sensors (pressure transducers, displacement transducers, accelerometers, and strain gauges) were embedded in the slab. The steel used was Grade 60 A615 steel, by definition having a minimum yield strength of 413.7 MPa (60 ksi), a minimum ultimate tensile strength of 620.5 MPa (90 ksi), minimum percent elongation of 9% in the #4 and #5 bars and 8% in the #7 bars, and a phosphorus content of less than 0.06%. The concrete used was specified to have a compressive strength of 27.58 MPa (4000 psi).

Various charge weights were placed at specific clearances above the slab, incrementally increasing after each test. The first two tests consisted of charge weights of 25 pounds TNT equivalent and 50 pounds of TNT equivalent, respectively. The clearance above the deck of 16” corresponds to the height of a car trunk loaded with explosives. In the first test, the 25-pound charge of TNT equivalent was placed at the mid-span of one span. The second test of the 50-pound TNT equivalent charge was placed at mid-span of the other span. The third test consisted of a charge of 200 pounds of TNT positioned at the same location as that of Test 1. The minimal damage resulting from the first test enabled the re-use of the span for the third test, with significant assurance that the effects of the first blast would have very little effect at the macroscopic structural response level on the third test.

The test results were the following. The 25 lbs of TNT explosive equivalent caused relatively no damage to the top of the deck, but produced spalling approximately 3 feet in diameter and a little less than 2 inches deep, stopping at the transverse reinforcement. The 50 lbs of TNT explosive equivalent punched a hole through the thickness of the slab with a diameter of 20 inches on the top and 4 feet on the bottom. There was yielding of the reinforcement located directly beneath the detonation point, but relatively little residual displacement throughout the structure. The 200 lbs of TNT explosive devastated the specimen. A hole was created approximately 7 feet in diameter, and the concrete on the perimeter of the hole was fractured. Extreme yielding of the structure occurred, with multiple fractures of reinforcement and extreme residual displacement estimated at 22 inches at mid-span. Figure 1a shows the concrete deck before testing and Figure 1b shows the deck after the test with 50 pounds of TNT equivalent.

The finite element code ABAQUS was used for the analytical analyses of the field tests. The damage prediction curves were consistent with the 25 and 200 lbs TNT explosive equivalent tests on the reinforced concrete deck, but under predicted the damage inflicted from the 50 lbs of TNT


a


b

Figure 1a. Concrete bridge deck prior to the tests. Figure 1b. Underside of the concrete bridge deck after the test with the 50 pound charge. The damage crater is about 48 inches across.

explosive equivalent test. A numerical model was developed to predict the concrete spall resulting from the blast loading. Models were created using realistic upper and lower bounds for strain rate and failure criteria. The analytic model predictions of concrete spall were found to be consistent with the experimental results.

2.2 *Blast Testing of an Orthotropic Steel Deck*

Blast testing of an orthotropic steel deck was conducted at Naval Air Warfare Center Weapons Division at China Lake on a full scale typical orthotropic steel deck test specimen designed to the specifications for the girders in a self-anchored suspension bridge. The tests were performed to provide benchmarks for evaluating the performance of an axially loaded orthotropic steel bridge deck subjected to blast load.

The specimen consisted of five longitudinal closed ribs between three diaphragm plates, which were used to model the floor beams in the bridge ribs, in two spans. It was fabricated by Universal Structural Inc. (USI), following the Orthotropic Deck Testing Specifications document. The specimen was treated with the specified zinc primer on the top of the deck plate before shipping to the Pavement Research Center at Richmond Filed Station. A 50-mm (2 in.) epoxy asphalt concrete was then positioned on the deck plate with two courses, a leveling course and a surface course to the total nominal thickness. The epoxy asphalt concrete was cured at 125°F for seven days before it was shipped to the test site. The specimen was post-tensioned by two hydraulic jacks to 10 MN (2260 kips) before seating, which induces the axial stress of 97 MPa (14 ksi) at the deck plate. Multiple sensors (pressure transducers, displacement transducers, accelerometers, and strain gauges) were positioned on the test specimen.

A blast test was conducted on each span. Span 1 was tested with the 200 lbs TNT explosive positioned in the trunk of a car while the specimen was post-tensioned. Span 2 was tested with the 200 lbs TNT explosive positioned on a wooden pedestal at the same distance as the car trunk above the deck to simulate a free-air burst loading. In the second test, there was no post-tensioning force in the specimen. The pattern of damage after each blast test was similar: a hole was created at the center of three ribs by blast loading. However, the partially confined explosion (TNT stored in the car trunk) caused more damage for the rest of the bridge deck than the free-air-burst explosion. The blast load field tests showed that a 200 lbs TNT equivalent car bomb would create a 1.5 m × 2 m (4 ft × 6 ft) hole in the orthotropic bridge deck which can easily be covered with steel plates to restore functionality of the bridge. The subsequent diagnostic analyses showed that the observed field blast damage would result in a force redistribution around the hole in the actual bridge deck with stress levels in the deck still below yield immediately adjacent to the hole. Thus no local or global stability issues arose.

A buckling analysis of the main span of the suspension bridge was conducted in order to assess the extent that damage to the superstructure affects the buckling behavior of this Self-Anchored Suspension Bridge (SASB). The analyses were performed mainly using the finite element analysis program ABAQUS; however, the finite element programs SAP2000 and ADINA were used to verify the model of the SASB built in ABAQUS. The preliminary results from both an elastic buckling analysis and an elasto-plastic buckling analysis show that the global buckling behavior of the SASB is relatively insensitive to removal of sections of the deck and the roadway girders. A detailed global stability analysis of the SASB showed that unrealistically large portions of the deck or the box girder cross-section would need to be removed before global stability to the SASB under self-weight would become an issue. Thus, the modeled SAS span can be considered rather insensitive to blast loads on the bridge deck.

3 EXPLOSIVE LOADING LABORATORY (BLAST SIMULATOR) TESTING PROGRAM

The Explosive Loading Laboratory Testing Program, funded by TSWG, has constructed a hydraulic-based blast simulator to simulate explosive events without the use of explosive materials. The blast simulator is performing repeatable, controlled blast load simulations on critical structural elements

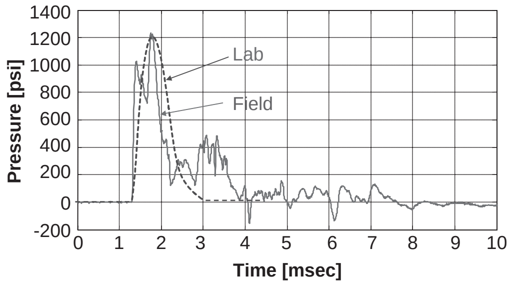

Figure 2. Typical pulse characteristics of field data.

such as columns, beams, girders, and walls, to characterize the progressive collapse of these structural components and systems when subjected to local or close standoff distance charges. It is also being used to investigate the response of bridge components such as decks, piers, and towers to simulated short standoff explosive loads. Standardized test protocols and test fixtures are being developed. Blast simulator data are validated by comparison with field test data. Simulator data are used to validate computational blast physics codes and models. New and existing blast retrofit designs including commercial-off-the-shelf technologies and carbon fiber reinforced polymers (CFRP) are being investigated in parametric studies. The CFRP retrofit technologies developed for seismic mitigation have proven especially effective in mitigating blast damage.

3.1 *Blast simulation*

For a large class of problems involving the explosive loading of structural elements, the energy deposition time associated with the explosive event is short when compared to the characteristic response time of the structure/element. For such events, the *impulse* (i.e., the integration of the pressure-time history) constitutes the most important loading parameter. An example is a "car bomb at the curb" where the energy deposition time can be as short as a few milliseconds (Figure 2).

The strategy for the laboratory simulation of a blast event is to impart an initial velocity distribution to a structure. For this purpose, a number of blast generator (BG) units are employed. Each BG is designed to produce the required initial velocity or impulse by a controlled impact of a mass onto the specimen. The main elements include a hydraulic actuator that can reach a high velocity in a relatively short stroke, an impact mass ("flyer plate"), and a "programmer", a polyurethane pad on the front. Different programmers are designed to produce the critical impulse characteristics of actual explosions. The hydraulic actuator accelerates the mass and programmer assembly to the velocity required for the desired transfer of energy and momentum while the programmer controls the shape (amplitude and duration) of the force-time history of the mass-to-specimen impact. Table 1 lists the performance specifications for the simulator.

The blast simulator is configured with one to four blast generators depending on the geometry of the article being tested. The impact velocities range from 1.5 m/s to 30 m/s to simulate typical energy deposition signatures of different live blast loads. The simultaneity of the hits of multiple blast generators is in the range of 1/3 ms to 2 ms depending on the test parameters. The force-time histories associated with the mass plate plus programmer-specimen impacts, as determined by acceleration time histories, reflect a pulse width of 2 ms to 4 ms depending on the programmer type and the impact velocity. Longer pulse widths are achievable with softer programmers.

3.2 *Comparison of laboratory and field tests*

Comparison of post test laboratory and field test data from similar tests conducted on similar column test specimens have revealed excellent correlation in failure mode, deformation, and impulse. Similar good correlations have been seen for CFRP hardening for columns, originally developed as seismic retrofit concepts, and for wall specimens subject to the same simulated charge size and standoff distance as in the live field tests.

Table 1. Technical specifications for the blast simulator.

Blast generator	
Quantity	4
Maximum energy (each) with mass:	
50 kg	30 kJ
100 kg	51 kJ
200 kg	76 kJ
400 kg	101 kJ
Maximum velocity (with 50 kg mass)	1300 in/s (34 m/s)
Repeatability of velocity	4% or 0.1 m/s whichever is greater
Simultaneous impact	Within 0.002 s
Impact Mass	
Mass range (bolted to BG rod)	50 to 400 kg (including mass of piston)
Mass range (free mass)	10 to 50 kg
Guidance of free mass	Guided by 1 inc diameter shift
Shape of mass	To match test specimen

4 STEEL CELLULAR BRIDGE TOWER SECTIONS FIELD AND LABORATORY TESTS

One of the main focus areas in the blast simulator program is the investigation of the structural response of the steel cellular towers of long span bridges to short stand off explosive loads. Long span bridges are high probability targets for terrorist attacks and the steel cellular towers are critical structural components. Testing is being done to characterize these structures against blast loads and to develop retrofit mitigation strategies. Full scale, high fidelity bridge tower section field tests are being performed at the Energetic Materials Research and Testing Center (EMRTC) of New Mexico Tech, our partner in the TSWG blast mitigation program. Blast simulator tests on similar test articles are being done at UCSD. Test results are being used to validate computational physics analyses and to verify blast mitigation retrofit strategies for these vulnerable national landmarks. A steel cellular tower bridge computational analysis supporting the lab and field tests is discussed in Section 5.

4.1 *Field tests*

Two field tests with test specimens that simulated a full scale section of a typical steel cellular bridge tower were performed at EMRTC in the last half of 2005. The two tests were designed to investigate the material properties and response of a single cell to an explosive load. The test specimens consisted of three cells each 3'-6" tall by 3'-6" wide and 8'-0" in length. Each cell was constructed of 7/8" A36 steel plates and was connected at the intersections via 8" × 8" × ½" angles and 1" diameter A307 bolts at 4" on center. In each of the two tests, the specimen was then loaded with 220 lbs of flaked TNT with a C4 detonator with approximately 6" of stand-off at the midspan of the center cell.

The two field tests differed in how they were connected to the foundation. In the first field test, the specimen was open on the bottom and had anchor bolts connecting the specimen to a flat concrete slab. The foundation of the second field test consisted of a flat concrete slab and raised pedestals that were 12" × 24" × 10'-0". The test specimen was then connected with anchor rods to the pedestals. The second test had an additional 3'6" × 3'-6" × 8'-0" long plate placed along the center cell. In the exterior cell a shorter plate was placed along the bottom of the specimen.

The results of both explosive field tests showed similar destruction and failure methods in the test specimens. In both tests, a flyer plate of approximately the same dimensions as the charge was created in the top plate of the center cell directly below the applied load. In the first test, the 2' square flyer plate was forced down into the foundation, bounced off of the foundation and came to rest approximately 20' away from the original specimen. In the second test, a similar flyer plate was created directly under the charge. This first flyer plate impacted the bottom steel plate of the test specimen, creating a second flyer plate. The speed of the first flyer plate before impact with the bottom plate was approximately 3,400 feet per second.

a b

Figure 3. Steel cellular tower section, pre-test (a) and post-test (b).

In addition to the holes left by the separation of the flyer plates, the test specimens showed significant deformation. In both tests, the edges of the hole in the top plate showed substantial petalling and tearing of the steel plate. The rupture of the top plate occurred in the center cell where the explosive load was placed. In addition to the rupture of the plate, the bolts and the angle pieces that supported the plate also deformed. The angle pieces stayed connected to the side walls of the center cell but the supporting leg was bent. The steel section seen in front of the specimen is what remains of the top plate where it came to rest after the second test. The damage that occurred to the specimen was basically limited to the cell that had the load placed upon it. Figure 3 shows the test specimen before (3a) and after (3b) the second field test.

4.2 *Blast simulator tests*

Blast simulator tests are planned on similar test specimens to obtain complementary data on the response of these structures to simulated blast loads. The test articles for the lab tests are slightly smaller in size and material thickness. The lab specimens are two cells wide with supports at the end walls to simulate the rigidity that a real bridge would provide. The two cells are 3’-0” tall by 3’-0” long and 2’-8” wide and are constructed of 1/2” A36 steel. These test articles are similar to the first field test article as there is no plate along the bottom of the specimen closing the cell. The connections of the plates are supported by 8” × 8” × ½” angles with ½” diameter A307 bolts at 4” on center. The cells are then impacted with a 16” square, 260 kg mass at the center of each cell allowing for both cells to be loaded simultaneously.

5 MODELING BRIDGE STEEL TOWER RESPONSE TO BLAST LOADING

The spate of recent terrorist attacks around the world has increased the need for a coupled Computational Fluid Dynamics (CFD) and Computational Structural Dynamics (CSD) methodology capable of modeling the response of targeted structures to blast loading. In response, several such codes were recently developed. Among these are DOE’s sponsored codes as ZAPOTEC (coupling of CTH and Pronto) and DYNA3D, and DOD sponsored codes such as NSWC’s DYSMAS and DTRA’s FEFLO. Commercial codes were also developed to fill the need. Among these are DYTRAN3D, LS-DYNA3D, and AUTODYN. Each of these codes has advantages, limitations and shortcomings.

Two approaches are used to model fluid/structure interaction. The so called ‘tight coupling’ approach that requires solving both CFD and CSD as one coupled set of equations, and the ‘loose coupling’ approach that decouples the CFD and CSD sets of equations and uses projection methods to transfer interface information between the domains. We adopted the latter method, as it allows us to use pre-existing, well-established, validated codes: FEFLO98 for CFD, and variants of DYNA3D for CSD. These variants include General Atomic’s GADYNA and ES3’s MARS3D. FEFLO98

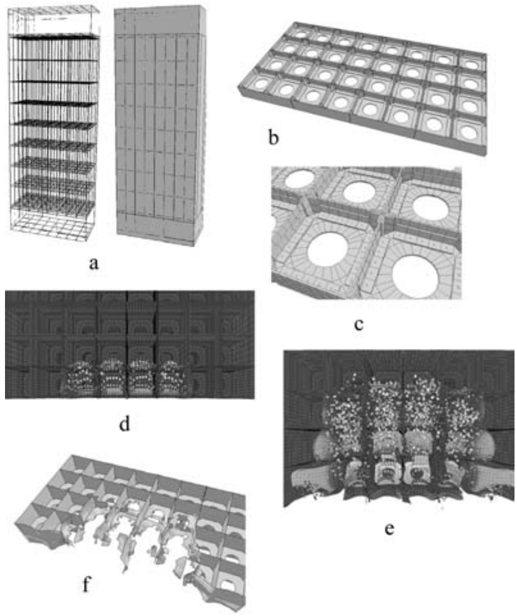

Figure 4. The eight level structure with expanded views of one level of the structure, CSD velocities at $t = 0.5$ and 2.5 ms, and a single CSD level at $t = 2.5$ ms.

solves the time-dependent, compressible Euler and Reynolds-Averaged Navier-Stokes equations on an adaptive, unstructured mesh of tetrahedral elements. DYNA3D solves explicitly the large deformation, large strain formulation equations on an unstructured grid composed of bricks and hexahedral elements.

The initial domain coupling of the CFD and CSD methods was based on the so-called "glued-mesh" approach, where the CFD and CSD faces matched identically. Failure of this approach to model severe structural deformations, as well as crack propagation in steel and concrete, led us to the development of the so-called "embedded-mesh" approach, where the CSD objects float through the CFD domain. While each approach has its own advantages, limitations and deficiencies, the embedded approach has proven to be superior for modeling severe structural deformations under blast and fragment loading. Critical applications of both approaches were conducted. These include weapon detonation and fragmentation, airblast interaction with a reinforced concrete wall, fragment/airblast interaction with a steel wall, and external airblast interaction with a generic steel ship hull.

Of interest here is one of the more complex simulations conducted: external blast interaction with a multi-chamber, multi-floor bridge-support steel tower. All dimensions and material properties for the compartments, rivets, reinforcements, plates and construction methods were modeled accurately according to industry standards and practices. The structure modeled here is only a small section of a generic steel tower supporting a suspension bridge. Only the section of the tower near the road (four sections above and below road level) was modeled, using appropriate loading boundary conditions on top (weight of upper tower section plus the suspension cable load) and bottom. The eight-segment tower is shown in Figure 4a. A single level construction is shown in Figure 4b, with an expanded view in Figure 4c showing the exact modeling of the structure, including all relevant details such as rivets and L-plates.

The modeling assumes a charge located on the roadway near the tower, about the mid-height of eight-section. The distance between the charge and the tower was determined by examining several bridges. The numerical methodology models the HE detonation initiation, the detonation wave propagation within the explosive, detonation products diffraction after detonation completion, blast load interaction with the structure, structural deformation, steel panel fragmentation and blast load diffraction between the accelerating fragments to load the next steel panel layer.

The results presented in Figures 4d and 4e show the CSD velocity contours at times of 0.6 and 2.5 ms, respectively. Figure 4f presents the damaged central-level structure at t = 2.5 ms. Although the maximum airblast pressure has been reduced by this time, a significant amount of kinetic energy is still contained in the moving structure and fragments, with the maximum fragment velocity of about 1200 m/sec. For this charge, most walls at the center-level were breached, resulting in tower collapse. Pressure contours show the blast wave diffraction about the tower, and the failure of the steel panels closest to charge.

The analysis shows a very interesting process, not often observed in blast-structure interaction modeling. Typically, one would expect to see a shock wave reflection from the impacted surface. However, in this case, because the surface failed so soon after blast wave impact, the resulting rarefaction wave caught up with the reflecting shock and eliminated it completely. The side view shows some weak reflections above and below the blast plane, as blast pressure amplitude weakens, and the surface does not fail as fast as on the blast plane.

6 COMPARISON OF BLAST AND SEISMIC HAZARDS

Blast mitigation research to date has shown that there are many similarities to earthquakes in the hazard and structural response. On the load side, both blast and earthquake loads are largely unknown in terms of location, magnitude, intensity, type, etc., and both have characteristics of rapid attenuation with distance from the source mechanism. In terms of consequences, both actions can result in progressive structural collapse, requiring redundant structural systems for mitigation. The critical local structural response, i.e., the performance of individual structural elements or members, is dominated by brittle shear failures in both cases and requires shear strength and ductility to prevent local failures. Finally, the load-structure interaction can modify the force and deformation response significantly. Thus, based on the above outlined similarities, a unified design and/or retrofit approach to multi-hazard mitigation seem reasonable. Table 2a presents a summary of the similarities between seismic and blast loads.

On the other hand, there also exist significant differences in these two extreme event scenarios, such as the load duration and associated strain rate effects and the extent of initial damage or excitation. In addition to strain rate effects, the blast load case also has a shattering or fragmentation effect on brittle materials for close-in charges and mitigation measures require containment considerations, something not paramount to seismic hazard mitigation. Table 2b presents a summary of the differences between seismic and blast hazards. Both the similarities and differences should be further explored to form the basis for multi-hazard mitigation in bridge structures.

Table 2a. Similarities between seismic and blast hazards.

Seismic	Blast
Input loading history unknown (location, intensity, duration, frequency content, fling)	Input loading history unknown (standoff, charge weight, type, modifications)
Attenuation of input	Attenuation of pressure
Strength required for shear	Strength required for shear
Deformation capacity required for flexure (ductility)	Deformation capacity required for flexure (ductility)

Table 2b. Differences between seismic and blast hazards.

Seismic	Blast
Entire system is affected	Local sub-assemblages are affected
Bridge response 1–2 sec	Bridge component response 0.1–1 sec
Strain rate effects can be neglected	Strain rate effects are significant
Duration 10 to 45 sec	Duration 1–2 msec
	Containment required against fragmentation and punching shear

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Bridge management: Actual and future trends

Joan R. Casas
Technical University of Catalonia. Civil Engineering Department. Barcelona, Spain

ABSTRACT: The paper presents the actual situation worldwide concerning the evaluation of condition state and assessment of remaining capacity of existing bridges, their maintenance options and the possibilities on repair and strengthening. The future trends and foreseen research needs in each one of those subjects related to the management of existing bridges are also discussed. Looking at the present situation, one of the main conclusions derived is that an emerging gap is appearing between the increasing needs in the management of the existing infrastructure and the educational skills that civil engineering students are receiving with the existing university curricula, mainly focused on the design and construction of new structures.

1 INTRODUCTION

A complete bridge management policy is a rather multidisciplinary task where subjects from the fields of structural engineering, computer science and economics have to be involved. The recent developed Bridge Management Systems (BMS) are very efficient tools where the mentioned fields are altogether combined with the final objective of the optimization of maintenance funds within an stock of existing bridges. All aspects (structural, computer science and economics) are of vital relevance for the good performance of such BMS. However, in this paper only the bridge management matters related to structural engineering will be discussed. The objective of the paper is not to give an overview of the actual situation and next future of the BMS as a whole, but only of one of their main components. In fact, in Thompson 2004 we may find a complete analysis of the future challenges of BMS and it is not our aim to repeat them here. In the paper by Thompson, is explained how much effort has been done at the network-level data, but at the project level still a lot has to be done. The present paper aims to focus more on the works related with this project level. The idea is to get an overview on where we are and where we go in the following aspects related to bridge management: inspection, assessment, maintenance and repair, and strengthening.

Moreover, the new professional and research challenges that the maintenance of the increasing bridge stocks puts into evidence, requires the education of young civil engineers with more advanced skills and knowledge of all aspects related to inspection, assessment and repair of existing bridges. Additionally, the higher number of existing bridges versus the new ones has derived in a situation close to the point where more funds will be necessary for the maintenance of existing bridges than for the construction of new ones, at least in the most developed countries. Despite disciplines as economics, computer science, electronics and concepts as deterioration modeling, life-cycle cost, etc. are becoming more and more important in the management of existing structures, this evolution has not yet taken place in the curricula of the courses offered by civil engineering departments worldwide. Only few civil engineering Departments and Schools around the world offer specific courses oriented to the service-life aspects of structures. As the stock of existing infrastructure systems is growing and growing, the question arises: are we really educating the new generations of bridge engineers in the matters that will be of more demanding requirements in the next future? Or a divorce is starting between the educational and social and practical needs of the future engineers? The aspects related to the educational demands on bridge maintenance and how they should be included in the future curricula of the civil engineering schools are also addressed in the paper. This educational effort is of paramount importance to reach one of the major missions of IABMAS, to bridge the gap between theory and practice.

2 INSPECTION AND CONDITION ASSESSMENT

2.1 *Present Status*

Different integrated projects have been developed in Europe at international level and funded by the European Union dealing with the subjects of inspection and condition monitoring of bridges (BRIME, COST345, LIFECON, SUSTAINABLE BRIDGES). The analysis of these projects jointly with the existing Codes and Standards available in different countries (USA, Canada, Japan) reflects the actual status in the assessment of the condition of existing bridges. The bridge condition is normally assessed via the condition rating or *condition index*, as a measure of the bridge condition by comparison with others. This condition index, at least, should be and indicator valid for:

1. primary ranking to screen the most deteriorated bridges
2. obtaining the capacity reduction factor (or *condition factor*) for use in the capacity assessment of the bridge.
3. assessing the tendencies of deterioration processes and providing a rough estimation of the expected service life using the condition rating calculated at successive time intervals.

The review of the methods used by different owners and administrations, both of highway and railway infrastructure, has highlighted the following items (Casas 2004a):

1. All administrations have a systematic inspection procedure (normally divided in 3 levels), but not all use the results of the inspection in a comprehensive and objective way to derive a condition rating, either numerical (in the range from 1 to 10, 1 to 100,…) or grammatical (poor, fair, acceptable, good,…).
2. At the present time, all administrations and transportation agencies use the visual inspection as the main source of relevant data to carry out the condition assessment.
3. Only in a reduced number of countries the code or guideline for the load capacity assessment is using directly the final result of the condition assessment in the capacity rating process.
4. There are basically two approaches to the evaluation of the condition of the whole structure based on the condition assessment of its elements. The first is based on a cumulative condition rating, where the most severe damage on each element is summed for each span of the superstructure, each part of the substructure, the carriage way and accessories. The final result is the condition rating for the structure. The second method uses the highest (or lowest depending on the measuring scale) condition rating of the bridge components as the condition rating for the whole structure itself.
5. Each administration and/or country is using different condition rating techniques. Such situation may derive in the fact that the same bridge, assessed by two engineers from different countries can be rated with different grades.
6. A clear division exists between methods that are purely subjective, those based on simple scoring, by assigning a number of deficiency points to the inspected structural member, in compliance with the rules adopted for the classification and evaluation of damage, and those where the final condition rating is obtained via a calculation where the rating of a set of selected essential damage types is done based on the report by the inspector.
7. Most methods divide the whole bridge in several parts or components and these components into elements. In some cases there is no clear or objective indication how to pass from the condition of the individual elements to the global condition rating of the bridge and this is made by the engineering judgement of the inspector.

Although important by itself, even more important than the condition index for a reliable management of the bridge stock, is the calculation or modelling of the *condition index deterioration* with time. Numerous models have been developed which describe the transition of a variety of condition states over time. Many of the models are based on a linear deterioration of condition states where the condition rating at any time *t* can be computed and the deterioration rate can be expressed in terms of condition rating loss per year. Hearn et al. 1995 compiled an extensive list of these models. Some are based on data while others rely on expert opinion. Actually, most of the existing BMS use the Markov chain approach to model condition deterioration.

2.2 *Future trends*

Future trends in this area search to incorporate the techniques based on the fuzzy set theory and neural networks for assignment of conditions based on expert's opinion and the results of visual inspection (Kawamura and Miyamoto 2003).

Regarding the future trends related to the prediction of condition deterioration with time, these are connected to the modeling of deterioration processes in bridge materials. This trend is directly linked to the adoption of a probabilistic framework for the assessment of bridge condition. In fact, Frangopol and Das 1999 have claimed the limitations of the condition index to characterize discrete condition states and the Markovian approach for deterioration modeling among others in the life-cycle cost management of existing bridges. Much effort has been done in the formulation of deterioration models, mainly in the case of chloride induced corrosion of concrete bridges, where several analytical, semi-empirical and empirical methods exist. However, at this moment, no conclusive model still exists as very often predicted values differ from the experimental results. Therefore, two main areas of research are still open First, the updating of existing models for chloride induced corrosion via structural monitoring of relevant variables (depth of chloride ingress (Rafiq 2005), corrosion rate (Mckenzie 2005)). Second, the development of models for other relevant deterioration processes present in concrete structures (sulfate attack, alkali-silica reaction).

Recently, a rising issue is the possibility of using the condition assessment database to quantify bridge deterioration instead of using mathematical models based on the physics of deterioration processes. In fact, considering that the condition rating at different service times has provided a time-stamped data series, statistical analysis may be an effective way of converting data into a kind of "knowledge" that can be used later on to predict deterioration. In the case of concrete bridges, attempts have been carried out to the calculation of condition index evolution using statistical causal relationship, combining logics and statistics to complement physical models. However, some authors (Thompson and Johnson 2005) have shown how the development of such models may not be an easy task when they are based in the analysis of historical data.

The use of non-destructive testing, monitoring and health monitoring in bridge condition assessment as an alternative and supplement to the actual procedure based on visual inspection is another area of increasing activity. The development of sensors based on the fiber optics technology, among others, has been of paramount importance in this sense (Casas and Cruz 2003, Villalba and Casas 2004) and application to real bridges is everyday increasing (Figueiras et al. 2005, Matos et al. 2005)**.** Two main topics are of major interest in this area: how to deal with the measuring errors, uncertainty and noise inherent to the different sensors and measuring techniques and how to integrate the data from the point-in time- NDT and continuous monitoring in the assessment of the condition state. It seems that using Bayesian-updating techniques is the best way (Faber and Sorensen 2002, Enright and Frangopol 1999 and Rafiq 2005).

How to prioritize maintenance needs to bridges that are most significant to the functionality of the network in addition to scheduling these interventions over the time horizon in order to achieve the overall cost effectiveness is another subject of future research (Liu and Frangopol 2005a). The concept of bridge network connectivity reliability appears as a measure of the degree of the satisfactory network performance. Open questions in this area are which should be the target (or minimum) value of the connectivity reliability index of the network and which criteria could be used to obtain it.

Contrary to the USA, where since long time ago a unified condition assessment procedure exists for all States, in Europe, at this moment, there are different condition assessment methods in each country. This is clearly a paradox situation taking into account that traffic may cross from country to country without any limitation. Therefore the harmonization between countries is needed in the next future.

3 SAFETY ASSESSMENT

3.1 *Present status*

Assessment of existing structures can be either deterministic or probabilistic. The review of existing methods mostly used actually by road and railway agencies (Casas 2004b) shows that they are mainly based on deterministic or semi-probabilistic (partial safety factors) assumptions. From the analysis of the methods used worldwide, the following conclusions are drawn:

1. Still few countries (UK, Denmark, USA, Canada) are using specific guidelines or standards for structural safety assessment of highway bridges. In European countries with a huge stock of highway bridges as France, Germany, Italy, Poland and Spain do not exist specific methods of structural safety assessment of highway bridges and, in general, the basis of assessment calculations is the same as for the design of new bridges. Specific codes for bridge safety assessment are not available too for Asian and South-American countries.
2. There is consensus that the structural safety assessment should be based on a Limit States format as already assumed in the design.
3. There is a consensus that the most efficient assessment process is based on the application of different and increasingly sophisticated assessment levels. A five level models is presented in BRIME, COST345, Highways Agency 1998 and SAMARIS. These levels of assessment, numbered 1 to 5 with Level 1 being the simplest and Level 5 the most sophisticated, are summarized in figure 1.

   This "step-level" philosophy has been also assumed in Germany (Hille et al. 2005) in the current preparation of a guideline for the assessment of existing structures and in the future European Guideline for the capacity safety assessment of existing railway bridges, actually under preparation within the SUSTAINABLE BRIDGES project.

<table>
<tr><th>Assessment Level</th><th>Strength & Load Models</th><th>Calculation Models</th><th>Assessment Methodology</th></tr>
<tr><td>1</td><td rowspan="2">Strength and load models as in design code.
Material properties based on design documentation and standards.</td><td>Simple, linear-elastic calculation.</td><td rowspan="3">LRFD-based analysis. Load combinations and partial factors as in design code.</td></tr>
<tr><td>2</td><td rowspan="4">Refined, load redistribution is allowed, provided that the ductility requirements are fulfilled.</td></tr>
<tr><td>3</td><td rowspan="2">Material properties can be updated on the basis of in-situ testing and observations using Bayesian approach.</td></tr>
<tr><td>4</td><td>LRFD-based analysis. Modified partial factors are allowed.</td></tr>
<tr><td>5</td><td>Strength model including probability distribution for all variables.</td><td>Probabilistic analysis.</td></tr>
</table>

Figure 1. General scheme of the 5 assessment level (BRIME).

4. A clear consensus also exists in the fact that the same resistance formulas for undamaged elements can not be assumed for deteriorated ones. However the consensus fails when dealing with how the influence of deterioration in damaged bridges should be considered.
5. A clear consensus on how the loading tests must be used in bridge assessment does not exist. Because experience with bridge testing has often revealed bridge behaviour which deviates from results expected from the conventional analytical methods, some Codes reflect the load testing possibility. It is generally accepted in many countries that load tests may be used as a supplementary source of information in the theoretical assessments process, providing information on the actual structural behaviour of the bridge. However, complete evaluation using only experimental load testing is normally not considered. The exception to this general rule is the LRFR evaluation Code in USA (AASHTO 2003).
6. It seems clear that target reliability levels different of those assumed in the design of new structures may be considered in the safety assessment of existing bridges. As an example, in table 1 the target reliability levels used in some countries and proposed by international organizations are shown. The lower reliability levels for assessment are justified in the fact that evaluation is performed for a much shorter exposure period (inspection every 2 to 5 years), consideration of site realities and the economic consideration of rating versus design.

3.2 *Future trends*

The actual and future trend in safety assessment is the use of reliability-based methods. Although there have been a number of application of reliability-based assessment for bridges in some countries (Lauridsen 2004, Casas 1999a, Casas 2000) and also some countries have developed standards, guidelines and codes for the evaluation of existing bridges based on probabilistic models (Canada, Denmark, Slovenia,USA), the practice is not yet widely used mainly due to the lack of information and standardization. An important step in the direction of the necessary standardization is the work developed by the JCSS (Joint Committee on Structural Safety 2001) in the Probabilistic Model Code. Initiatives like this will provide the needed basis for the increasing future application of reliability methods in bridge capacity assessment.

Another future trend where much research is needed is related to the following question: how to model in a capacity assessment calculation format the deterioration detected in the bridge during the inspection and summarized in a condition state? If the concept of condition factor is used, the question is how this factor can be derived from a condition index obtained basically from visual inspection. This condition factor multiplies the nominal resistance of the element evaluated with the real dimensions and strength. The general rule is to use the engineering judgement and there is no direct relation between the condition rating as derived from the inspection process and the condition factor. Only in Slovenia and USA (AASHTO 2003), there is a direct link between condition rating and condition factor.

A special issue which potentiality is recognised in many Codes and Guidelines, but that still has a lot of work to be developed, is the integration of load testing results in bridge assessment. Despite many countries already recognize the usefulness of *diagnostic load tests* in the updating of available theoretical models used in the evaluation process, by now, only the USA (AASHTO 2003) has standardized the possibility of using a load test to directly assess the bridge capacity or safety, via the so-called *proof load testing*.

Table 1. Target values of reliability index at member level for ULS and reference period of 1 year and normal consequences of failure.

	Canada	USA	Eurocode	JCSS	Denmark	ISO
Design	3.75	3.75	4.7	4.2	4.2	4.7
Assessment	3.25	2.5	–	–	4.2	4.7

4 MAINTENANCE AND REPAIR

4.1 *Present status*

Maintenance actions are normally taken to slow a structure's rate of deterioration, improve its performance, and lengthen its service life. The options range from preventive maintenance, developed long before deterioration becomes serious, to partial repairs or replacement of major components to prevent the structure from falling below a critical performance level (essential maintenance or repair). Maintenance actions have been traditionally undertaken by Bridge Agencies based on the condition state of the bridge, mainly on a daily basis and without a maintenance plan and long-term vision of the future state of the bridge stock as a function of the maintenance actions disposed.

Actually, in more countries the maintenance actions are planned on a long-term basis thanks to the adoption and development of BMS. The key issues to define are the performance indicators to be used as well as the modeling of the deterioration processes.

The life-cycle cost minimization criterion is the most widely used in maintenance management of civil structures and bridges (Hawk 2003).

4.2 *Future trends*

The main trends regarding the optimum bridge maintenance policy are the *accuracy of deterioration models* to predict the future condition state and safety and the *discount rates* used in the economic analysis of maintenance actions. The selection of the discount rate is extremely important as it can influence the selection of different maintenance options (lower discount rates favour higher initial costs and lower future costs). The adoption of an appropriate discount rate is a very difficult task and must be based on studies on life quality and societal investment (Ditlevsen 2004, Corotis and Gransberg 2005).

As a result of recent research, it seems that other issues of development in the near future concerning bridge maintenance are to combine essential and preventive maintenance, the use of time-and-performance based approach rather than only time-based and the use of more than one indicator (condition and safety) of bridge performance (Frangopol et al. 2001, Neves 2005). In order to get the safety and condition up to a certain threshold values considered as the minimum acceptable by the Agency, the trend will be to use a time-and performance-based intervention rather than only a time-based intervention in the case of essential maintenance. Further on, the time-and performance-based model has to combine a number of different maintenance actions (silane application, concrete repair,…) to obtain admissible results. This approach has been successfully applied to individual and groups of bridges in the United Kingdom and the United States (Frangopol et al. 2001, Neves and Frangopol 2005, Neves 2005). The use of more than one indicator for the bridge performance (safety and condition) leads to a multiobjective optimization problem which most of the cases is numerically solved by the use of genetic algorithms (Neves 2005). The first practical application of such an scheme using condition state evaluation and safety assessment has been carried out in the BMS developed in the Autonomous Province of Trento in Italy (Zonta et al. 2006).

Other future issues are the use of new materials more durability oriented in bridge maintenance and repair (Brühwiler et al. 2005) and the integration of monitoring in the maintenance tasks. In order to confidently predict long-term performance deterioration, it is essential to identify major deterioration mechanisms and simulate the resulting structural deterioration process. There is still a need for better deterioration models that may accurately predict the bridge condition in the future. Meanwhile these models are developed the alternative to a better estimation of future bridge performance is the use of long-term monitoring (structural health monitoring). The use of monitoring allows the up-dating of actual bridge performance as predicted by deterioration models and the up-date of the input parameters of deterioration models for future prediction as well (Aktan et al. 2000, Budelmann et al. 2005). Anyway, either to the fact that even the most sophisticated models will not be able to capture all variables involved in the phenomena of deterioration, or the fact that advanced deterioration models will need a very complicated set of input parameters to be effectively accurate, the application of monitoring with advanced sensors seems to be a helpful tool in the maintenance decision-making process. This is even more evident in the case of preventive maintenance. The efficacy of smart materials and systems has so far been mostly exhibited in tightly controlled experimental settings. Their long-term performance durability and functional reliability especially under extreme loads and aggressive environments is still uncertain and thus requires much investigation. A promising long-term monitoring scheme is based on the use of fiber optic technology (Casas and Cruz 2003, Villalba and Casas 2004) and MEMS (MicroElectroMechanical Systems).

The minimum life-cycle cost criterion is commonly used to determine a single optimum maintenance management solution. Recent studies show that cost minimization alone may not necessarily lead to long-term bridge performance levels adequate to meet bridge managers' specific requirements (Shepard et al. 2004). Therefore, more rational bridge management decisions should be made by considering the following aspects (Liu and Frangopol 2005a,b): (i) the actual structural capacity under deterioration should be accurately modeled; (ii) improving long-term structure performance and decreasing life-cycle expenditures need to be simultaneously considered; and (iii) maintenance management should be conducted from an overall network level perspective while ensuring satisfactory performance of individual bridges within the network and considering their relative importance on the network connectivity.

5 STRENGTHENING

5.1 *Present status*

The use of external prestressing (addition of forces) and the use of overlays of concrete or steel (addition of materials) are two well-consolidated techniques widely used in bridge strengthening.

There has been a big amount of research on structural strengthening using FRP composites in the last years. Different publications deal with the state-of-the-art on the application of FRP in bridges and structures (ACI440.2R-02 2002, The Concrete Society 2000, FIB 2001). A comprehensive and concise state of the art of the strengthening of concrete structures with FRP is presented in Teng et al. 2003.

5.2 *Future trends*

Despite many experiences are available and the number of interventions is daily increasing, the use of Fiber Reinforced Polymers (FRP) as a substitute of steel in the strengthening of bridge beams in bending or shear has not yet became a popular technique. This is mainly because generally accepted Codes of design do not yet exist and to high costs. Once the codification problem will be solved and the cost of FRP will become more competitive (from a life-cycle cost analysis), the use of FRP will be more and more used in the future. Peeling off and debonding are the most usual failure modes in the FRP strengthened structures. Therefore, most of the research work at the present moment is oriented to the derivation of analytical equations or experimental tests to accurately predict the maximum strain allowed in the FRP before debonding failure occurs (Ramos et al. 2004, Pascual and Casas 2005).

Issues still under research are the fatigue and long-term behavior and the response to fire and high temperatures of FRP strengthened structures (Breña et al. 2005). An issue that requires further investigation as well is the scale effect. In fact, most of the available theoretical models to predict the behavior up to failure of strengthened structures have been calibrated using results from laboratory tests with small specimens. Tests carried out on larger specimens, with dimensions closer to real structures (Casas 1999b, Casas et al. 2002, Ramos et al. 2004) have shown that a scale effect may exist related to the maximum strain where debonding occurs in the vicinity of a discontinuity (bending or shear crack). The models calibrated with tests in small specimens must be used carefully since the extrapolation of the results to real structures can result in values in the unsafe side (Pascual and Casas 2005, Casas and Pascual 2005).

6 EDUCATIONAL ASPECTS RELATED TO BRIDGE MANAGEMENT

6.1 *Present status*

Probably, the actual status of the education related to bridge management aspects is the less satisfactory. In fact, at this moment very few Civil Engineering Departments have specific courses on management of existing structures. The issues related to management (if contemplated) are normally incorporated in the more general courses where usually more emphasis is devoted to the design, analysis and construction of new structures rather than to the service life aspects. The current situation is therefore not good in the sense that current higher education is not facing the new problems that are appearing during the service-life period of structures. It looks evident that the real world and the practical needs for the new civil engineers are evolving much more quickly than the Academic Institutions.

6.2 *Future trends*

The most necessary action in the near future is the introduction of subjects related to management of bridges in particular and structures in general in the new curricula. Of course, the content of the new courses to be developed would integrate different disciplines related to management other than just structural engineering aspects (material sciences, economics, social sciences,…). The evaluator and managing engineer should have an education on such subjects, but always keeping in mind that those disciplines are not important by themselves, but what is important is their practical application to solve real problems faced by an infrastructure owner.

7 CONCLUSIONS

Bridge management is still a young discipline, in an initial phase of application. Many research works are being carried out worldwide on inspection and health monitoring, capacity assessment, life-cycle cost analysis and maintenance and repair. However, still a gap exists between the advances in research and their practical application by daily bridge managers. To help bridging this gap, efforts should be addressed in the following directions:

1. Development of simplified methods, based on the findings of advanced research but made more user-friendly to the managing engineer.
2. Need of codification and standardization in some specific areas as the condition assessment and the use of new materials and techniques for bridge repair and strengthening.
3. Introduction in the university curricula of specific courses oriented to the education of a new generation of engineers with a solid basis regarding the management of existing infrastructure, besides the knowledge in design and construction.
4. Interaction and cooperation between the academic/research and professional environment. This is the most important mission of IABMAS.

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Life time assessment of bridges

U. Peil, M. Mehdianpour, M. Frenz & K. Weilert
Institute for Steel Structures, Technical University Braunschweig, Germany

ABSTRACT: The prediction of a realistic life cycle and the prolongation of the service life is an important task to reduce costs of civil engineering structures in the future. The precise assessment of the life cycle will become an important challenge. This paper gives at first an overview about the possibilities of assessing the state of the structure (the anamnesis) which must precede every life cycle determination. Subsequently, new ways and possibilities of precise life cycle determination are presented. The methods are developed at the Collaborative-Research-Center "Monitoring of Structures" at the Technical University at Braunschweig (Germany).

1 INTRODUCTION

The usual life cycle assessment methods are not very reliable (Schuetz 1994). Differences between theoretically calculated and observed life cycles may differ in an order of magnitude. The life cycle assessment by means of parallel structural health monitoring (SHM) can considerably improve the accuracy of the prediction. SHM measures should be included in the design process of new structures as it is standard for important devices such as turbines, reactors etc. concerning the field mechanical engineering. In Germany some large bridges with hollow cross sections are no longer corrosion protected. The humidity of the air inside the hollow section is monitored. If the air humidity reaches a threshold, air drying devices are activated. This measure is remarkably cheaper and more effective than (repeated) corrosion protection. Beside the economic benefit SHM in civil engineering helps to reduce potential damages. Increasing damage rates of bridges built in the sixties of the last century in particular show severe problems.

2 OVERVIEW OF THE PROCEDURE

Figure 1 shows a flow chart of the life cycle assessment by means of SHM. At the beginning of any monitoring measure a conscientious check up of the state of the bridge and its accumulated damage is indispensable. This first inspection should go from an overall examination to a more detailed examination of the details; one should not only be focused on visible damages or faults but also on symptoms and damage indicators, as e.g. local changes in the color of coatings.

During this first inspection doubts can be cleared up or increased and suggestions can be made for further action. If one decides to perform a precise anamnesis, first the global system geometry and the local geometry must be determined for the subsequent modeling of the structure. Very old structures are often problematic because their technical documents, as drawings, calculations are not available: the geometry has to be determined on site. Different methods are available. A very efficient one is the laser scan method. A laser scanner measures the horizontal and vertical angles and the distance using rotating reflectors. The measurement speed is high. Up to 800,000 points in space can be sampled in a second. The accuracy depends on the distance; values between 1 m and 0.1 mm are possible.

The next step is the determination of the material parameters. Non destructive testing techniques are only useable for rough estimations. If more precise material parameters are needed drilling

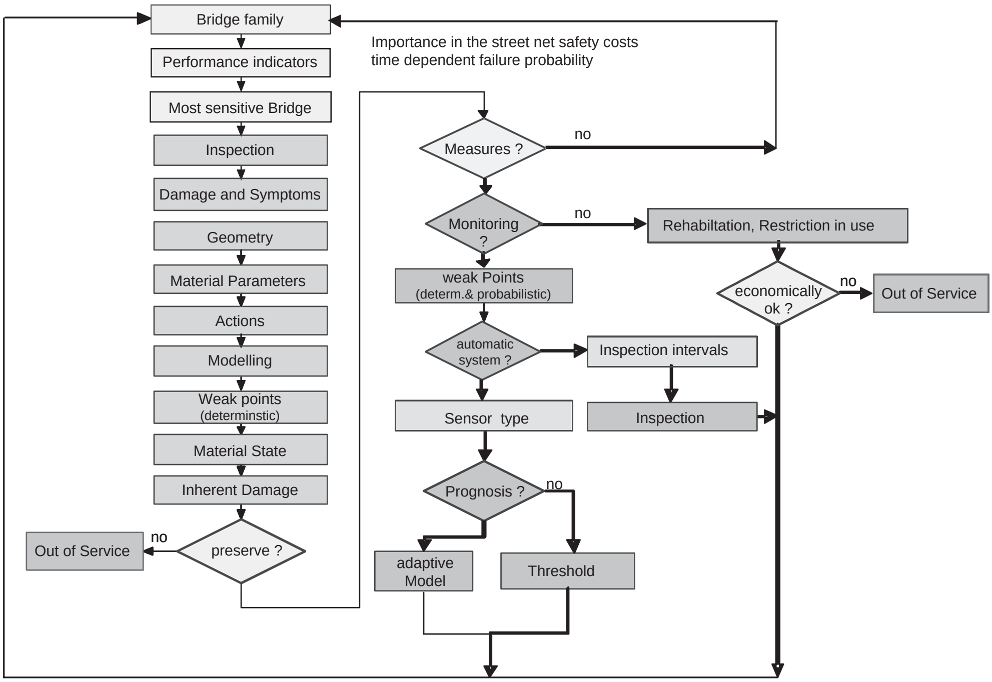

Figure 1. Overview of the procedure.

cores must be taken from less strained areas near the weak point. The size of the drilling core is a problem: desirable small cores show a remarkable size influence, small local defects as pores, slag inclusions, concrete aggregates already show over proportional influence on the behavior of the specimen. Concrete drilling cores usually have diameters three to four times the maximum aggregate (see DIN 1048). Steel drilling cores with a diameter of 50 mm to 60 mm can be used to realize a tension test, an ISO-V-specimen and a modified CT-specimen to determine fracture mechanical parameters. The remnant can be use for the determination of the chemistry of the material (Peil 2005).

An essential prerequisite to determine the strains of the structure is the determination or definition of the environmental effects, as loads, temperature, chemistry of the air etc. For a simple first step assessment of the state of the structure, simple and safe side definitions are sufficient. Dynamic effects should be taken into account, e.g. by use of dynamic amplification factors in this first step, because they could alter the response remarkably.

Now we are in a position to model the structure: geometry, material and loads are known and the local stresses and strains can be determined. This must be done with high accuracy, because if the structure is modeled in a too simple manner, this can hide or fake weak points in the structure (Nather 1993). Experience teaches that use of the Finite-Element-Method (FEM) will give the best results. Figure 2 shows for example the response of an old railway bridge calculated by FEM. The overall structure is modeled via beam elements and the local interesting points are refined using volume elements. The weak point is a very local stress concentration. This point shows the maximum stress compared to all other details.

The overall material state near the critical weak points shall then be checked by non destructive techniques (NDT) as e.g. ultrasonic, X-ray, magnetic tests etc.

In many cases the result of this first step evaluation of the structure shows that no further measures must be taken, or that the structure should be preserved. If the result is unfavorable concerning the expected life cycle, monitoring measures together with adaptive prognosis models could help considerably.

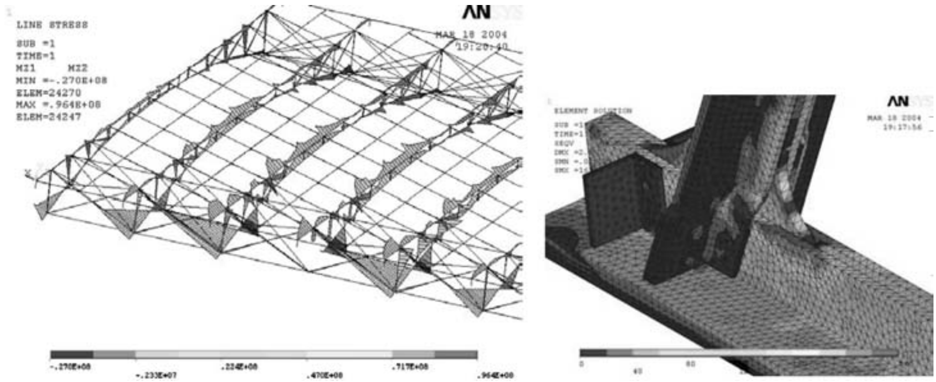

Figure 2. Railway bridge with locally refined modelling.

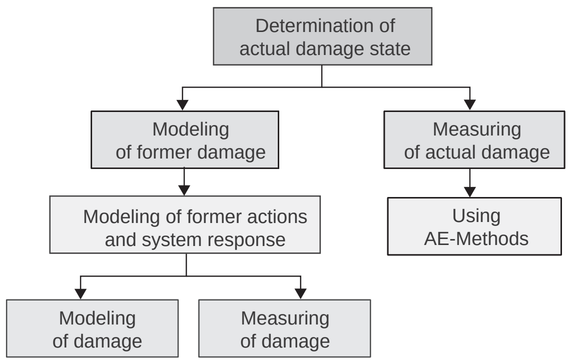

Figure 3. Determination of the actual damage state.

3 INHERENT DAMAGE STATE

The assessment of the inherent damage is one of the most difficult parts of the whole procedure. There are two possibilities for determining the damage state, a more theoretically based and an experimentally based procedure, see Figure 3. Both methods are described below.

3.1 *Measuring of the inherent damage*

The acoustic emission method (AE-Method) is used to assess the current damage of a structure. Apart from the classical way of analyzing amplitude, duration, number of threshold crossings etc., the entire transient signal is taken into account. It is assumed that different material processes lead to different transient signal forms. Thus the significant features of a signal are evaluated and, based on this features, the signal is assigned to a certain class of signals. These can be assigned to a certain material damage process, which is characteristic one for the actual damage state.

The outset range of an AE-signal contains the most substantial information about its source, because reflections are not yet superimposed. This initial range is, therefore, used for the extraction of features. The comparison of the feature vector with average values of features of signal classes defined before, provides for classification into these associated signal classes. The statistical spread of the class features are taken into account by the Mahalanobis distance classifier.

Single step tests with steel S355-J2G3 show characteristic signal forms for certain life span phases. The evaluation of acoustic emission activity over the number of load cycles of a test with strain amplitude of ± 3 micro strains, is used as example in Figure 4.

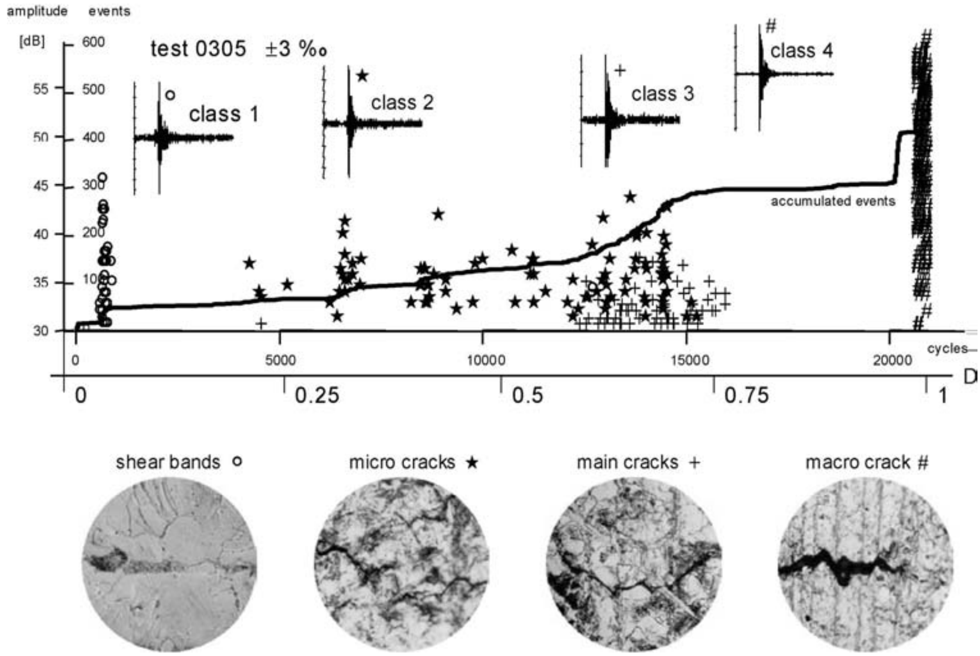

Figure 4. Correlation of signal classes and damage state.

The abscissa represents the number of cycles and the dimensionless damage variable D. The ordinate shows both the accumulated number of events and the amplitude of typical measured and classified events. These events are represented by different symbols according to their class. Four different stages of acoustic emission activity can be observed: an initial very short phase, which seems to be coupled with the formation of shear bands. The acoustic emission activity then continues with the evolution of micro cracks. A further signal class is assigned to the evolution of main cracks. At the very end of the life time the number of events rises strongly, due to the growth of macro cracks, which is accompanied by a large number of typical signals with wide spread amplitudes.

3.2 *Modeling of the inherent damage*

The actual damage of a structure can be assessed theoretically via a model (left branch of the flowchart in Figure 3). As an input the loading history of the bridge is needed as a prerequisite of the determination of the damage, which could be performed by means of the well-known damage models or more experimentally oriented as it will be shown later on. First the time history of local strains must be estimated or generated by a model. The generating method consists of two procedures: a load generating procedure and a strain generating procedure (Peil et al. 2001b).

Traffic data and axle loads are usually known by traffic authorities for different regions. Also WIM data on a lot of structures are available nowadays (Moses 1979) and (Tatsuya 2000). If measurements are not available (which is normal), re-extrapolated or estimated load probability distribution functions should be used.

The type of the micro traffic (typical vehicle sequences, clustering of trucks, typical temporal distances between vehicles) must be taken into account as well, because it influences the fatigue state significantly. The simplest way is to start from the present, i.e. to measure the actual micro traffic and to extrapolate it into the past.

In order to take typical load sequences and typical temporal distances between single loads into account, the measured time history is classified into four groups which represent the four types of vehicles mentioned above (Peil 2001b). Temporal and sequential data of this stream are

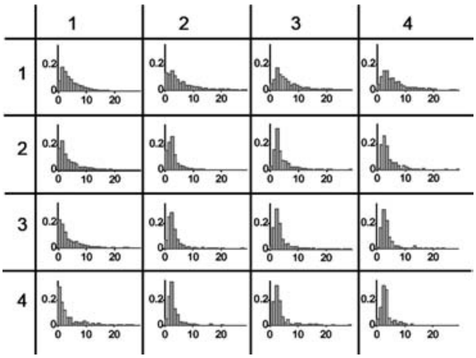

Figure 5. Densities of temporal distances.

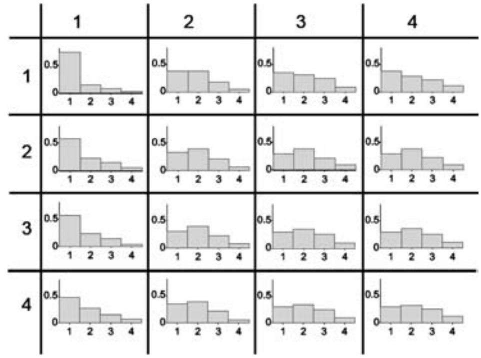

Figure 6. Densities of sequences.

representative of loading scenarios of the structure which depend on the location of the structures (e.g. uphill road) and on human driving behaviour. Figure 5 shows the density functions of the temporal distances in seconds between two successive vehicles sorted in a matrix. The row of a matrix element indicates the actual vehicle type and the column the next one.

To identify typical sequences, three successive vehicles are usually considered. The first two vehicles are used to express the occurrence probability of the third one. The distributions of the sequences are stored in the matrix of Figure 6. The row i of the matrix indicates the first vehicle of the actual sequence, the column j the second (the recent!) one. The histogram of element i, j (sequence i, j) shows the probability of occurrence of the next vehicle of that sequence. It is obvious that the probability of occurrence of succeeding trucks is high (see the high mode of the distributions in elements with a column number greater than 1) (Peil 2001b).

The statistical input information are now used to generate a synthetic time history of the traffic of the past, using the Monte-Carlo-Method. A random generator produces vehicle types which always consider the most recent sequence generated on the basis of the densities. Temporal distances are chosen in the same manner according to the densities. The actual load of a vehicle is then determined on the basis of the corresponding density of the forecasted vehicle type.

The strains in the structure must be determined by means of a precise model, preferably a FE-Model. An additional dynamic influence develops from the roughness of the road surface, i.e. the pavement (Peil 2001b, Drosner 1989). This influence is taken into account using measured different pavement states. These are described by means of a Gaussian, stationary, ergodic process, characterized by a power spectral density function (PSD). The different types of vehicles are idealized as damped 2-mass systems for each wheel. They consist of a wheel mass, a mass containing the corresponding part of the vehicle body and pay load, spring and damper between both and another spring representing the tire tangent stiffness.

4 PROBABILISTIC ASSESSMENT OF CRITICAL WEAK POINTS

Independent of the type of the structure to be monitored, the assessment of critical weak points is one of the most important points in the procedure. Finding critical points of the structure is one of the main tasks in monitoring. Weak points or weak spots are areas of the structure which are prone to damages or where possible damages cause non tolerable consequences. Weak points of older structures, usually designed with very different safety levels, are normally well known and can be determined by existing structural calculations or by experience. New structures however show an equally distributed safety level over a high number of critical details. Because of the increasing costs for every new detail to be monitored, the critical details can be detected using probabilistic methods, leading to a lower number of measuring points. The procedure for reliability-oriented determination of weak points classifies critical weak points as those which contribute the largest part to the overall failure probability of the structure. These points must be monitored. To determine the failure probability the description of the limit state functions, limit values, stochastic model, mechanical model, analysis of sequences of events, fault trees are prerequisites.

In (Hosser 2003) a concrete structure is investigated such, in (Peil 2003) a steel structure with fatigue damage demonstrates the use of the probabilistic method. The procedure is now upgraded to automatically perform the overall analysis of a fatigue endangered structure.

5 CONTINUOUS AND AUTOMATIC OR DISCRETE MONITORING

After identifying the weak points it must be decided, if a continuous and automatic monitoring shall be performed, or if a time discrete monitoring is sufficient. Often the last possibility is the cheapest one. The problem is in which time intervals the discrete monitoring of the structure shall be performed.

Inspection intervals could be determined using the reliability method. First all relevant damage developments (cracks, chloride seep etc.) must be modeled as a stochastic process just as load and other environmental processes. Using a mechanical model e.g. the FE-Method the statistical parameter near the weak points can be determined. By means of a damage model, e.g. the Paris–Erdogan crack propagation model used in fracture mechanics, the damage at a point t_i in time can be determined. The structures fails if the expected size of the damage (e.g. crack) $\tilde{a}$ is greater then the critical value a_{crit}.

The expected size of the damage depends furthermore on the statistical values of the strength at the site of the damage, i.e. the mean value and the rms-value and probably from higher statistical moments. In addition the probability with which the damage could even have been detected must be known. Small cracks will be detected with less probability than large ones. Details of damage detection probability are very rare in general. An exception is the detection of cracks in steel or alloy structures. A model of the probability of detection with an exponential distribution is given in (Sindel 1989) and (Soerensen 2001).

With these values it is possible to describe the detection of a crack at the inspection time t_{insp} by a comparison of the detectable crack length adet with the expected one $\tilde{a}$ (t_{insp}).

6 DECISION ON THE MONITORING STRATEGY

The decision on the most effective monitoring strategy depends on both the demands and knowledge of the structure. If the knowledge is incomplete, then a prognosis of the life cycle using an adaptive model is useless, because the input parameter of the model is more or less unknown. Monitoring can then only be used to observe threshold values of defined damages or damage symptoms of important details which are essential for the structure, such as limit stresses or strains, cracks, crack lengths, crack lengths rates, corrosion depth, etc.

If the knowledge about the structure is more complete, due to a conscientious anamnesis and a good knowledge of the former environmental impacts and loads, then a life cycle prognosis can be performed by means of a model. The model should be an adaptive one, i.e. the input parameter should be directly measured on site. The model input parameter can be continuously updated and will give the best forecast. Adaptive models could be the same as usual models though on the other hand they could be much simpler than the usual ones. The usual models must take into account many more external and internal variables to give a sufficient, reliable prognosis of the life cycle. In chapter 7 an adaptive procedure is described which does no longer needs any model.

7 ADAPTIVE MODELING BY MONITORING AND TESTING IN THE LAB

The prediction of the life cycle of a steel bridge under repeated loads is usually performed theoretically on the base of a model chain. The prediction model chain used consists of a load model,

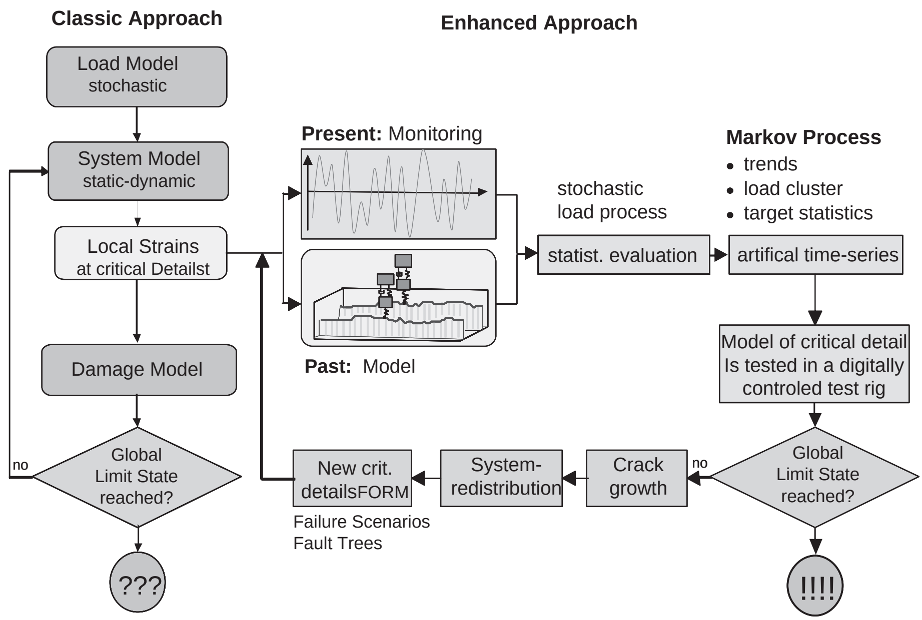

Figure 7. Flowchart of the experimentally oriented procedure.

a system-transfer model and a damage model. Every model shows more or less greater random and systematic errors. Because the results of one model serve as an input for the next model, the systematic and the random errors of every model are multiplied. Thus the results of this model chain is rather unreliable, especially the influence of the uncertain load and damage models controls the final reliability of the result.

The uncertainties of the above mentioned three models can be minimized when using monitored data of the random strains at the critical points and additional tests in the laboratory. The method can be used for new and for existing structures as well. The reliability of the result is high.

In a first step, the random strains at critical constructional details are monitored. Due to the monitored strains, a load model and a system model is not required any more, thus the uncertainties of both models are avoided, see Figure 7.

To avoid the damage model, an artificial i.e. generated time history is generated using a so called snake algorithm. It is used as an input for a digitally controlled test rig, in which a sample of the actual constructional detail is tested. The artificial time history contains the overall statistics of the load process, taking into account the past and the estimated future of the loading situation including e.g. clustering of trucks and traffic jams, see chapter 3. If the critical detail is small, the detail can be tested in full scale. If large or complex shaped details have to be monitored, it is sufficient to look at the hot spot. The hot spot must be determined before by means of the FEM. The preciseness of the method is higher in an order of magnitude compared with the usually used methods (Peil 2005).

8 MEASURES AS A RESULT OF MONITORING

Depending on the result of the monitoring measure a decision can be made about the future of the structure. Possible measures could be: maintenance, repairs, restrictions or extension of use, shortening of discrete inspection intervals, closing of the bridge. The decision about the future of the structure must include not only technical and economic aspects but also the associated legal background. Building regulations and laws, public laws and product liability can easily become problems which result in the abandonment of the decision to repair a structure, causing the structure to be demolished.

9 CONCLUSIONS

The procedure for the assessment of bridges is demonstrated. First a conscientious check up of the structure is necessary. Main points of this step are the determination of the actual strains and of the inherent damage of the structure. In many cases the life time of a structure can be extended by means of monitoring. The decisions and steps which should be taken are explained. If a prognosis of the life time of the bridge should be determined, adaptive models can be used. These models can include results from parallel lab experiments.

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Cost-effectiveness of seismic bridge retrofit

Masanobu Shinozuka, Youwei Zhou & Swagata Banerjee
University of California, Irvine, CA, USA

Yuko Murachi
Kozo Keikaku Engineering Inc., Tokyo, Japan

ABSTRACT: Past experience showed too often that earthquake damage to highway components (e.g., bridges, roadways, tunnels, retaining walls, etc.) can severely disrupt traffic flows and thus negatively impacting on the economy of the region as well as post-earthquake emergency response and recovery. Furthermore, the extent of these impacts will depend not only on the nature and magnitude of the seismic damage sustained by the individual components, but also on the mode of functional impairment of the highway system as a network resulting from physical damage of its components. In order to estimate the effects of the earthquake on the performance of the transportation network, an analytical framework must be developed to integrate bridge and other structural performance model and transportation network model in the context of seismic risk assessment.

Among the engineered components, bridges represent potentially the most vulnerable components under earthquake conditions as demonstrated as vividly in the San Fernando, Loma Prieta, Northridge and Kobe Earthquakes. Recognizing this, the California Department of Transportation's (Caltrans) seismic retrofit program has been underway since the 1971 San Fernando Earthquake, and accelerated since the 1989 Loma Prieta event. At this time (2005), 23% of Caltrans freeway bridges in Los Angeles and Orange Counties have been retrofitted by the steel and composite jacketing of the columns as well as rebuilding and upgrading of the restraining devises at expansion joints for which the seismic retrofit was deemed necessary. It is therefore most timely at this time to assess not only the engineering significance of such retrofit but also the socio-economic benefit arising therefrom. In order to estimate the effects of the earthquake on the performance of the transportation network, an analytical framework must be developed to integrate bridge and other structural performance model and transportation network model in the context of seismic risk assessment.

The purpose of this research therefore is to assess the socio-economic impact of seismic retrofit implemented on the Caltrans' bridges on the freeway network in the Los Angeles and Orange Counties. The research concentrates on the evaluation of the socio-economic benefit resulting from the retrofit performed on the Caltrans' bridges primarily by means of column jacketing with steel. The three major tasks of this research are (1) development of fragility curves of the bridge, (2) assessment of the seismic performance of the freeway and (3) related socio-economic analysis.

In order to perform a seismic risk analysis of a highway network, it is imperative to identify seismic vulnerability of bridges associated with various states of damage. As a widely practiced approach, the vulnerability information is expressed in the form of fragility curve to account for a multitude of uncertain sources involved (Shinozuka et al. 2000). A manageable number of representative bridges are selected for the fragility analysis (Kim and Shinozuka 2004). Nonlinear Finite Element Model (Figure 1) for each of the representative bridges, without or with retrofit (column jacketing with steel) is developed and used to perform nonlinear dynamic time history analysis. Based on the result of this dynamic analysis, a family of fragility curves associated with various states of damage are estimated with a statistical procedure. The seismic performance improvement of the retrofitted bridges is evident in that the median value of fragility curve of these bridges is

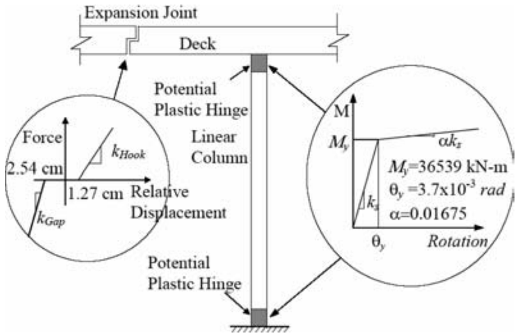
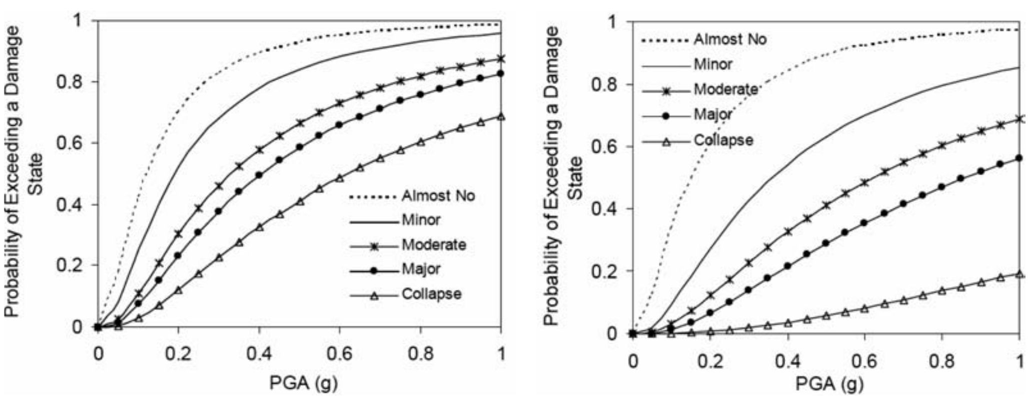


Figure 1. Nonlinearities in bridge model.

Table 1. Fragility parameters of bridges before and after retrofit (oblong columns).

	Median Value in Different Damage States (g)									
Br.	Almost No		Minor		Moderate		Major		Collapse	
#	Before	After	Before	After	Before	After	Before	After	Before	After
1	0.12	0.15	0.19	0.36	0.33	0.62	0.40	0.86	0.62	2.31
2	0.44	0.44	0.55	0.67	0.88	1.30	1.18	1.99	1.83	2.08



Before retrofit After retrofit

Figure 2. Fragility curves of Bridge 1 before and after retrofit (oblong column).

significantly increased (Table 1). The median value is one of the two fragility parameters with the other being the log-standard deviation. The enhancement ratio is defined as (median value for retrofitted bridges)/(median value for bridges not retrofitted). Figure 2 plots the enhancement curves of bridges averaged over all types of columns, which shows 40%, 55% 75%, 104% and 143% improvement for each damage state described on the x-axis. The enhancement ratios for median values of analytical fragility curves for all types of columns are then applied to empirical fragility curves based on bridge damage data obtained from the 1994 Northridge Earthquake to consider the effect of the bridge retrofit (Table 2).

After the introduction of major features of seismic risk analysis for spatially distributed system, seismic modeling methods are described. Particularly, a set of 47 probabilistic scenario earthquakes is provided and used for representing the regional seismic hazard for the probabilistic seismic risk

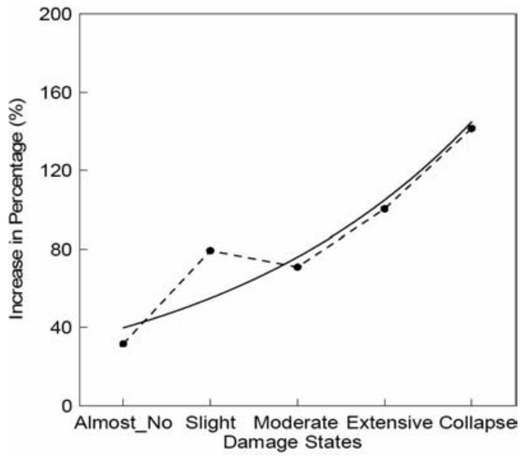


Figure 3. Interpolated enhancement curve of bridge retrofit (all types of columns).

Table 2. Parameters for empirical fragility curves (composite).

Fragility type	Median value in different damage states (g)				Log-standard
	Minor	Moderate	Major	Collapse	Deviation
Before retrofit	0.64	0.80	1.25	2.55	0.7
After retrofit	0.99	1.40	2.56	6.19	0.7

Table 3. Assumptions for link residual capacity.

Link damage	Link residual capacity ratio (%)		
	Assumption 1: Low	Assumption 1: Moderate	Assumption 3: High
No	100	100	100
Minor	100	100	100
Moderate	75	50	25
Major	50	25	10
Collapse	50*	25*	10*

*Local detour route considered.

analysis for the highway transportation network in Los Angles and Orange Counties (Chang et al. 2000).

A methodology is developed to evaluate the seismic performance of highway transportation network in terms of related social cost. Based on fragility curves developed above and the site ground motion originating from scenarios, the damage states of bridges are simulated, which determine the reduced link traffic capacity by bottleneck principle (Table 3). A comprehensive traffic assignment analysis, which features realistic consideration of trip reduction and recovery after a damaging earthquake, is then performed in the degraded highway network with variable OD input. The daily social cost, including the traffic delay time and opportunity cost, is used to measure the post-event performance of the damaged highway network. The enhancement of the network performance is then studied by comparing the social cost in using fragility curves of bridges before and after retrofit in the network performance simulation under the same scenario earthquake.

The analysis applies a comprehensive index of total transportation cost (drivers' delay), λ, based on post-earthquake network topology relative to pre-earthquake intact conditions. Drivers' delay is defined as:

$$\lambda = \sum_a x'_a t'_a(x'_a) - \sum_a x_a t_a(x_a) \tag{1}$$

where x_a = flow on link a in intact network (pre-earthquake); t_a = travel time on link a in intact network (pre-earthquake); x'_a = flow on link a in damaged network (post-earthquake) and t'_a = travel time on link a in damaged network (post-earthquake).

Reduced travel demand is an impact from the earthquake, assumed in this analysis via building damage, and it implies another type of social cost. Trip is derived from various activities, such as working, and shopping. If drivers cannot make trip in any reason, they also cannot achieve the purpose of activity that used to cause the trips. If the activities they used to perform have any economic value, they lose the value by not making the trips. And the value of this loss, called as opportunity cost, should be included in total cost, along with the cost from drivers' delay. Opportunity cost of trip type p, ϕ^p is calculated as:

$$\phi^p = \sum_i \sum_j \left(\frac{\left(q_{ij}^p - q'^p_{ij}\right) \cdot \left(c'^p_{ij} - c_{ij}^p\right)}{2} \right) \tag{2}$$

where q_{ij}^p = trips of type p from zone i to zone j in intact network (pre-earthquake); c_{ij} = travel time zone i to zone j in intact network (pre-earthquake); q'^p_{ij} = trips of type p from zone i to zone j in damaged network (post-earthquake); and c'_{ij} = travel time zone i to zone j in damaged network (post-earthquake).

The estimation of bridge restoration (repair/replacement) cost is based on the bridge replacement value and its damage state. For the given scenarios, the expected bridge restoration cost is calculated for each of the 3 cases of bridge retrofit status: No retrofit, 23% retrofit (current status) and 100% retrofit, assuming that no freeway bridges (in Los Angeles and Orange Counties), 23% of them (actual % at the time of writing this report) and 100% of them have been retrofitted (Table 4).

To estimate the total social cost resulting from an earthquake, the network restoration curves are developed. Using a probabilistic time-dependent bridge repair model (Table 5), the new set of bridge damage states are determined based on Monte Carlo simulation at any given time point after an earthquake. At the same time, a time-dependent OD recovery model is also considered. The traffic assignment analysis is performed again to obtain the corresponding daily social cost for the partially restored network. The integration of the daily social cost over the restoration period gives the total social cost in time for a particular earthquake event. The economic loss due to the time cost is estimated by considering the local unit time value. The HAZUS bridge functionality restoration model is also used for network performance restoration simulation (HAZUS 1999).

Finally, whether a retrofit strategy is cost effective is evaluated by a cost-benefit analysis. The restoration cost (Table 4) for the damaged bridges, the retrofit cost and economic loss due to social cost are estimated. The difference between the economic loss without and with retrofit represents the cost avoided. The economic benefit is then measured by the cost avoided minus the cost of retrofit. The economic analysis is performed for each of the probabilistic scenario earthquakes and expected annual benefit of the retrofit measure obtained by considering the annual probabilities of these scenarios. The cost-effectiveness of the retrofit is expressed in terms of the ratio of the present value of the cost avoided to the retrofit cost. Obviously, the larger this ratio, the more cost-effective the retrofit. Table 6 list total retrofit, total social cost avoided and total bridge restoration cost avoided and evaluate the cost-effectiveness ratios in retrofit Case 2 (current retrofit status with 23% of all the bridge are retrofitted) and Case 3 (all bridges are retrofitted).

We can observe that the cost-effectiveness ratio in terms of bridge restoration cost avoided in case 2 is bigger than in case 3, but the ratios in both cases are much lower than 1 (in fact, less than 0.1). It shows that the retrofit is not cost-effective if it is only for reducing bridge restoration cost.

Table 4. Expected bridge restoration cost.

Event no.	Case 1	Case 2 ($ Million)	Case 3	Event no.	Case 1	Case 2 ($ Million)	Case 3
1	186.89	144.10	38.17	25	0.01	0.01	0.00
2	235.57	177.94	57.59	26	15.94	12.53	3.38
3	162.32	134.60	35.02	27	46.70	38.01	11.80
4	104.21	88.55	20.93	28	34.93	24.01	7.62
5	76.39	65.45	14.62	29	28.59	19.23	6.67
6	183.21	129.40	45.29	30	9.48	7.53	1.49
7	37.83	29.59	6.28	31	146.22	101.98	35.57
8	3.43	2.59	0.37	32	131.96	91.10	31.44
9	58.11	45.90	14.41	33	137.30	101.30	32.84
10	240.41	176.99	58.90	34	92.95	80.73	19.15
11	104.13	81.04	23.99	35	57.15	45.82	11.14
12	207.04	149.76	51.64	36	25.67	21.72	4.58
13	144.75	115.27	28.73	37	2.62	2.22	0.53
14	39.84	26.70	8.59	38	0.52	0.42	0.07
15	87.19	60.25	20.45	39	0.07	0.05	0.01
16	62.33	45.29	13.59	40	76.54	60.15	18.90
17	46.26	42.19	8.99	41	24.50	19.79	4.19
18	34.26	29.38	6.34	42	216.17	161.57	53.49
19	31.20	25.10	5.86	43	260.22	190.36	65.39
20	11.36	7.50	1.91	44	0.87	0.66	0.08
21	8.46	7.39	1.42	45	5.12	4.26	0.93
22	9.70	8.43	1.60	46	2.11	1.60	0.26
23	0.10	0.09	0.02	47	109.30	86.08	21.28
24	0.24	0.23	0.03	48*	113.95	88.27	25.76

*48: the 1994 Northridge earthquake.

Table 5. Bridge repair process model parameters.

Damage state	Days for repair completion (days)	
	Minimum	Maximum
Minor	10	150
Moderate	20	200
Major	60	250
Collapse	75	300

The cost-effectiveness ratios in both retrofit cases are significantly increased when the social cost avoided is considered. The contribution of social cost avoided to the total benefit is far more than that of bridge restoration cost avoided. This indicates that most of the benefit due to retrofit comes from the social cost avoided.

Table 7 summarizes the cost-benefit analysis results based on Shinozuka's bridge repair process model (Table 5). In this table, cost-effectiveness is defined as "No" if "benefit/cost" ratio $r < 1.5$, "Moderate" if $1.5 \leq r < 2.5$, and "Yes" if $r > 2.5$. By adjusting the discount rate, we can see that the cost-effectiveness ratio decreases as the discount rate increases, as expected, and the cost-effectiveness ratio is dominantly controlled by the selected discount rate. Higher cost-effectiveness ratios are also observed when lower link residual capacity ratios are assigned to the damaged links of the freeway network. In fact, the cost-effectiveness is also very sensitive to the magnitude of the link residual capacity ratio. Since the link residual capacity relates to the traffic flow effectiveness

Table 6. Cost-benefit analysis (moderate link residual capacity; discount rate = 3%)

Benefit-Cost	Case 2: 23% Retrofit	Case 3:100% Retrofit
Total Social Cost Avoided ($Million) (1)	697	3121
Total Restoration Cost Avoided ($Million) (2)	24.2	86.7
Total Retrofit Cost ($Million) (3)	393	1665
Cost-effectiveness in terms of Restoration Cost Avoided (4) = (2)/(3)	0.062	0.052
Cost-effectiveness in terms of Social Cost Avoided (5) = (1)/(3)	1.78	1.88
Total Cost-effectiveness Ratio (6) = (4) + (5)	1.84	1.93

Table 7. Cost-benefit analysis summary.

Discount rate	Link residual capacity	23% Retrofit		100% Retrofit	
		r	Cost-effectiveness	r	Cost-effectiveness
3%	High	0.697	No	0.726	No
	Moderate	1.84	Moderate	1.93	Moderate
	Low	5.81	Yes	4.39	Yes
5%	High	0.495	No	0.515	No
	Moderate	1.30	No	1.37	No
	Low	4.12	Yes	3.12	Yes
7%	High	0.374	No	0.389	No
	Moderate	0.986	No	1.04	No
	Low	3.11	Yes	2.36	Moderate

through local detour routes, more accurate value of this ratio should be found by incorporating the local highway network into the freeway network analysis in future research.

The cost-effectiveness ratio is also different when different bridge repair process model is used. Though it seems that the cost-effectiveness ratio based on HAZUS model is approximately equal to that based on Shinozuka's model with link residual capacity under Assumption 2 (moderate link residual capacity), further study is required to examine both process models for possible integration taking advantage of their complementary temporal characteristics, for example, completely probabilistic (Shinozuka's model) versus totally deterministic (HAZUS model) (Shinozuka et al. 2005).

In conclusion, the research results indicate:

(1) The bridge seismic performance is significantly improved after column retrofit by steel jacketing, and the retrofit is more effective in reduction of more severe damages (major or collapse) than lighter damages (minor or moderate).
(2) The number of damaged bridges in the network is greatly reduced under earthquake attack. The accompanying benefit is the reduced bridge repair cost required for the damaged bridges. As more bridges are retrofitted beforehand, this reduction is more obvious.
(3) The system social cost consisting of drivers' delay and opportunity cost decreases significantly after bridge retrofit under seismic condition.
(4) If only reduction in bridge restoration cost is considered, either retrofit condition (23% retrofit or 100% retrofit) proves to be not cost-effective. The dominant part of the benefit is provided by

the social (drivers' delay and opportunity) cost avoided due to the enhanced network resilience resulting from the bridge retrofit.

(5) The retrofit is more cost-effective if the social cost avoided is included in the benefit from seismic retrofit when the network link residual capacities are lower and discount rate are smaller.

(6) Future study should be performed in the improvement of the modeling of link residual traffic capacity, bridge functionality restoration, OD recovery and bridge remaining service time, and in statistical data collection for more accurate bridge repair and retrofit cost estimation.

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The important roles of bridge maintenance and management on transportation safety and efficiency

M. Myint Lwin
FHWA, Washington, D.C., USA

ABSTRACT: Highway bridges form major links in the global transportation networks. The roles of bridge maintenance and management are vital to meet these expectations.

Transportation agencies are using systematic preventive maintenance, bridge management systems and asset management principles to balance demands and resources.

There are about 595,000 highway bridges in the U.S. State and local governments own most of the highway bridges. Concrete, steel, and prestressed concrete are the predominant materials used in construction. The average age of these bridges are 40 years. About 28% of these bridges are considered deficient. Effective use of bridge management systems and timely maintenance are required to control deterioration and extend the service life of aging bridges. A legislation was enacted in 1971 and established a national bridge inspection standards for bridge inspection and safety evaluation of existing bridges. Federal-aid funds are provided for the replacement and rehabilitation of deficient bridges. Systematic preventive maintenance is eligible for Federal-aid funds.

A good inspection program of existing bridges is essential for collecting timely and quality data regarding the conditions of bridges. Quality and consistent data are important for use in bridge management systems and prioritizing preventive maintenance activities. A legislation was enacted in 1971 and established a national bridge inspection standards (NBIS) for bridge inspection and safety evaluation of existing bridges. Federal-aid funds are provided for the replacement and rehabilitation of deficient bridges. Systematic preventive maintenance is eligible for Federal-aid funds. The bridge owners (States, counties, municipalities, etc.) are responsible for NBIS inspections with oversight by the State Department of Transportation (DOT). Information is collected on the bridge composition and conditions, and reported to FHWA where the data is maintained in the National Bridge Inventory (NBI) database.

The Surface Transportation and Uniform Relocation Assistance Act of 1987 expanded the scope of bridge inspection programs to include special inspection procedures for fracture critical

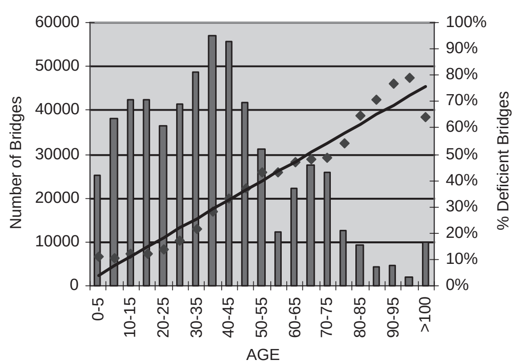

Figure 1. Ages vs. Deficiency.

members and underwater inspection. Additionally, FHWA provided guidance for scour assessment and identification of scour critical bridges.

The Surface Transportation Assistant Act of 1978 replaced the Special Bridge Replacement Program (SBRP) with the Highway Bridge Replacement and Rehabilitation Program (HBRRP) extending funding to include rehabilitation to restore the structural integrity of a bridge on any public road, and rehabilitation work necessary to correct major safety defects.

The Surface Transportation Assistance Act of 1982, the Surface Transportation and Uniform Relocation Assistance Act of 1987, and the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA) continued the HBRRP. Additionally, ISTEA allowed Federal participation in bridge painting, seismic retrofitting, and calcium magnesium acetate applications.

The Transportation Equity Act for the 21st Century (TEA-21) continued HBRRP. It authorized the set-aside of $100 million for each FYs 1999–2003 for discretionary allocation by the Secretary for major bridges with the provision that not to exceed $25 million would be made available for seismic retrofit of bridges, including projects in the New Madrid fault region. It also authorized set-aside of $25 million for FY 1998 for seismic retrofit of the Golden Gate Bridge. TEA-21 changed the HBRRP eligible work activities to include: sodium acetate/formate or other environmentally acceptable, minimally corrosive anti-icing and de-icing compositions or installing scour countermeasures.

The Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users of 2005 continued the HBRRP for replacement or rehabilitation of structurally deficient and functionally obsolete highway bridges in the States. Under this legislation the following work activities are eligible for HBRRP funding:

- Painting.
- Seismic retrofit.
- Systematic preventive maintenance.
- Installation of scour countermeasures.
- Application of calcium magnesium acetate, sodium acetate/formate, or other environmentally acceptable, minimally corrosive anti-icing and de-icing compositions.

Furthermore, this legislation has a special rule for preventive maintenance, allowing a State to perform seismic retrofit, systematic preventive maintenance or installation of scour countermeasures for a highway bridge without regard to whether the bridge is eligible for replacement or rehabilitation.

Effective maintenance starts with good design and detailing to assure durability, inspectability, maintainability and replaceability. The bridge designers must keep maintenance in mind when they prepare the bridge drawings for construction. Certain bridge elements and components, such as steel elements, bearings, connections, expansion joints, movable parts, require periodic inspection, maintenance and eventual replacement. The designers must provide access, platforms, lighting, ventilation, and attachment devices to simplify the work and life of maintenance personnel so they can perform their duties as efficiently as can be.

Effective maintenance starts with quality construction. Construction defects left uncorrected will soon become a maintenance nightmare. Lack of quality invariably results in early maintenance requirements and frequent repairs. The maintenance personnel are encouraged to join the owner's construction personnel in performing the final acceptance inspection.

Systematic preventive maintenance and preservation activities are necessary for assuring proper performance of the transportation infrastructure. Experience has shown that preventive maintenance is a cost-effective way for extending the service life of highway bridges and structures.

A State may carry out preventive maintenance for a highway bridge without regard to sufficiency rating or deficiency status. Systematic preventive maintenance implies the use of an effective maintenance strategy or a prioritization and optimization system to gain the most benefit from the investment on preventive maintenance activities. A bridge management system can help bridge owners to make sound technical and financial decisions on maintaining the structural health and serviceability of a bridge or a network of bridges.

Management and analytical tools are needed to collect and analyze the bridge data for predicting the present and future bridge preservation activities and related costs. In 1986, FHWA initiated a demonstration project to conduct workshops across the U.S. to sought development of sound bridge management practices. This demonstration project resulted in a collaborative effort between FHWA and six State DOT's in developing a generic bridge management system, later named Pontis. Pontis was made available to the States in 1991. In 1995, Pontis was incorporated into the AASHTOWare product line. It has been undergoing continuous enhancement under the guidance of a Task Force chaired by a member of the AASHTO Highway Subcommittee on Bridges and Structures.

In the U.S., the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA) mandated that State Departments of Transportation (DOT) and Metropolitan Planning Organization (MPO) used bridge management systems that maximize resource allocation for maintenance planning. This legislation initially established a deadline in 1995 for States to implement a bridge management system. Later the National Highway System Designation Act of 1995 repealed the mandate. However, FHWA and AASHTO continue to encourage and support the use of effective bridge management systems.

The BRIDGEWare suite includes three software products that address distinct phases of the bridge life cycle: Pontis for inspection and management of existing bridges, Virtis for load rating of existing bridges, and Opis for design of new bridges.

Pontis is a bridge management system that assists transportation agencies in making sound, fact-based decisions about maintenance, rehabilitation, and replacement of structures. Pontis stores complete bridge inventory and inspection data, including detailed element conditions; formulates network-wide preservation and improvement policies for use in evaluating the needs of each bridge in a network; and makes project recommendations to derive maximum benefit from scarce funds. Additionally, Pontis reports network and project-level results and forecasts individual bridge life-cycle deterioration and costs. Thirty-nine States, five municipalities and five international organizations are licensed users of the Pontis Bridge Management System (Figure 2.)

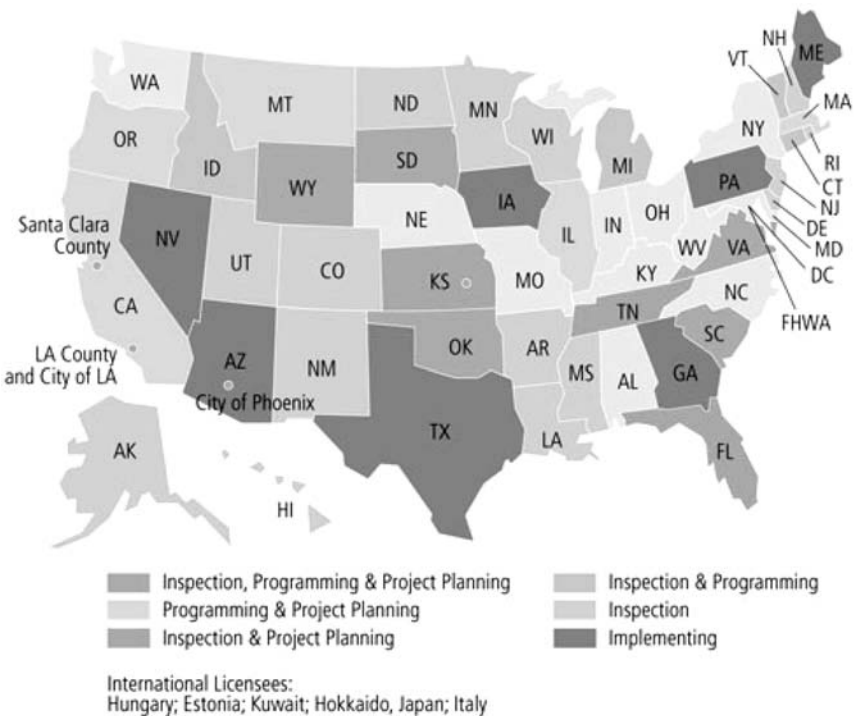

Figure 2. Pontis® licensees and applications in 2005.

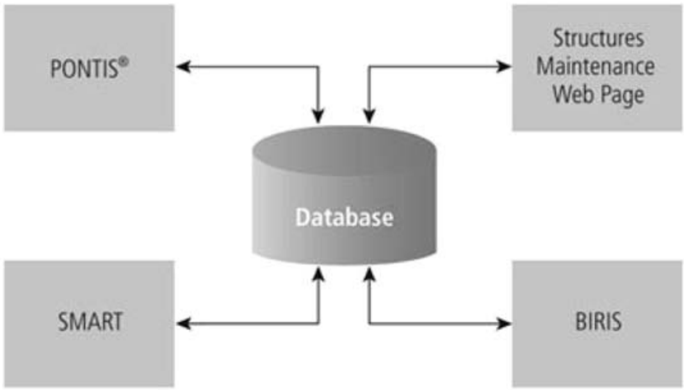

Figure 3. Caltrans Bridge Management System schematic.

The latest version of the Pontis software, Pontis 4.4, is available from AASHTO. Pontis 5.0 will include a web-based version of the Pontis inspection module. FHWA offers a training course on Pontis.

California is responsible for the inspection and preservation of approximately 24,500 bridges. The California Department of Transportation (Caltrans) Division of Maintenance Office in Sacramento, with a staff of more than 140 bridge inspectors, structural engineers, and bridge management engineers, performs periodic inspections and maintenance for all of California's 12 districts. Of the 11 bridge management engineers in the Division of Maintenance Office, 2 develop software, 2 run bridge management programs in Pontis®, 1 enters data, and 6 monitor all ongoing projects.

Database: All the information necessary to manage the integrity of California's bridge infrastructure is contained in a single database with sharing features using the Pontis data structure (Figure 3). Additional tables are linked to the Pontis structure for various mission critical activities, such as project tracking, maintenance recommendations, detailed fracture critical, scour and load rating information, and postearthquake inspection activities.

Bridge Inspections Using SMART: Caltrans inspectors are required to be State-licensed civil engineers. The inspection teams are responsible for all biennial inspections as well as fracture critical and underwater inspections. The teams collect inventory data and condition information in the field on system-generated pre-inspection reports and enter the inspection results into an electronic format to comply with National Bridge Inspection Standards (NBIS). The information includes detailed fracture critical findings, load rating information, photos, and commentary for each structure in the bridge inventory. All textual as well as graphic information from these statewide inspections is then entered into the centralized bridge management database.

The bridge information entered into the database through the inspection process is ultimately presented in a bridge inspection report. The inspection team is responsible for the inspection report and for making recommendations for preservation actions based on their findings in the field. The bridge inspection report documents the current condition of the bridge and all recommended work for that structure. This inspection report is the primary means of conveying the results of the inspection to the bridge owners.

In addition to the bridge report, the bridge management database is used to generate various lists and reports for district maintenance crews, project planners, Caltrans management, and the California Transportation Commission.

Project Prioritization Using Pontis: The bridge management staff uses the Pontis Bridge Management System to perform deterioration modeling and project prioritization. Within the Caltrans Division of Maintenance Office, the Office of Structure Maintenance and Investigation has the primary role for determining scope and priority for all bridge maintenance and preservation projects.

Bridge management engineers review and prioritize the needs identified by the inspectors. Projects are designed to minimize impacts to traffic and to maximize improvements with the funds available. Improvement projects involving, for example, new alignment or bridge widening, are initiated by the Caltrans Office of Planning along with Regional Transportation Planning Organizations.

Project Archiving Using BIRIS: California maintains a complete image archive of all bridge "as-built" plans, bridge reports, photos, and other significant correspondence in the bridge database. Caltrans has effective management of its entire bridge inventory, because the bridge management data are held in a single large database that is accessible to all users through Pontis programming tools.

The public has made significant investments in the construction, maintenance, and operation of the highway infrastructure, and expects transportation agencies will be responsible stewards of public funds. This information can help make cost-effective investment decisions.

Asset management is a strategic approach to allocating resources – dollars, people and data – for the preservation, operation, and management of the transportation infrastructure. The challenge to asset management is the ability to effectively integrate the following through a strategic framework:

- maintenance management system,
- pavement management system,
- bridge management system,
- tunnel management system,
- management of highway hardware, such as guardrail, signs, lighting, barrier, monitoring equipment and operating facilities,
- management of other highway assets, such as human resources, corporate data and information, real estate, equipment, etc.

The importance of systematic preventive maintenance and effective management of the transportation infrastructure is recognized worldwide. Technological advances and understanding in materials, design, construction, computer applications, bridge inspection, structural health monitoring, deterioration models, life-cycle costing, risk assessment, and economic, will enhance the management systems in making reliable comparison of alternative investments.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Application of the structural health monitoring system to the long span cable-supported bridges

Sung-Pil Chang
Seoul National University, Seoul Korea

ABSTRACT: In Korea, there has been a wide consensus about the necessity of bridge monitoring system primarily due to a chain of accidental bridge collapses since a decade ago. The 1st generation bridge monitoring system was originated from instrumentation system introduced upon the reconstruction of New Haengju Bridge after its sudden collapse in 1995. On-line instrumentation system installed in existing cable-supported bridges including Namhae Bridge, Jindo Bridge and Dolsan Bridge can be considered as the 2nd Generation Bridge Health Monitoring System (BHMS), which is followed by the development of 3rd generation bridge monitoring system represented by Seohae Bridge and Yeongjong Bridge in use to date. This paper tracks the background about the advent of Korean bridge monitoring system and its developmental history until now; brings up the current associated studies in progress and limitations of contemporary BHMS technologies as well; and thereby suggests possible remedial solutions for follow-up bridge monitoring system.

1 INTRODUCTION

The structural health evaluation technology that monitors the current status of bridges by analyzing the materials collected through automatic data acquisition is generally an extremely useful means of maintaining function, especially for cable-supported, long-span bridges that have extremely low accessibility. Still, the collected and analyzed information can be used as a determinate basis that can supplement and/or revise the existing bridge design standards. In addition, the structural health monitoring system can be used as a warning system that can detect the abnormal behavior not only of existing bridges but also long-span bridges that introduce new materials or construction methods for the first time and for which no design standards have been established yet.

Although pursuing industrialization relatively later than other industrialized countries of the world, Korea fostered the nation's key industries in full scale in the 1970s and emerged as the 13th largest trade country. Such fast economic growth necessitated the immediate construction and/or expansion of social infrastructure. As a result, many bridges have been designed and built without sufficient examination and within an extremely restricted time using limited budgets.

The ensuing negative results were shown during the series of bridge collapses in the early 1990s. Furthermore, the earthquake that occurred in the southern part of Hyokogen, Japan has led to the serious distrust of the public in the seismic performance of Korean bridges whose construction was rushed. The results of the analysis revealed several causes of continued bridge collapses, e.g., the government's administrative convenience-oriented construction policy and shortage of engineering technologies such as design technologies and non-destructive evaluation technologies of Korean bridge engineers. Following the collapse of the Sungsu Bridge, the Korean government conducted safety inspections covering most major bridges built until that time in order to evaluate the bearing capacity and repair or supplement them if necessary as well as consequently improve performance.

In addition, the government reinforced its legal system that required not only the inspection of the design and construction of all major bridges to be designed thereafter but also the installation of bridge health monitoring systems especially for cable-supported bridges in order to cope effectively with unexpected accidents that may occur during construction or use.

2 DEVELOPMENT HISTORY

2.1 *1st Generation of Bridge Health Monitoring System (BHMS): dawning period*

Modernization from 1970s and the advancement of bridge technology in 1980s had the bridge engineers imitate the design of foreign bridges. However, incomplete design and inspection technology resulted in the collapses of two prestressed concrete cable stayed bridges under construction.

In October 1994 the Gerber truss bridge, Sungsoo Bridge which was built in 1979, was collapsed 15 years after opening to public. The collapse of Sungsoo Bridge raised enormous impact to the Korea society, and caused nationwide safety awareness of civil structures.

In result, Korean government started to implement construction safety related provisions in both existing and newly-constructed structure, including the law for enforced safety inspection. In an effort to ensure the safety of civil structures, health monitoring utilizing the instrumentation was introduced to newly-constructed long span bridges. This can be considered as the 1st generation health monitoring systems in Korea.

The Shin-Haengju Bridge health monitoring system was the first attempt to utilize instrumentation to understand the behavior of a real bridge in service for better public safety and maintenance. It is now undergoing rebuilding process with new hardware and internet based monitoring software for better access and maintenance.

Even though the 1st generation bridge monitoring system did not performed as expected, it provided valuable lessons for the monitoring systems developed later.

1. The installation of sensors should be performed by experienced technician and well protected to ensure stable measurement. The sensors should be suitable to measure the range of response we are interested in, and noise protection must be considered.
2. The bridge monitoring system needs operating personnel and remote access capability when access to the monitoring computer is difficult. And stand alone type data logger is better suited for bridge monitoring system then lab testing data acquisition systems.
3. The measured raw data need to be in the form of database for easy manipulation and reference with software to utilize it.

2.2 *2nd Generation of BHMS: applications on existing bridges*

Namhae Bridge, completed in May 1973, is a three-span suspension bridge with main span length of 404 m and two side spans of 128 m each, as shown in Figure 4. A long-term monitoring system for Namhae Bridge was employed to monitor the structural response of the bridge to the view of identifying the deterioration rate over a long-term period.

An extensive and modernized version of the monitoring system was installed on the Namhae Bridge in Korea. Basic elements of the Namhae Bridge Monitoring System have been installed since December 1996 including event recording and most of the sensor channels. The system was finally completed in May of 1997. The Namhae monitoring system provides 74 static and 36 dynamic channels.

Another good example of health monitoring system installed in existing bridge would be the case of Jindo Bridge which was completed in May 1984. It is a three-span cable-stayed bridge with a main span of 344 m long and each side span of 70 m long. It has two steel pylons and stay cables are arranged in semi-harp type.

A safety evaluation project for this bridge was carried out in 1993 and it was found that camber at mid-span had become lower than the original design profile and one stay cable had lost tensioning force by small amount and some natural deterioration had been occurred. And also, it is turned out that the bridge deck is weak for torsional flutter by aero-dynamic study. Since 1996, the health monitoring system along with the total Bridge Management System has been adopted in this bridge. The conceptual diagram of monitoring system is shown in Figure 6. It is mainly composed of three parts: automated data acquisition system, automated data processing system and data management system. The characteristics of the second generation is that data acquisition and processing system are located in the station near by the bridge and management system is located in an office that is far from the site.

2.3 *3rd Generation of BHMS: applications on newly-built bridges*

The bridge monitoring system installed across domestic cable support bridges opened after Year 2000(including Seohae Bridge, Yeongjong Bridge and Gwangan Bridge) is built up more securely and stably based on technicians' experiences than previous ones, and it is also built up in order that bridge engineers can utilize stored data more effectively with more formulated measurement database.

Seohae Bridge, which is located on Seohaean Highway, is a grand bridge of total length 7,310 m, which connects Pyeongtaek city(Gyeeonggi province) and Dangjin-goon(Chungnam province) with each other. It started to be constructed in Nov. 1993 and became opened up in Nov. 2000. It also comprises the largest cable-stayed bridge in Korea (center span: 470 m, total length: 990 m, and main tower's height: 182 m). This cable-stayed bridge is equipped with instrumentation for measuring temperature, wind and seismic load acting around bridge, as well as other equipments to measure any load-induced response such as deformation of stiffening girder, cable tension, main tower's tilt, etc.

3 APPLICATIONS OF BHMS RESULTS IN FIELD

It is mandatory that newly constructed special bridges including Seohae Bride and Yeongjong Bridge are inspected with loading test before their opening. During this inspection, field engineers obtain static and dynamic initial value from bridge, while preparing a possible numerical analysis model for bridge maintenance along with calibration. In particular, Samcheonpo Bridge was once tested with field experiment to find out whether it is possible to detect any variation of structural system through neural network technique; it was difficult to get sufficient precision in the 2nd level damage detection to locate any damage, but we could get considerably meaningful results in the 1st level damage detection to find out any variation of structural system. This chapter introduced experimental studies conducted in actual bridges on site.

3.1 *Ambient Vibration Test of Namhae Bridge*

An Ambient Vibration Test (AVT) for the whole bridge was carried out in August 1999. Mode shapes as well as natural frequencies were found to give a more precise description of the current stage. A simple impact test was also carried out in order to compare the results.

The FE model was updated using the measured modal parameters. The comparison of measured and computed frequencies shows that FE program can simulate vertical behavior of the suspension bridge within 13% of error. But this program could not give satisfactory result for lateral modes due to the assumptions inherent in the program such as neglecting bending stiffness of hanger cables.

3.2 *Initial loading test and baseline model evaluation of Seohae Bridge*

To identify the initial behavior of the Seohae Bridge, static and dynamic loading tests were performed before opening to the public. The finite-element (FE) model was updated with the measured data, to increase the accuracy of structural analysis. The updated model built for the bridge maintenance will be used for further state evaluation of Seohae Bridge.

Three kinds of result were compared. The first is the analysis result from the initial model based on design parameters, and the second is the results of baseline model updated with measured data. The final is measured. The updated result was satisfactory and the average error of the first 5 natural frequencies was reduced to 0.6% while that of the initial model was 5.1%. In other hands, difference between static displacement of LC1 is regarded as negligible, considering measuring error.

Table 1. Analysis Result Comparison.

Analysis	ID	Initial model	Baseline model	Measured	Remarks
Dynamic	V1	0.244 Hz	0.263 Hz	0.262 Hz	
	V2	0.307 Hz	0.329 Hz	0.331 Hz	
	L1	0.345 Hz	0.362 Hz	0.362 Hz	
	V3	0.525 Hz	0.542 Hz	0.534 Hz	
	V5	0.597 Hz	0.625 Hz	0.629 Hz	
Static	LC1	131.9 mm	123.5 mm	128.1 mm	L/2 section of main span, downward displacement

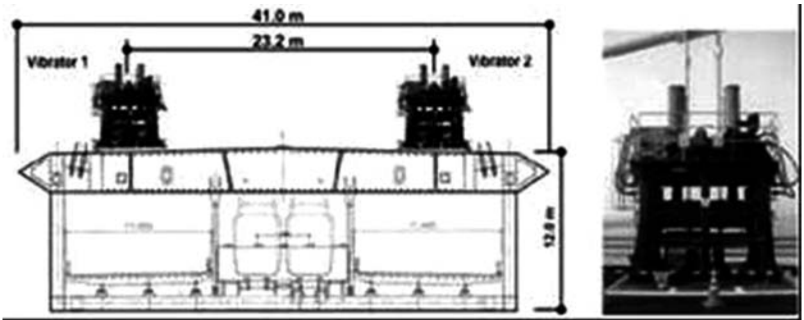

Figure 1. View of the vibrators installed on Youngjong bridge for field loading test.

3.3 *Forced vibration test of YeongJong Bridge*

The SHM system has been exploited at first to identify the dynamic properties of the bridge before opening by means of field loading test using two vibrators, which generated flexural and torsional vibrations of the bridge (Figure 1). Comparison of the measured data such as natural frequencies, vibration modes and damping ratios with design values showed good correspondence attesting for the reliability of the bridge (Koh et al., 2002, 2004).

3.4 *Structural identification of Samcheonpo Bridge using vibration signal*

Samcheonpo Bridge is a cable stayed bridge (103 m + 230 m + 103 m) with composite deck and 80 cables, connecting Sacheon city and Changsun Island in Korea Peninsula. The FVT and loading test are performed on Samcheonpo Bridge and the damage detection technique is applied to the bridge based on the experiment. In this study, an attempt at better damage detection results with only acceleration responses is made through the new ANN-training method on the basis of the basic concept of the pattern recognition. The proposed method is validated through the experiment on the model bridge and its applicability is also discussed by the field experiment conducted on Samcheonpo Bridge.

4 RESEARCHES UTILIZING MONITORING RESULTS

In early days, the bridge monitoring system (or BHMS) was designed to detect any possible or potential risky factor during the construction of bridge in advance and prevent any resulting sudden accident as well. It was also implemented in the form of construction monitoring system, and evolved into a long-term monitoring system that aims to maintain installed H/Ws for construction monitoring even after completion of bridge, and complement them with extra H/Ws as required to secure user's safety during the use of bridge.

The crucial onset of BHMS is originated from a series of accidental bridge collapses, so associated studies on BHMS mostly related to detection of any damage around bridge structure. There have been a variety of studies conducted so far so as to use static/dynamic measurement data to identify current conditions of structure and detect any potential variation (or damage) of structural system in earlier step, and many relevant studies are still conducted now. However, they face challenging obstacles – such as insufficient degree of freedom in measurement and measurement noise – against technical commercialization. The associated studies include inverse analysis with SI method; signal analysis and pattern recognition approach using AI technique like neural network; methodology using statistical time series analysis, and so forth. The current profile across each academic field and impending challenges are outlined

Investigation of long-term responses of cable supported bridges showed that the behavior of the bridge is mainly governed by daily and yearly variations of temperature. Following, researchers aimed the assessment of the actual state of the bridge through the analysis of structural responses regard to temperature.

Generally, there are two major different approaches in analyzing temperature effect. The first one filters out the temperature effect from measured data so that the engineer can analyze data without considering the influence of temperature. The other one attempts to figure out the change of a structural system by investigating relationships between temperature and responses.

In the first approach, temperature effect is removed by simply subtracting moving averages of proper period from the original response signals. This is equivalent to filtering out the responses which have 1-day and/or 1-year cycle in frequency domain by appropriate band-pass filters. This approach is mainly used for analyzing the effect of traffic load only.

The other approach focuses on exploiting temperature effects for structural assessment. For this purpose, functional relations between temperature and structural response have to be defined before analyzing currently measured signals. Structural change or damage is found to occur when measured signals do not agree with the so-predefined temperature-response relationships (Hwang, 2003; Sohn et al., 1999). Various kinds of system identification methods including neural networks, statistical method, and optimization method can be employed to construct a mathematical function. Among them, the ARX model, a statistical time series analysis method, is used for Yeongjong Bridge's data analysis.

5 FUTURE EXTENSION OF BHMS DATA UTILIZATION

5.1 *Utilization of BHMS data for design improvement*

It is highly anticipated that the design process of a bridge can be improved by using BHMS in standardization and realization of the design loads which are the most important factors in the design process. Principal loads for bridge design are truck load as well as wind and earthquake load. Regarding these loads, several items for measurements are specified in BHMS. Long term data which are measured from anemoscopes, anemometers and seismic accelerometers located at important positions of a bridge can be used efficiently to update the wind and the earthquake load for future design purpose.

Wind data are measured by anemoscopes and anemometers which are installed at several locations on the stiffening girders and the pylons. The wind directions and wind speeds measured at several locations can be utilized to define the probabilistic distribution of the mean and turbulent wind with respect to time and space. Wind speed distribution, fluctuation and duration for several locations of a bridge during severe wind can be utilized as valuable data for the estimation of the basic and/or design wind speed for large-scale bridges such as cable-supported bridges. In addition, these data can be applied to determine the wind environment model such as turbulence intensity, turbulence spectrum and the cross-correlation of turbulence between separated points so as to provide more reliable prediction of buffeting effect on aerodynamic design of long-span bridges.

The structural response data, measured at the same time by displacement measuring equipments and accelerometers, can be utilized to verify the buffeting analysis tools. In this process, aerodynamic admittance and other aerodynamic and/or aero-elastic parameters can be calibrated based on the possible differences between measured and predicted responses of the bridge.

Also, BHMS Data can be utilized for realization of traffic load. In Seohae Bridge, the structural behavior of the cable-stayed bridge has been observed during 2 years after its completion. Results showed that the annual variation of the vertical deflection in the stiffening girder satisfies the allowable design limit with a range of −320 to 30 mm and that deflection due to live load presents a variation range of 189.7 mm, which represents only 25% of 808.8 mm, the design limit. The stress range in the stiffening girder due to live loads showed good correlation with the volume of traffic monitored during 2 years and, since it represents only 5 to 12% of the design stress, stress margin appears to remain considerable. It could thus be said that actual highway bridge design specifications are producing excessively conservative structures.

5.2 *Improvement of bridge management process*

Today, the maintenance of bridge structures relies much on visual inspection, and the analysis of measured data obtained through monitoring system like BHMS functions as a complementary role in visual inspection.

However, in view of findings from recent studies, it is estimated that technical commercialization is feasible within relatively short duration till the 1st level damage detection to determine any damage of structural system through analysis of monitoring data. Although it is still difficult to get sufficient precision in 2nd level damage detection for identifying damaged locations, it is reasonable that the region of bridge under inspection could be reduced as much as 1/2 or 1/3 to help field engineers perform correct check and identify current status within shorter duration as possible, if there is any estimated occurrence of damage around structural system, to the extent that the position and extent of damages should be identified with precise safety diagnosis based on visual inspection.

In addition, long-span bridges including cable-supported bridges often include zones that are difficult or impossible for field inspection staffs to access. To overcome this obstacle, mobile inspection vehicles or the like are set up now on site around bridge In the future, it is advisable and necessary to develop inspection robots to overcome the limitations of accessibility, and the requirements for such inspection robot include 1) mobility, 2) vision, 3) non-destructive testing, 4) remote control and data (signal) transmission, and so on.

6 CONCLUSIONS

This paper went back to the background about advent of BHMS in Korea and its developmental history as well, while covering current associated studies in progress and the limitations of current technologies. Based on these findings, this study comes to the following conclusions:

1. In order to complement inevitable shortage in bridge design and construction technology which fall as long as about 150 years behind those in foreign advanced countries, Korean bridge engineers have developed BHM System with their own technologies and applied those technologies to domestic construction fields successfully.
2. Today, in Korea, there are studies focusing positively on identifying the possible behaviors of bridge under construction in terms of localized BHM System. Furthermore, theoretical and experimental studies are conducted briskly to apply BHMS to bridge maintenance system in regard of whole life cycle.

   However, it is too early to apply each of their findings to detection of possible damages occurring on site in practical manner. Fortunately, the latest technologies developed through these studies have reached the level of possibility to detect any damage around or in bridge structures across broad scope. Thus, it is possibly feasible to apply this damage detection method in parallel with existing visual inspection for the sake of much more efficient bridge maintenance compared to current bridge management system.

3. Although current BHM System under development and operation cannot accurately detect the extent and location of damages, it is possible to further precisely identify any behavior around or in actual bridges by analyzing long-term data obtained from bridge structure. That is why current criteria of bridge design in use can be complemented more to meet actual requirements. Especially, in order to develop bridge design concepts into performance criteria or risk-based design concept, BHM System can be a very useful means for them. In addition, it will be possible to define the destruction of bridge in more rational manner.
4. The most critical purpose of applying BHM System is possibly to detect any abnormal condition of bridge and pinpoint damaged positions. However, it is unreasonable to pinpoint damaged positions only with SI method or monitoring system itself as developed so far. Since it is also challenging and unfeasible for persons to inspect whole intervals of difficult-to-access cable-stayed bridge at first hand, it is required to develop inspection robots instead of human resources.
5. Finally, BHM System comprises very sensitive hardware and software, and also requires maintenance staffs qualified for analyzing and synthesizing collected data from measurement. So the success of actual maintenance depends on the availability of appropriate budget required for the operation of BHM System and the consistency of principal maintenance staffs.

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Innovative structural health monitoring of bridges in Portugal

J.A. Figueiras
LABEST, Faculty of Engineering, University of Porto, Portugal

C.M. Félix
LABEST, School of Engineering, Polytechnic Institute of Porto, Portugal

ABSTRACT: The structural monitoring, using new sensing/processing and modelling/identification techniques, is a field with enormous potential to be applied in civil infrastructures. The access to continuous and structured information on the behaviour of structural systems permits new insights and deeper knowledge, pointing out new ways of assessing the design, construction and maintenance of civil engineering structures. In Portugal, the LABEST as other research groups is stepping up the application of the novel monitoring concepts to bridges and other civil structures. The present paper describes some aspects of the work developed in the field of structural monitoring of bridges. The benefits of structural monitoring are here appraised through the presentation of short term monitoring examples. The pioneer work of implementing a permanent optic based monitoring system in the centenary Luiz I Bridge is also described.

1 INTRODUCTION

Civil engineering structures are complex interconnected systems made up of interacting engineered natural and human systems. Our knowledge and insight on the actual day-to-day performance of infrastructures under environmental actions and applied loads is often very scarce. Observations, measurements and experiments involving actual infrastructures and constructed systems, in the field, constitute essential steps for acquiring this knowledge. The design of more advanced structures, the introduction of new materials and the use of novel construction processes are demanding increased responsibilities and risks to the designers, constructors and owners. Engineers have long sought for means to obtain information on how structures behave during construction and in service, so that they can feel more confident in building less conservative structures and accepting a controlled risk of deterioration or failure (Mufti & Ansari 2004).

Civil infrastructures systems, the most expensive assets in any country, are deteriorating at an alarming rate due to inadequate maintenance, excessive loading and adverse environmental conditions. Feedback on their state of health is practically inexistent. Civil engineering professionals have maintained the integrity of bridges, tunnels and other structures by means of manual systems of inspection, nondestructive evaluation and interpretation of data using conventional technologies (Shehata et al. 2004, Bergmeister 2004). The profession has relied heavily on evaluation parameters given in codes of practice that are conservative and often costly.

In the last years, an huge effort has been made in our country to the renewal of the road and rail networks but, by and large, today's bridges and tunnels continue *blind, deaf and dumb*, not taking advantage of the benefits of advancing information technologies. Although, the management decisions are based on the current best practice, the inaccuracies of the current rating methods result in the retrofitting or replacement of many bridges that, in some cases, need not be retrofitted or replaced. Worse is the possibility that some bridges needing engineering renewal or replacement are not identified.

LABEST – Laboratory of Concrete Technology and Structural Behaviour is a research unit integrated in the Portuguese Scientific Network and based on the Faculty of Engineering of University of Porto that has been devoted in the last years to the problematic of the SHM of bridges. The work has began with field load tests and short term monitoring of structures wherein electrically based sensors and automatic acquisition systems were used (Costa et al. 2004). Then, public funding research projects have been carried out to develop laboratory demonstrators and new competences in the sensory and signal interrogation systems. Electric and optic (Bragg grating) based sensor holders have been developed and appraised in view of a more reliable and durable monitoring of civil engineering structures. Demonstration projects have been undertaken, with public and private funds, to promote the application of the new concepts of the SHM. The research project SMARTE, supported by AdI-Inovation Agency, joined Portuguese research laboratories (LABEST, INESC–Porto) and a highway infrastructure owner (BRISA) with the objective of implementing an efficient health monitoring system that could help to face the bridge maintenance issue. Agreements with infrastructure owners and constructors to supply them specific monitoring systems have been paving our way towards a new engineering approach to design, construct and maintain civil structures making use of the SHM potentialities (Alves et al. 2005). The monitoring work developed during the renewal and strengthening operations carried out on Luiz I bridge to be integrated in the *Metro do Porto* network is an example of a pioneer intervention of LABEST. The present paper describes some aspects of the work developed in the field of structural monitoring of bridges.

2 BENEFITS OF STRUCTURAL MONITORING

Monitoring of bridges can provide many benefits to designers, constructors and operators helping to build and maintain more durable and safer structures. The main objective of Structural Health Monitoring (SHM) is to identify any deficiency that a structure might develop in the future. However, before future deficiencies are identified, a monitoring system must be able to identify the current behaviour of the structure. The difficulties in interpretation of data to determine the current behaviour of monitored structures is actually a major knowledge gap to be bridged for the general acceptance of SHM by bridge owners (Bakht 2004).

LABEST research unit has been developing and applying this new area of knowledge to different structural systems, answering to construction companies and owners needs and contributing to cheaper and safer structures that can be in operation for long periods of time with minimum maintenance. The development and application of numerical models has been an important issue in assisting to the application of monitoring systems. They can be used to assess accuracy and to improve confidence on the monitoring results achieved and they can help in defining adequate algorithms for structural damage detection.

The benefits of structural monitoring are here appraised through the presentation of short term monitoring examples. The cases briefly described below are selected from a list of personal experiences of the authors. The assessment of the structural behaviour and monitoring data is generally accomplished by comparing the measured data with the corresponding quantification given by an adequate numerical modelling.

2.1 *Controlling the construction phase*

By demanding of APOR – Agência para a Modernização do Porto (Agency for the Modernization of Porto), and the designer (Grid – Consultas, Estudos e Projectos de Engenharia) a prestressed concrete cable stayed viaduct, the east entrance of the city of Porto, was monitored during the construction phase. Strains, temperatures, rotations and deflections have been monitored on the deck and on the mast during construction, subsequent loading tests and current service-life by using a system, based on electrical sensors providing the automatic and simultaneous interrogation of all sensors, with a minimum of human intervention (Felix 2005).

The viaduct in an asymmetrical structure crossing railways, roads and other urban equipments, along the total length of 316.5 m (see Fig. 1) with two main spans (120 m and 69 m) and two lateral spans on each side. The bridge deck is a prestressed concrete mono-celular caisson 18 m wide and

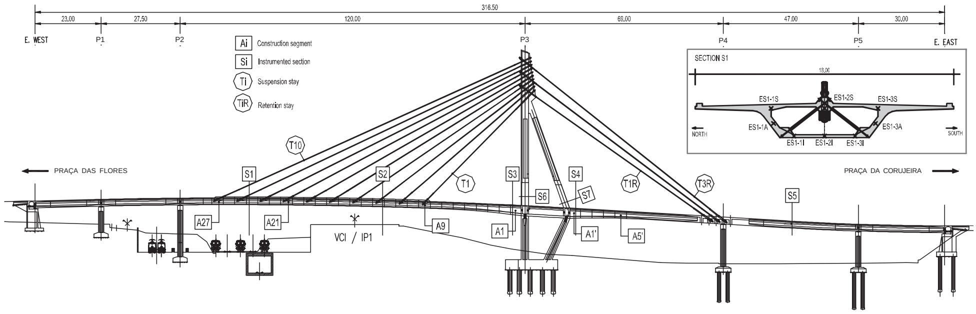


Figure 1. General geometry of the cable stayed viaduct. Location of the instrumented sections (S1 to S7).

a constant height of 2.4 m, except near the mast (pier P3) where it has a parabolic shape with a maximum height of 3.6 m.

The stay cables are distributed in three planes: one plane of ten stay cables is suspending the major span of 120 m, which are fixed at the middle of the deck, and the other two planes, with three retention stays each, are fixed at lateral zones of the girder over the pier P4. The mast is composed by two partial tubular legs with variable sections. This shape allows to increase the longitudinal stiffness and to balance the asymmetry of the two main spans without making use of more retention cables.

During construction the monitoring plan included the instrumentation of five sections at the deck and two sections of the mast (Fig. 1). For the observation of these sections, thirty eight electric strain gages (ES) and eleven temperature sensors (TS) were embedded into concrete. There were also applied four pairs of strain gages at the steel diagonals inside the deck, three ambient temperature sensors and four strain gages embedded into prismatic specimens to monitor in-situ concrete shrinkage (2 inside and 2 outside the box girder).

At the suspension sections of the main span (see Fig. 1), the transmission of forces from the stays to the box girder is realized by pairs of diagonals that consist of hollow rectangular shaped steel sections of 250 mm × 150 mm and 8 mm thickness, filled with concrete and prestressed by two high strength bars of 40 mm of diameter. In order to closely follow the process of force transfer to the girder, the first pair of steel diagonals installed at the insertion of stay T1 (segment A9) was instrumented with strain gages glued to the diagonal section, as shown in Figure 2. To keep the sections of diagonals under compression, the following design procedure was established for the tension operations: (i) application of a prestress of 450 kN per diagonal ($BT01_1$); (ii) tensioning of stay cable T1 up to 50% of maximum force ($T1_1$); (iii) additional prestressing of 450 kN applied to each diagonal, being then the bars sheathings injected and the anchorages sealed ($BT01_2$); and (iv) additional tensioning of the stay cable up to the design force value ($T1_2$/$T1_3$).

Figure 3 presents the monitoring results obtained with the strain gages applied to the diagonals during the first week of operations, with a five minute interval between consecutive readings. It is possible to clearly identify and quantify the forces induced in the steel diagonals by the two phases of bars prestressing ($BT01_1$, $BT01_2$) and the effect of the first three phases of tensioning of the stay T1 ($T1_1$, $T1_2$ and $T1_3$). These monitoring results have been used to appraise the detailed finite element model of the suspension zone elaborated by the designer (Santos 2004). An incident occurred during the application of the additional prestressing ($BT01_2$) to one of the diagonals (anchorage slip?) which resulted in a significative reduction of the compression strain in this element (see Fig. 3). As a consequence this diagonal (the right one) will be in tension (+150 μstrains) during the service phase, whereas the other (on the left) will work slightly in compression (Felix 2005). The variation of the diagonals strains during the last three days (Fig. 3) is only due to thermal effects.

The importance of the present monitoring results may be highlighted for three order of reasons, namely: (i) calibration of the suspension zone design; (ii) appraisal of the construction procedures, and (iii) the real-time information of any fail permits a deeper understanding and the quantification of its effects, enabling the designer to take the adequate options during the construction process.

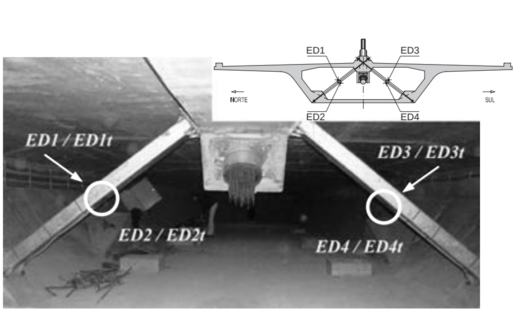


Figure 2. Four pairs of strain gages applied to the steel diagonals of A9-segment.

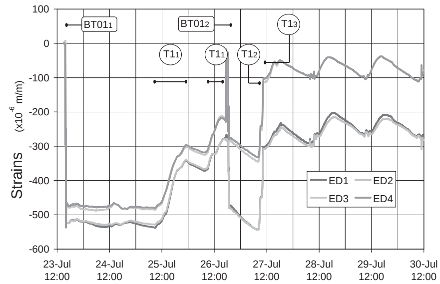


Figure 3. Strains measured at the diagonals of A9 segment during the tension operations (first seven days).

Table 1. Vertical deflections and rotations for load case 1L (see Fig. 4).

Vertical deflection at main span (mm)				
	DV1	DV2	DV3	DV4
Model	25.0	36.5	37.1	28.2
Measured	23.6	34.2	35.0	26.2

Longitudinal rotation at the mast legs ($\times 10^{-3}$ degrees)		
	IE1x-x	IE2x-x
Model	−8.6	−3.9
Measured	−7.2	−3.8

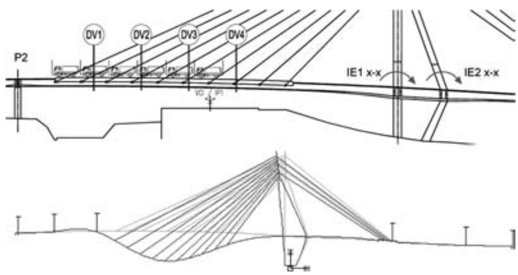


Figure 4. Load case 1L with six pairs of 20 tons trucks on main span and the corresponding deformed geometry given by the numerical model.

To finish the presentation of the monitoring results of this urban cable stayed viaduct during the construction phase, some results of the proof tests carried out at the final of construction are illustrated in Table 1 for the load case 1L, corresponding to six pairs of trucks, 20 tons each placed at the most critical position on the main span (see Fig. 4). The vertical deflections measured at four sections of the main span (DV1 to DV4) and the rotations observed at the insertion of the mast legs with the deck (IE1-xx, IE2-xx) are specified in Table 1 and compared with the corresponding values given by the finite element model used for validation. The initial and the deformed geometry (amplifying $\times 500$) is illustrated in Figure 4 for the load case 1L. The application of an adequate numerical model to validate the measured results and detect possible damage has been mandatory.

3 MONITORING OF LUIZ I METALLIC BRIDGE

3.1 *Introduction*

The Luiz I bridge is a centenary steel arch bridge, crossing the Douro river in the city of Porto, in northern Portugal (Fig. 5). It was designed by Teophile Seyrig, a Belgian engineer disciple of Gustave Eiffel, and has been in continuous operation since October 1886 (Seyrig 1884, Azeredo & Azeredo 2002, Cruz & Cordeiro 2001).

The bridge is constituted by a metallic double hinged arch, supporting simultaneously two decks at different levels over the river crossing. The arch has a parabolic geometry both in vertical and plan views, presenting 172 m long and 45.1 m of maximum height, with a variable apparent vertical thickness from 16.7 m near the supports to 7.1 m at the crown. Two continuous truss girders, attached to the crown arch and sustained by roller supports over seven piers and the abutments, materialize the different 13 spans of the upper deck, performing a total length of 391.25 m.

Figure 5. Overall view of Luiz I bridge.

Despite some operations of rehabilitation and maintenance or minor changes suffered to accommodate the passage of new types of vehicles, the structure has been in continuous operation since its conclusion date in October 1886. However, recently a strengthening and rehabilitation process took place on this bridge, in order to allow the integration of its upper deck in the infrastructure of the Porto Metro Network. One of the major operations fulfilled during this process was the replacement of the upper deck bridge concrete pavement, resting on steel beams, by a suitable metallic profile grid, able to transmit properly the new railway traffic loads to the truss girders.

The gentleness of some of these operations, the appraisal of the strengthening solutions performance and the significance of this historical bridge, made the structural monitoring of its behaviour mandatory. Before the strengthening operations took place, an experimental assessment of the bridge was performed. The objective of the monitoring campaign was the appraisal of the bridge structural integrity and the ascertainment of the structural model elaborated to support the strengthening design. An electrically-based monitoring system was applied to critical sections of the arch and the upper deck girder to measure strains and displacements during a load test and during a short monitoring period under service conditions (Figueiras et al. 2005). An advanced monitoring system based on the fiber optic Bragg grating sensors was applied to the Luiz I bridge to permanently assess the service performance and the structural safety. This system was designed to be installed during the construction process, taking advantage of the general scaffolding mounted for the bridge rehabilitation. A brief description of this novel system is presented in the next sub-section (Costa et al. 2006a).

3.2 *Instrumentation and processing system*

The implemented instrumentation system for the observation of the bridge performance during its new duty, allows measuring the following three items: (i) strains in selected truss elements of the arch, upper deck girders, metallic piers and suspension ties; (ii) relative horizontal displacements of the expansion joints at the abutments and between the bridge upper deck and the masonry piers; (iii) temperature experienced by the steel and the environment, both in arch and in upper deck elements.

In order to assess the relevant strains, 118 fiber optic sensor holders were installed, distributed by pairs in the selected 59 critical sections of the elements to be observed. Figure 6 illustrates the general location of the strain sensors applied to the bridge, indicating the number of strain sensors in each position. In this figure it is specified the notation used for two sensors applied to the bottom cord of the downstream girder beam (ET-I7/I8), and for four sensors applied to the top and bottom cords of the upstream arch crown (EA-S13/S14; EA-I15/I16).

To observe the relative movement between the bridge upper deck and the abutments and pier P5, eight displacement transducers based on fiber optic Bragg gratings were applied. The long term observation of the supports performance is mandatory to identify any bearing disfunction.

The sensor holders used to appraise the field strain are surface mountable sensors. These sensors are single fiber optic Bragg gratings (FBGs), with the grating embedded in carbon fiber reinforced polymer (CFRP). The procedure used to install the sensor holders is concisely described in

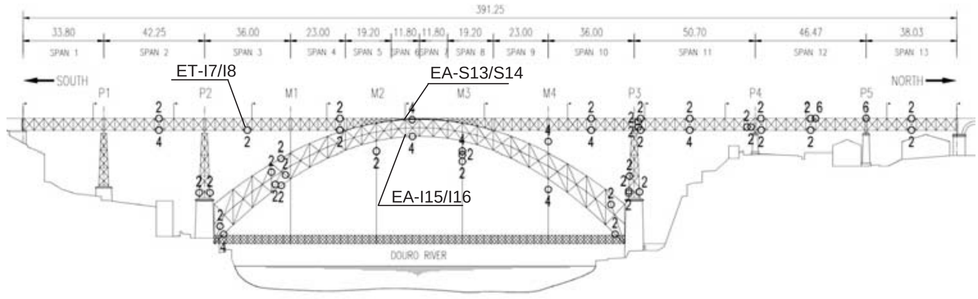
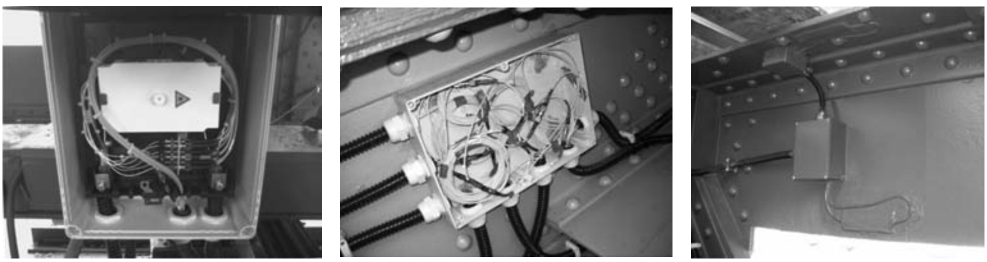


Figure 6. Location of strain sensor holders applied in the monitoring system of Luiz I bridge.



(a) (b) (c)

Figure 7. Fiber optic network installation: (a) fiber optic splice enclosure; (b) signal-concentrating box located at pier M1; (c) sensors and junction box at a section of the upper deck top cord.

Costa et al. 2006b. A protective cork layer covered by a polyester sheet impregnated with epoxy resin is applied to protect the sensor. When the resin cure is completed, epoxy paint used on the weathering coat of the steel is applied to the installation surface (see Fig. 7c). This kind of cover provides mechanical protection, avoids direct insolation and enables the resistance against environmental factors, such as UV radiations, rain and moisture.

The network was designed taking into account technical and economical conditionings, and the particularities of the bridge, selecting a tree configuration with a main optical cable with derivations to each branch containing up-to 10 sensors connected in series. The fiber optic channels are linked to the main optic cable through fiber optic splice enclosures (see Fig. 7 a). For each pair of sensors, there are two patch cords connecting them to one of the signal concentrating boxes (see Fig. 7 b) located at bridge technical path. The Bragg grating strain and temperature sensors, and the optic cables and connections were supplied by Fiber Sensing.

The data acquisition of the 136 optic sensors installed (118 strain sensors, 10 temperature sensors and 8 displacement transducers) consist of an interrogation system, Si425-200 swept laser interrogator, provided by Micron Optics, Inc., a 1 × 16 fiber optic twoplane switch for single mode fibers (Metrotek), a desktop computer and a modem. The optical sensing interrogator uses an Ethernet port to network to the desktop computer for data analysis using on-board controls and LabView™ remote utility. The raw data, in wavelength format, is sent by modem through a telephone to a server, integrating the Data Storage/Processing System, located at the Faculty of Engineering of Porto University. The data organized by graphical outputs and standard periodic reports are provided to the client through a website portal specially developed for this monitoring work.

3.3 *Load test results*

The bridge structural behaviour has been assessed before the renewal operations (Figueiras et al. 2005) and after the rehabilitation and strengthening process (Costa et al. 2006c) by performing adequate loading tests. In both cases a loading sequence with 10 similar trucks weighting around 13 tons each has been considered to perform 10 different load cases (1 to 10) which maximize

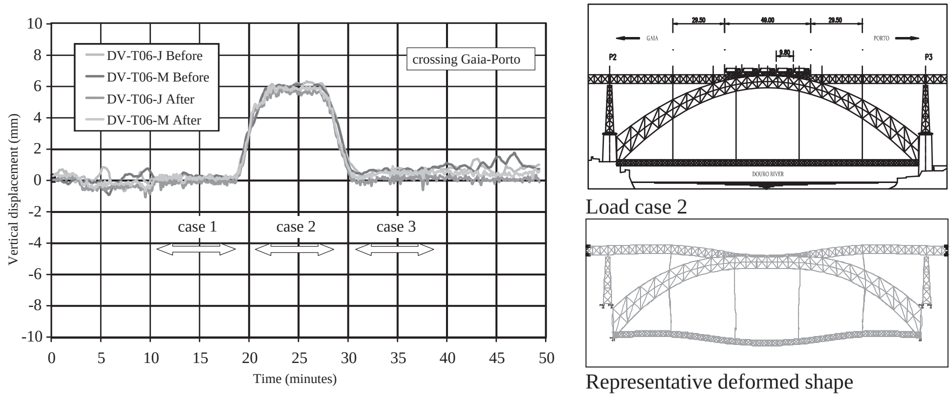


Figure 8. Comparison of the upstream and downstream vertical displacements of the arch crown, before and after rehabilitation for load case 2.

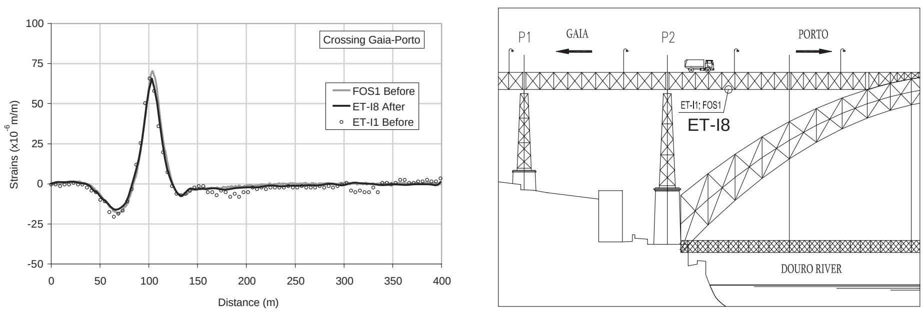


Figure 9. Comparison of the bottom cord strain of the downstream girder beam, span 3, before and after rehabilitation for the crossing of a pair of trucks.

the monitored parameters and an additional load case corresponding to the passage of a pair of trucks, crossing the bridge, moving slowly (v = 0.45 m/s) along the upper deck.

The vertical displacement measured at both sides of the arch crown for the load cases 1 to 3, before and after the rehabilitation, are illustrated in Figure 8. The trucks position for load case 2 and the corresponding schematic deformation exhibited by the bridge model are also shown in the same figure. The average deflection exhibited by the arch crown for load case 2 was 5.5 mm, whereas the corresponding displacement given by the model is 6.0 mm.

A comparison between the strains obtained in the bottom cord of downstream girder beam, span 3 (see Fig. 6) is illustrated in Figure 9. The comparison is made between the results of a fiber optic Bragg grating sensor (FOS1) and an electric strain gage (ET-I1) at the same location, obtained before the bridge rehabilitation, and the new fiber optic sensor (ET-I8) installed at the same local after the bridge rehabilitation. The graphical data refer to a slow-moving crossing Gaia-Porto of a pair of trucks. It can be observed a very good correlation between the obtained results.

4 CONCLUDING REMARKS

Structural health monitoring is a rapidly growing field of interest in the civil engineering industry. However, only through successful SHM projects will the public be convinced that the monitoring of structures is a beneficial process. The end product of any monitoring system for a bridge or other structure should be tangible information that could be used by bridge designers, constructors and owners to make decisions about the behaviour and the management of the structure. More than the installation of a nice monitoring system is necessary a thoughtful interpretation of the structural behaviour to integrate in the bridge the right sensors at right locations and the adequate signal interrogation and processing system with minimum costs. An expert interpretation of the collected data to deeply understand how the structure behaves is mandatory for the success of any SHM undertaking.

Between the several aspects wherein the SHM research work are being developed there are two topic that the LABEST group has been pursuing. The first one, it is the search for reliable and more durable sensing systems to install in civil engineering structures. The application of the sensor holders and their protection, in special when construction works are going-on, with a great rate of success is a problem to solve. The confidence in the measurements obtained by the sensors in long term monitoring is an open question that is being gradually solved. The second topic concerns with the development and implementation of numerical models more suitable to help in interpreting the data collected by SHM systems. Adequate structural modelling is essential to understand the monitoring data and to identify any damage. The existing numerical models has been applied with success but new and simpler nonlinear models, and more versatile software to be coupled with structural monitoring is being demanding.

ACKNOWLEDGEMENTS

The authors are gratefully acknowledged to Foundation for Science and Technology (FCT-MCES) and programs POCTI and POCI by the financial support provided. They also acknowledge the Ph.D. students of the LABEST group of Structural Monitoring, Américo Dimande, Bruno Costa, Helder Sousa, José Matos and Mário Pimentel for their contributions to the present work.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Developing a probability based limit states bridge specification – U.S. experience

J.M. Kulicki
Modjeski and Masters, Inc., Mechanicsburg, Pennsylvania, USA

ABSTRACT: Europe, Canada and the United States have been developing new probabilistic limit states bridge design specifications, the *Eurocode*, the *Canadian Highway Bridge Design Code* (and the earlier *Ontario Highway Bridge Design Code*) and the *AASHTO LRFD Specifications for Bridges* (AASHTO LRFD). This paper summarizes the seven-year long process that led to the decision to develop the AASHTO LRFD, the objectives, and review process culminating in adoption in 1993. The implementation, further development during the decade since, and evolving plan to continue development of the specifications are discussed.

1 THE PAST – HISTORICAL DEVELOPMENT (1986–1993)

1.1 *Background*

The apparent start of the process leading to implementation of load and resistance factor design (LRFD) in the *AASHTO LRFD Bridge Design Specifications* (AASHTO LRFD)(AASHTO 2004) was the initiation of The National Cooperative Highway Research Program (NCHRP) Project 20-7/31, entitled "Development of Comprehensive Bridge Specification and Commentary" (1987), in August of 1986. In reality, the process leading to this decision had started decades earlier. Engineers in Europe had an early start on consideration of limit states design and reliability-based design. Their pioneering efforts provided a history of successful experience which validated the underlying concepts. In 1979, the first edition of the *Ontario Highway Bridge Design Code* (OHBDC) (Ontario Ministry of Transportation 1991) was released to the design community as North America's first calibrated, reliability-based limit states bridge specification. Since that time, the OHBDC has been updated in 1983 and 1992 and re-released. Very significantly, the code contained a companion volume of commentary.

In July of 1986, a group of State Bridge Engineers met with the staff of the NCHRP to consider whether a project could be developed to explore the points raised in the Denver letter. This led to NCHRP Project 20-7/31 "Development of Comprehensive Bridge Specifications and Commentary", a pilot study conducted by Modjeski and Masters, Inc. with the following Scope:

- Task 1 – Review of the philosophy of safety and coverage provided by other specifications.
- Task 2 – Review the Standard Specifications for content and review other AASHTO documents for their potential for inclusion into a Standard Specification.
- Task 3 – Assess the feasibility of a probability-based specification.
- Task 4 – Prepare an outline for a revised AASHTO specification for highway bridge design, and a commentary, and propose an organizational process for completing such a document.

In May of 1987, the findings of NCHRP Project 20-7/31 were presented to the AASHTO SCOBS indicating that seven options appeared to be available for consideration. The Subcommittee requested funding through the AASHTO Standing Committee on Research for the NCHRP to initiate a project to develop a new, modern bridge design specification. This led to NCHRP Project 12-33 which was also entitled "Development of Comprehensive Specification and Commentary" (Modjeski and Masters 1994). Modjeski and Masters, Inc. began work in July, 1988 on a new specification that would introduce a new philosophy of safety, establishing a relationship between the chosen reliability level, the load and resistance factors, and load models through the process of calibration.

1.2 *Introduction to reliability as a basis of design philosophy*

An introduction to probability-based reliability theory can be simplified considerably by initially considering that natural phenomena can be represented mathematically as normal random variables, as indicated by the well-known bell-shaped curve. This assumption leads to closed form solutions for areas under parts of this curve, as given in many mathematical handbooks and programmed into many hand calculators and spreadsheets.

Accepting the notion that both load and resistance are normal random variables, the bell-shaped curve corresponding to each of them can be plotted in a combined presentation dealing with distribution as the vertical axis against the value of load, Q, or resistance, R, as shown in Figure 1. The mean value of load ($\overline{Q}$) and the mean value of resistance ($\overline{R}$) are also shown. For both the load and the resistance, a second value somewhat offset from the mean value, which is the "nominal" value, or the number that designers calculate the load or the resistance to be is also shown. The ratio of the mean value divided by the nominal value is called the "bias". The objective of a design philosophy based on reliability theory, or probability theory, is to separate the distribution of resistance from the distribution of load, such that the zone where load is greater than resistance is tolerably small. A conceptual distribution of the difference between resistance and loads, combining the individual curves discussed above, is shown in Figure 2.

It now becomes convenient to define the mean value of resistance minus load as some number of standard deviations, $\beta\sigma$, from the origin. The variable "β" is called the "reliability index" and "σ" is the standard deviation of the quantity R-Q. From probability theory, it is known that if load and resistance are both normal and random variables, then the standard deviation of the difference is:

$$\sigma_{(R-Q)} = \sqrt{\sigma_R^2 + \sigma_Q^2} \tag{4}$$

Given the standard deviation and considering Figure 2 and the mathematical rule that the mean of the sum or difference of normal random variables is the sum or difference of their individual means, we can now define the reliability index, β, as:

$$\beta = \frac{\overline{R} - \overline{Q}}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \tag{5}$$

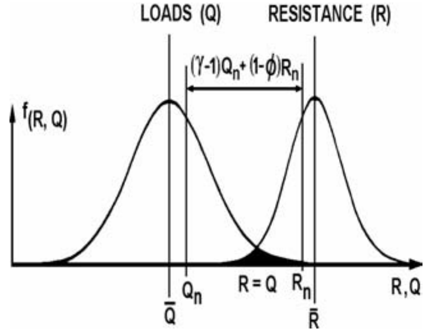

Figure 1. Separation of loads and resistance.

Calibration was done using a suite of test bridge designs. A simulated set of 91 bridges based on 175 real bridges was developed. Full tabulations of these bridges and their representative amounts of the various loads are presented in Nowak (1993).

The reliability indices were calculated for each simulated bridge for both shear and moment. The range of reliability indices which resulted from this phase of the calibration process is presented for the 91 simulated bridges in Figure 3 from Kulicki, et al (1994). It can be seen that a wide-range of values were obtained using the current specifications, but this was anticipated based on previous calibration work done for the OHBDC (1991).

Reliability indices were recalculated for each of the 91 simulated cases using the new load and resistance factors. The results are shown in Figure 4 which shows that the new calibrated load and resistance factors, and new load models and load distribution techniques work together to produce very narrowly-clustered reliability indices close to the selected target of 3.5.

1.2.1 *Rationale for a new live load model*

Early editions of the AASHTO Standard Specification contained a representation of a truck and/or a group of trucks for use in design. The very earliest editions of this Specification contained a single unit truck weighing up to 178 kN (20 U.S. customary tons), which was known as the H20 truck. Lighter variations of this vehicle were also considered and were designated as HXX, e.g., H15. Groups of 133 kN trucks, with an occasional 178 kN truck, were also utilized as a truck-train.

In 1931, the first edition of the Standard Specification instituted the "lane load", which consisted of a uniform load of 9.3 N per mm and a moving concentrated load or loads. A concentrated load of 116 kN was used for shear and for reaction, two 80 kN concentrated loads were used for negative

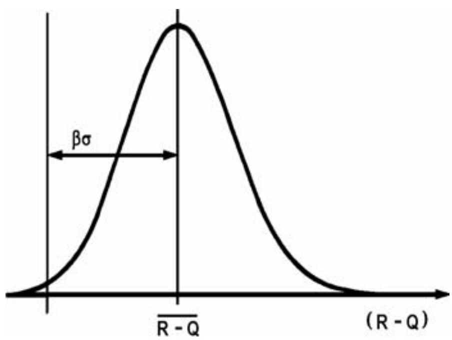

Figure 2. Definition of reliability index, β.

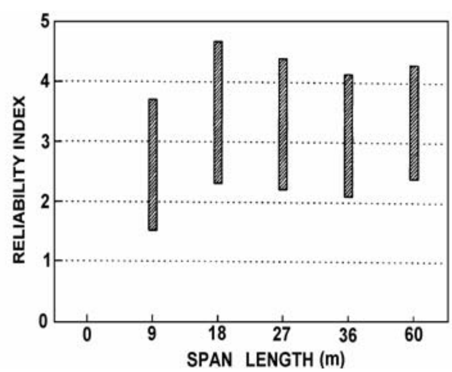

Figure 3. Reliability indices inherent in the 13th edition of the Standard Specifications.

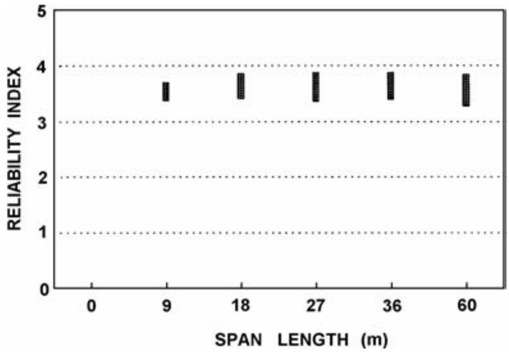

Figure 4. Reliability indices inherent in AASHTO LRFD.

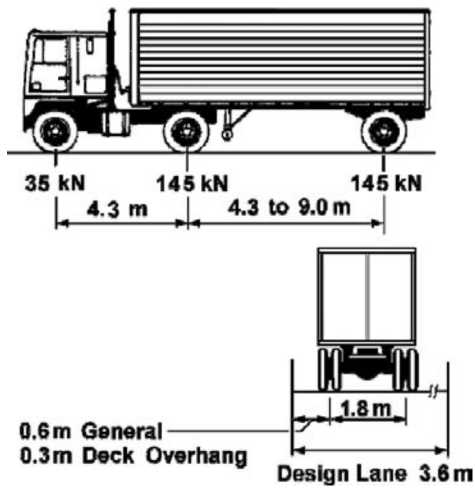

Figure 5. Configuration of HS20 truck.

moment at a support and were positioned in two adjacent spans, and a single 80 kN load was used for all other moment calculations.

In 1944, the design truck was extended into a tractor-semi-trailer combination, known as the H20-S16-44, commonly referred to as simply the HS20 truck shown in Figure 5.

During the late 1970s and early 1980s, some states raised their design load to HS25, or 125% of the HS20 truck and/or the Lane Load. During the construction of the interstate system, an additional design load, known as the "Interstate Load", or design tandem was also introduced, and this consisted of two 110 kN axles separated by 1.2 m. Some of the states which raised the design load to HS25 also increased the weight of the design tandem to 133 kN.

Over the years, many states have written exclusions into their regulatory policies, which permitted some vehicles in excess of legal loads to operate in an unrestricted manner. These loads are sometimes referred to as "grandfather provision" loads.

As a result of the evolution summarized above, it became increasingly clear that the HS loading did not bear a uniform relationship to many of the vehicles allowed on the roads. In developing a new design specification, it became apparent quite early in the process that if the objective was a more uniform and consistent safety of bridges, a new live load model would be necessary in order to produce that consistency.

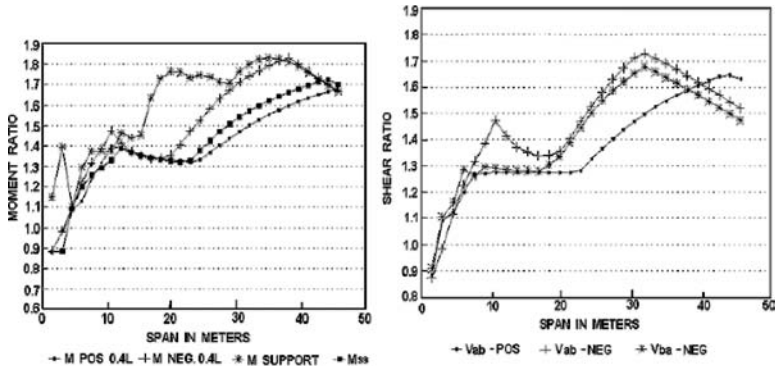

Figure 6. Moments and shears from exclusion vehicles normalized to HS20.

In 1990, the Transportation Research Board published Special Report 225, summarizing a study into allowable bridge loads or allowable vehicle loads. A group of 21 vehicles were identified in Special Report 225 which were legal in various states as exemptions, or "exclusions" to state weight laws, these and other "families" of vehicles were considered by calculating the envelope of force effects on bridge structures for:

– Centerline moment of a simply-supported girder.
– Positive and negative moment at the 0.4 L point of a two-span continuous girder.
– Negative moment at the interior pier of a two-span continuous girder.
– Positive and negative end shear and negative shear at the interior support of a two-span continuous girder.

The family of exclusion loads was the most significant of the several families studied. Figure 6 shows a comparison of the various moment-type force effects, identified above, for spans from 6 to 45 m generated by the exclusion vehicles normalized to the HS20 truck. Thus, a complete match of force effects, indicating that the HS20 vehicle was an accurate and representative model of the exclusion loads, would be indicated by a horizontal line passing through the vertical axis at a value of 1.0. Corresponding information for the shear force effects identified above are also shown in Figure 6 in which V_{ab} is the shear at the simply-supported end and V_{ba} is the shear adjacent to the interior support. Figure 6 confirmed that the HS20 vehicle is not representative of current loads on the highways.

A notional design load, know as the HL93 loading, was chosen after consideration of five possibilities which included an equivalent uniform load, a single multi-axle vehicle, a family of three axle strings and an associated before and after uniform load, HSXX, and the HL93 loading consisting of a combination of either a pair of 110 kN tandem axles and the uniform load, or the HS20 and the uniform load.

The HL93 had the best fit to the exclusion vehicles as indicated in Figure 7 which was developed in a manner similar to that in Figure 6, except that results are normalized to the HL93 loading.

1.2.2 *Rationale for new live load distribution factors*

One of the simplifications that has existed in the AASHTO Specifications, virtually from its beginnings, was an attempt to produce a simple way of determining the amount of live load carried by one element of a bridge system, e.g., one longitudinal girder. This gives rise to a so called distribution factor which is commonly written as "S/D" where S is the spacing between longitudinal elements and D is a constant. Early in the Interstate era, when girder spans were relatively short and the elements were relatively close together, this simple expression yielded reasonably realistic results. However, as longer girders replaced truss spans out to about 150 m, and the girder spacing changed from around 2.5 m to 3.5 m to 4.5 m, the simple S/D expression became increasingly more unrealistic. Fortunately, the results obtained with this simple approximation were usually quite conservative.

Early grid analyses of bridges, the growing need for curved structures, the competition between the steel and concrete industries, the requirement for alternative designs in steel and concrete, and the rise of contractor alternatives and value engineering all drove bridge industry to improve on S/D distribution factors. The decision was made to base load distribution in the LRFD Specifications on a two-level approach. The first level is to provide a relatively simple set of equations; the second level is to use two- or three-dimensional methods. The simplified equations were based on the work of Zokaie, et al (1991), done under the auspices of the NCHRP and AASHTO Committee T-5 for Loads and Load Distribution. Generally speaking, the equations developed were somewhat more complex than S/D because they attempted to take into account the relative longitudinal to transverse rigidity of the bridge, as well as the influence of span length. One such equation, when applicable to bending moments in interior girder-type bridges, is given below.

Girder Distribution Factor from LRFD,

$$g = 0.075 + \left(\frac{S}{2900}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{Lt_s^{3}}\right)^{0.1} \tag{9}$$

in which: $K_g = n(I + Ae_g^2)$ (mm^4)
where A = area of girder (mm^2); L = span of girder (mm); S = spacing of girders or webs (mm); t_s = depth of concrete slab (mm); n = modular ratio of girder material to slab material; I = girder moment of inertia (mm^4); e_g = eccentricity of the girder (i.e., distance from centroid of girder to mid-point of slab) (mm).

Figure 8, taken from Nowak (1993), shows a comparison of the distribution factor S/D and the results of Equation 9 neglecting the last term. Neglecting the last term, i.e. using $(K_g/Lt_s^3)^{0.1} = 1.0$, is generally to be conservative. As shown in Figure 8, the distribution factor produced by S/D is

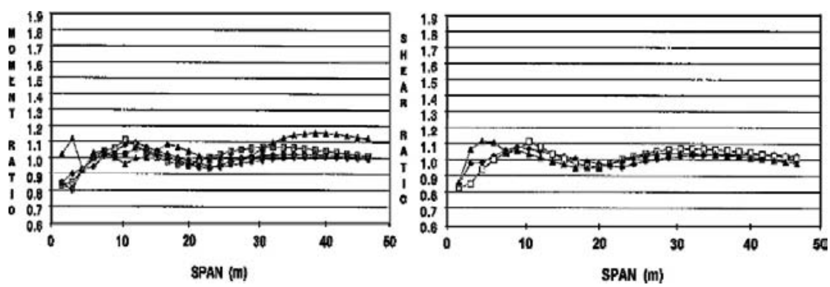

Figure 7. Moments and shears from exclusion vehicles normalized to HL93.

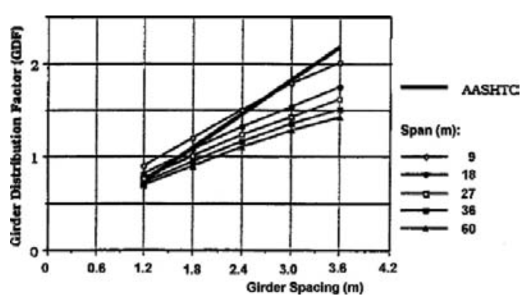

Figure 8. Initial (1992) comparison of S/D and LRFD distribution factor.

generally conservative, sometimes as much as 40% conservative for the girder spacings and spans shown.

2 NEW ADDITIONS TO AASHTO LRFD

Over this past decade, a companion AASHTO *LRFD Bridge Construction Specifications* (AASHTO 1998), *LRFD Specification for Movable Bridges* (AASHTO 2001), and the *Load and Resistance Factor Rating Manual* (LRFR) (AASHTO 2003) were also developed. This produced a well-rounded set of design and administration documents based on limit states design.

Foundation design provisions received considerable attention and were made more current and user friendly. A major revision to the foundation specifications occurred in 1996 when various collections of tabular data and design aids were re-introduced to the AASHTO LRFD in order to make it more useable. Similarly, major improvements and updating of design processes for retaining walls, and MSE walls in particular, had been developed based largely by the efforts of SCOBS Technical Committee T-15. These provisions eventually replaced Section 11 of the AASHTO LRFD.

Technical Committee T-15 of AASHTO updated Section 10, Foundation Design, in the ’06 Interim Specifications. The new provisions are based on major advances in foundation design, as well as refinements in resistance factors resulting from a relatively large body of test results which have recently become available. Much of the background work was developed in NCHRP Projects 24-17 “LRFD Deep Foundation Design”, 12-55 “Load and Resistance Factors for Earth Pressures on Bridge Substructures and Retaining Walls” and 12-66 “AASHTO LRFD Specifications for Serviceability in the Design of Bridge Foundations”. Background in calibration with emphasis on geotechnical application can be found in Allen and Nowak (2005).

The American Segmental Bridge Institute’s (ASBI) *Guide Specifications for the Design and Construction of Segmental Concrete Structures* was updated to LRFD (AASHTO 2003), and the bulk of those provisions were incorporated in an Interim to AASHTO LRFD.

In the U.S., the design of steel curved girder bridges has always been the realm of a Guide Specification (AASHTO 1993). A major updating of AASHTO guide documents in this area was undertaken by NCHRP Project 12-38 (Hall et al. 1999), resulting in the 2003 edition of the *AASHTO Guide Specifications for Horizontally Curved Steel Highway Bridges*. Throughout much of the 1990’s a research project on curved girder bridges was being pursued under the auspices of the FHWA’s Turner-Fairbank Highway Research Laboratory. In order to capitalize on information being developed in that research and simultaneously to provide a further updated curved girder specification compliant with AASHTO LRFD, the NCHRP initiated Project 12-52 (2005). As this research continued, analytic studies indicated that it would be possible to produce a combined specification for curved and straight steel highway bridges. As a first step in this process, the provisions for steel plate girders and box girders were totally rewritten and adopted by AASHTO in the Third Edition of the AASHTO LRFD. The ’05 Interim specifications contained the curved girder specific provisions which resulted in a virtually seamless specification. Arguably, straight girders are now a special case of curved girders.

Live load distribution factors, developed under NCHRP 12-26, “Distribution of Wheel Loads on Highway Bridges” (Zokaie 1991), had been a continual source of additional refinement. As the design of some types of bridges have evolved since that project was completed, it was clear that provisions were needed for designs outside the range of parameters for which some of the NCHRP 12-26 work was developed. Several times during the nine-year history of NCHRP 12-42, studies were undertaken by Modjeski and Masters, Inc. and one of the original researchers, Dr. Toorak Zokaie, to provide the basis for revisions to extend and clarify the applicability of various load distribution equations.

There is continued interest in trying to simplify the lateral load distribution provisions, and several states have developed state-specific implementation guidance in that area. Additionally, a project on simplified load distribution is also underway, NCHRP 12-62. The results of other NCHRP projects including 12-39 “Design Specifications for Debris Forces on Highway Bridges”, 12-48 “Design of Highway Bridges for Extreme Events”, 12-36 “Redundancy in Highway Bridge Superstructures” may yet be incorporated into future editions of the LRFD Specifications.

3 CLOSING COMMENTS

Fourteen years will have elapsed between the 1993 adoption of AASHTO LRFD and the FHWA 2007 deadline. This is certainly longer than might have been anticipated shortly after adoption. However, the intervening years have seen many challenges for the bridge community. To put this in perspective, after 25 years, only about one-third of the states were using Load Factor Design by the late 1990’s. Change takes time. The FHWA has issued a directive that all Federally-funded projects would be designed using the LRFD Specifications starting in the Year 2007.

ACKNOWLEDGEMENTS

This paper summarizes 18 years of work by many people. Almost 60 individuals contributed to the 1st Edition of the AASHTO LRFD. Scores of researchers, panel members, and individual engineers have contributed to the later enhancements. Appreciation is extended to all, to Dr. Thomas P. Murphy who reviewed the paper and suggested improvements, to Dr. Danielle D. Kleinhans who helped extract the summary paper, and to Ms. Diane M. Long who prepared the manuscript.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Stonecutters bridge – durability, maintenance and safety considerations

M.C.H. Hui & C.K.P. Wong
Major Works Project Management Office, Highways Department, the Government of the Hong Kong Special Administrative Region

1 INTRODUCTION

Stonecutters Bridge currently under construction in Hong Kong, with a main span of 1018 m, will become one of the longest span cable stayed bridges in the world. It will be the first major bridge situated in the urban area of Hong Kong with Victoria Harbour as the backdrop. The design contains a number of significant aesthetic features – primarily the circular mono-column towers with a metallic look at the upper part, and the steel twin box deck structure etc. Durability, maintenance and safe operation are naturally amongst the major considerations in the design. In view of the prominence of the bridge, it is important to maintain the aesthetic qualities throughout its 120-year design life. It is a challenge for the bridge engineers to achieve this without jeopardising the operation and the structural performance of the bridge. This paper describes the durability, maintenance and safety considerations for the bridge.

1.1 *Background*

Stonecutters Bridge is situated right at the entrance to the very busy Kwai Chung Container Terminal at the western part of Victoria Harbour. In view of its prominence, Hong Kong Highways Department organized an international design competition in 2000 in order to obtain the conceptual design for this land-mark structure. The winning design was subsequently selected for further detailed design development. The construction works of the bridge commenced in April 2004 and is scheduled for completion in end 2008. Figure 1 shows the location of the bridge in Hong Kong and Figure 2 shows an artist impression of the winning design.

1.2 *The bridge*

Stonecutters Bridge is a cable-stayed bridge with two single-column pylons each 298 m high and an aerodynamic twin deck. The total length of the bridge is 1596 m with a main span of 1018 m.

Figure 1. Location of Stonecutters Bridge.

Figure 2. An artist impression of winning design for Stonecutters Bridge.

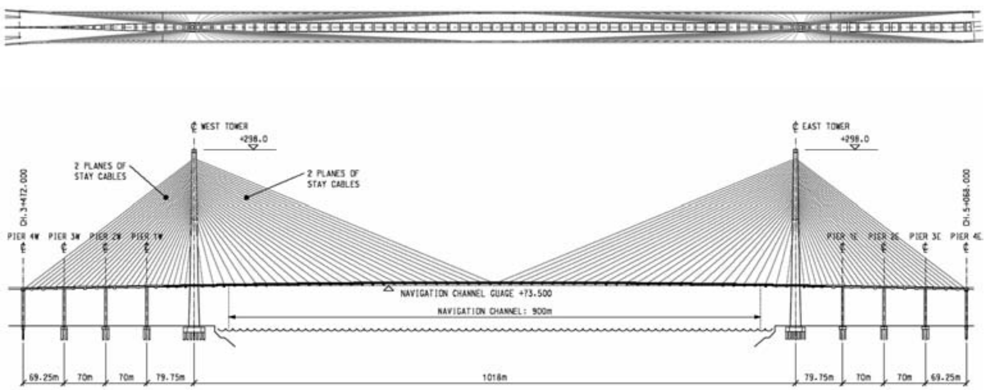

Figure 3. Stonecutters Bridge – plan and elevation.

It has 4 back spans each side of 79.75 m, 70 m, 70 m and 69.25 m. The towers are in concrete up to level +175 m and in steel-concrete composite from level +175 m to level +293 m with the outer skin being stainless steel. The top 5 m is a glass covered steel structure, which acts as an architectural lighting feature and provides storage space for maintenance equipment. The bridge deck at the central span and in the vicinity of towers will be of steel whilst the side spans will be of concrete. The twin longitudinal deck girders are separated by 14.3 m and are connected by cross girders at 18 m and 10 m intervals in the central span and side spans respectively. The 2 planes of stay cables take a modified fan arrangement, anchored at the outer edges of the deck also at 18 m spacing in the central span and 10 m spacing in the back spans. Please see Figure 3 for the general arrangement of the bridge.

2 DURABILITY CONSIDERATIONS [1]

2.1 *Introduction*

Whilst great efforts have been made to obtain a high standard quality design through the conduct of an international design competition, it is equally important to ensure that the bridge will be durable and that the special features built into the design can be long lasting with minimum maintenance effort. The conduct of a proper durability assessment will provide the necessary framework for achieving that goal.

The following sections will be devoted to the discussions of some special measures adopted in Stonecutters Bridge project from a durability point of view.

2.2 *The lower part of the bridge tower*

The lower part of the tower is of reinforced concrete structure. At the lower section up to +25 m level, the concrete must be considered as being exposed to sea spray even though the tower is 18 m away from the seawall. In other words, a “Very Severe” exposure will be encountered by the structure.

For durability design of Stonecutters Bridge, the covers for the concrete have been increased over SDMHR nominal cover requirement to 60 mm to provide good chloride resistance for the towers. In addition, we have adopted a design where the outer most vertical layer of rebars and links are stainless steel reinforcements. In this case however, the calcium nitrite corrosion inhibitor will not be required. Theoretical modeling predicts that such a design will provide a design life in excess of 120 years.

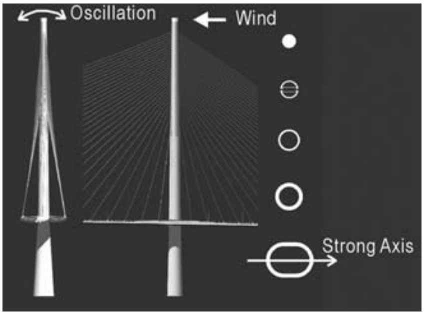

Figure 4. The tower and twin deck of the winning design.

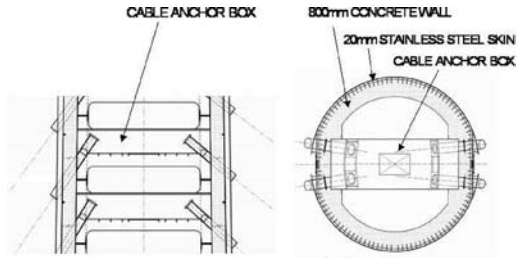

Figure 5. The typical sectional view of the upper part of tower.

Figure 6. The 1:1 mock-up of the stainless steel shell.

2.3 *The upper part of the bridge towers [2]*

The design of Stonecutters Bridge was originated from the winning design of a competition. The upper part of the tower of the winning design is circular shape steel structure. It is one of the most distinctive features of the design, which creates a sense of a “modern-look” for the bridge to be built in the twenty first century (see Figure 4). It is of paramount importance to maintain these features in the detailed design development. Wind tunnel tests however showed that the lightweight tower would be subject to vortex shedding vibration which in turn would pose a high risk of large amplitude cable-vibration due to linear resonance. To improve the dynamic behaviour while still maintaining the appearance, the upper tower section was designed as a steel and concrete composite structure in order to lower the natural frequency of the tower’s fundamental mode of vibration to avoid coinciding with that of the longest cable’s. Composite action is achieved by connecting the skin to the concrete wall by means of shear studs.

During the detailed design phase, Highways Department decided to further improve the tower design by using a stainless steel shell instead of ordinary carbon steel to further enhance the durability of the metallic finishes without the need to undergo regular repainting of the steel surface. A duplex austenitic/ferritic steel has been selected for this element to provide the required combination of strength and corrosion resistance. Figure 5 shows the detailed arrangement for the composite section of the tower.

A specialist, Ancon Buildings Products Ltd, in the U.K, was appointed to carry out a 1:1 mock-up of the fabrication of the stainless steel shell unit (Figure 6). The fabrication works included welding of pre-rolled stainless steel units with shear connectors and the connection between different units by bolts and nuts. The purpose of the assignment was to demonstrate the achievability of a good finish of the welding on the stainless steel surface.

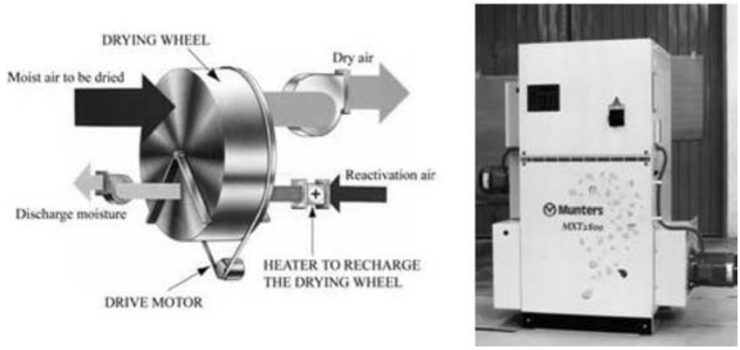

Figure 7. The operation of the dehumidification system (L) and a typical dehumifier (R) (Courtesy of Munters (HK) Ltd.

2.4 *The steel deck*

The steel deck is composed of twin box girders, which will be fabricated in sections from welded steel plate. The external deck surface will be protected by the following corrosion protection schemes, or equivalent:

Coating	Material	Thickness (μm)
Primer	Zinc Rich Epoxy	40
Barrier 1	Epoxy Micaceous Iron Oxide	150
Barrier 2	Epoxy Micaceous Iron Oxide	150
Finish	Polyurethane	50

All coats to be shop applied.
Preparation: Blast clean to Sa 2½ of ISO8501-1.

This system is expected to have a life to first maintenance of 20–30 years after which touch up work and repainting will be required.

The internal atmosphere of the deck boxes will be controlled to provide a relative humidity of less than 60%. This will effectively prevent corrosion of the internal surface of the steel boxes and so eliminate the need for the sophisticated corrosion protection scheme used on the outside. The operation cost for the dehumidification system is estimated to be a few times lower than the repainting cost of the inside of the deck. Figure 7 shows the operation principles of the dehumidification system and a typical dehumidifier for bridge deck boxes.

3 MAINTENANCE CONSIDERATIONS

3.1 *Wind and Structural Health Monitoring System (WASHMS) [3]*

A Wind and Structural Health Monitoring System (WASHMS) will be installed on Stonecutters Bridge to monitor the response of the bridge to different loading conditions including wind loading, temperature loading and highways loading. The WASHMS includes a set of sensors and their corresponding interfacing units for input signal gathered from various monitoring equipment such as anemometers, temperature sensors, dynamic weigh-in-motion sensors, corrosion sensors, hygrometers, strains gauges, displacement transducers, GPS and accelerometers etc. The system will play a key role in securing bridge integrity, minimizing maintenance cost, and maintaining longevity of bridges. It can provide verifications of design/analysis assumptions and important parameters for the operator to plan his inspection and maintenance schedules/strategies to ensure that the bridge can be operated in a safe and reliable manner. Figure 8 gives the layout of the sensory systems to be installed on the bridge.

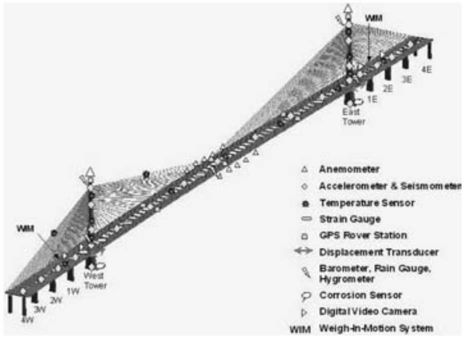

Figure 8. Layout of the sensory systems to be installed on Stonecutters Bridge.

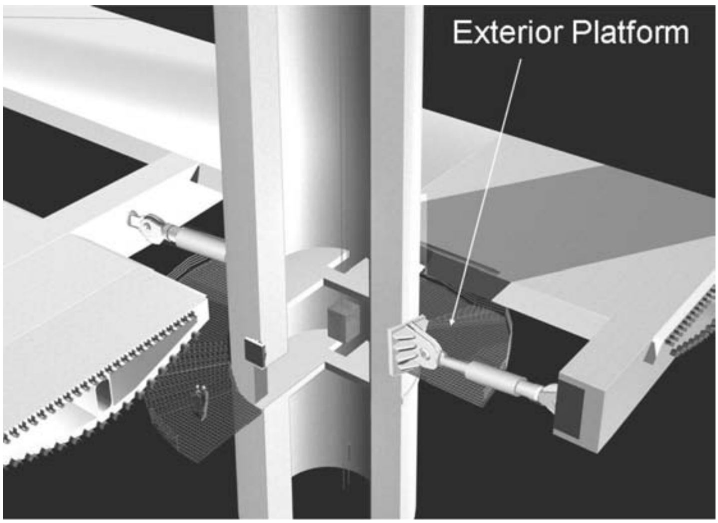

Figure 9. The exterior platform underneath the deck.

3.2 *Maintenance access [4]*

It is a basic principle in the design that all important parts of the structure should be accessible and facilities should be designed to meet this principle. Access to all important parts of the structure can be achieved without any disruption to traffic, which given the strategic nature of Stonecutters Bridge as a major route to the Hong Kong International Airport and Container Ports, is a major consideration.

From the main entry point at the base of the tower the full height of the tower is accessible by an internal rack and pinion lift. An exterior platform (Figure 9) is provided around the tower just below the deck level and is accessible from a chamber within the tower. From this platform entry is gained to the interior of the deck. The transverse bearings and hydraulic buffers for supporting the deck at the tower are also accessed from this platform. Inside the deck a walkway and a shuttle train provide access along the entire length of the deck for personnel and equipment (Figure 10). Transverse walkways between the twin girder can be gained via the cross girders. At the end of the deck a chamber allows access to the expansion joints and bearings. The exterior of the towers above deck level is accessible from a cradle suspended by a permanent derrick on the tower top (Figure 11). The exterior to steel deck and the stay anchorages are accessible from underslung gantries (Figures 12 and 13). Finally a gantry is used for access to the stay cables which would be deployed from a lorry temporarily parked on the hard shoulder.

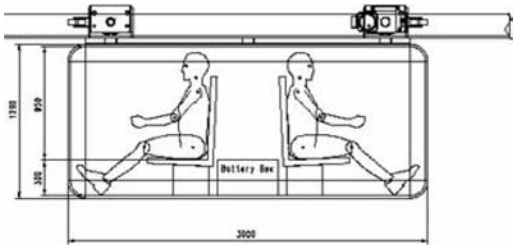

Figure 10. Internal shuttle inside deck.

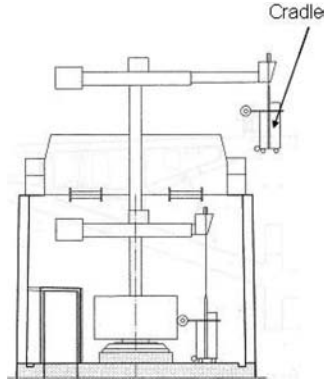

Figure 11. Tower top gantry.

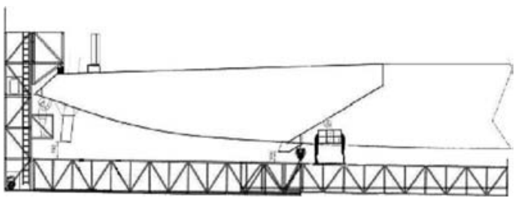

Figure 12. Steel deck gantry.

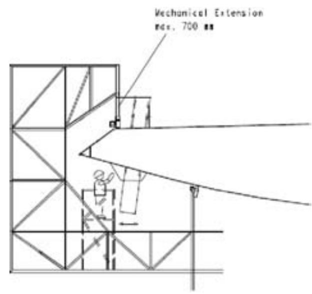

Figure 13. Concrete deck gantry.

4 OPERATIONAL SAFETY CONSIDERATIONS

4.1 *Navigation safety*

Stonecutters Bridge will straddle the Rambler Channel at the entrance to one of the busiest container ports in the world. In the design of the bridge, we have allowed for a minimum soffit level of 73.5 m.P.D. to provide adequate air draft for the passage of vessels of the next generation.

4.2 *Wind management*

A wind management system will be implemented on Stonecutters Bridge during strong winds or typhoons to ensure safe driving by motorists. When the measured gust wind speed at the bridge site exceed certain thresholds, corresponding traffic management will be implemented. U-turn facilities will be provided to facilitate vehicles to turn back to alternative route in case the bridge is closed under strong wind condition.

4.3 *Parapet impact tests*

Two designs of parapet were considered suitable for Stonecutters Bridge during the design stage – tension wire type and 4-rail type. In view of the better protection to motorists being offered by the 4-rail type, it was decided to adopt the 4-rail type, subject to the satisfactory performance of the deck from an aerodynamic viewpoint. See Figures 14 and 15 for the parapet tests employing small cars. Upon checking the vortex shedding vibration using a 1:20 scale section model in the National Research Council of Canada, the 4-rail parapet design is confirmed to be satisfactory with slight modification to the windward edge where a semi-circular fairing will need to be attached to increase the airflow through the parapet rails to suppress vortex shedding vibration.

Figure 14. Parapet impact test for tension wire parapet.

Figure 15. Parapet impact test for 4-rail type parapet.

4.4 *Reserved capacity of cables*

The cables to be used on Stonecutters Bridge are of Prefabricated Parallel Wire Strand (PPWS) type. The PPWS cable will be placed in a tight fit high-density polyethylene (HDPE) tube for corrosion protection purpose. The design has allowed for the extreme scenario for the concurrent rupture of two adjacent stay cables. The design life of the cables is 60 years. During the life of the bridge, cables would need to be replaced at least once. The design of the bridge has catered for this rare but yet important/unavoidable operation.

4.5 *Cable vibration control*

In order to avoid discomfort to motorists and to cater for fatigue design of stay cables, we have set out an acceptance criterion for cable vibration. When the 10-minute mean wind velocity at 150 m height is below 21 m/s, the maximum transverse amplitude of cable vibrations during a period of 10 minutes shall be less than 1/1700 times the cable length. During the design stage, we have conducted wind tunnel tests to ascertain that this criterion can be met with appropriate mitigation measures (see Figure 16). The Contractor shall conduct further wind tunnel tests to demonstrate that the cables that they intend to provide can meet this criterion after incorporation appropriate mitigation measures including adding dimples on the cable sheathes and cable dampers.

4.6 *Vibration control for the tower*

Tuned Mass Dampers (TMD) will be installed at the tower tops to mitigate the possible wind induced vibration perpendicular to the bridge axis. The TMD shall have a damping ratio exceeding 2.5% relative-to-critical. The damping device shall be designed in such a way that the damping frequency can be tuned to the actual oscillation frequency in service condition. It is expected that the TMD will be a 3-stage pendulum type so that the movement of the pendulum will be within the limited space available at the tower top.

Figure 16. Cable vibration test.

4.7 *Vibration control for the deck*

Whilst a number of aerodynamic investigations have been conducted for this record-breaking bridge to ensure that the vortex shedding vibration of the deck will be suppressed by providing guide vanes underneath the deck and adopting semi-circular fairings for the 4-rail parapets, wind tunnel experiment did reveal that vortex shedding vibration would likely to occur in the case where the deck is packed with a long queue of vehicles. Investigation is being conducted to explore the practicable means to retrofit TMD's for the deck should they become necessary in the future.

5 CONCLUDING REMARKS

The planning, design, construction and operation of Stonecutters Bridge constitute a lot of challenges to bridge engineers. The quality expected of a landmark structure justifies the conduct of an international design competition to obtain an elegant design. After that is achieved, the next important mission is to ensure that such a design will be buildable, durable, easy to maintain and safe to operate. A lot of efforts have been put into the various stages of the project to achieve these goals. With the construction of the bridge commenced in April 2004, the bridge is now at the most critical stage of its life where all the planned and design measures would be implemented to ensure that the bridge will be healthy when it is borned by the end of 2008.

ACKNOWLEDGEMENT

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Technical Contributions

Bridge management systems

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)
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The first regional level bridge management system application in Italy

Elisa Spallarossa
Archimede s.r.l. – Engineering and Architecture, Genova, Italy

ABSTRACT: The activities described in this paper have been performed within the collaboration between Archimede s.r.l. and Regione Liguria.

Archimede is an italian engineering company, performing design and engineering consulting and responsible on Pontis italian application.

Regione Liguria is one of the twenty Regions Italy is administratively divided in. Its territory is extended for 5421 sqm area and has 3957 Km of paved roads, which comprise 376 Km of highways, 942 of statal roads and 2639 Km of counties roads.

From a legislative point of view, Regione Liguria activated a Regional Law (Law number 3 dated 22 January 1999) to transfer road management to Counties, while maintaining the strategic role in programming and coordinating activities. For what concerns important structures as viaducts and bridges, the Regional Infrastructure, Transportation and Civil Protection Assessorship recognized the relevant impact of maintenance planning and programming on safety and financial management, for this reason a study was performed to set the BMS that best fitted Regione Liguria needs. Regione Liguria chose Pontis because it assessed its own needs (different public administrations with different needs might opt for a different system), being developed by AASHTO, that is a non profit association, and being widespread among nearly all the United States. Furthermore using Pontis provides a systematic approach for a bridge management system first application.

The main idea has been to involve the four Liguria Counties (Genova, Savona, Imperia, LaSpezia) in the creation of the "Regione Liguria Bridge Catalogue" establishing a cooperation that has been sealed with a mutual agreement (called "Protocollo d'intesa") signature by Counties and Region Infrastructures Assessors. The paper describes the main statements included in this document and Regione Liguria Pontis implementation state of the art. In particular bridge catalogue collection Guidelines and Pontis training for regional and counties technicians written and hold by Archimede are described.

For what concerns future issues, the paper underlines that to achieve a complete BMS on the entire regional territory the main step to be performed is organizing inspections based on AASHTO Commonly Recognized (CoRe) elements identification and condition states evaluation and that two main difficulties have been dealt with Pontis implementation:

- from a political point of view it is more effective to invest money into new structures rather than in maintaining the old ones.
- public administrations, by now, use the "emergency" maintenance approach in contrast with the "preventive" maintenance.

Concluding remarks underline the strategic role played by Regione Liguria representing the first Italian Public Administration application as, following its example, it is likely other Public Administrations will move towards maintenance management system. The challenge will be to win political aspects and bias.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Development of lifetime maintenance strategies for highway structures based on the experience of a Japanese Highway Agency

M. Matsumoto
Investigation and Foreign Affairs Department, Express Highway Research Foundation of Japan, Tokyo, Japan

D.M. Frangopol
University of Colorado, Boulder, Colorado, USA

ABSTRACT: Nowadays, proper maintenance and management of deteriorating highway structures are becoming serious issues in many countries including the United States and Japan. It is

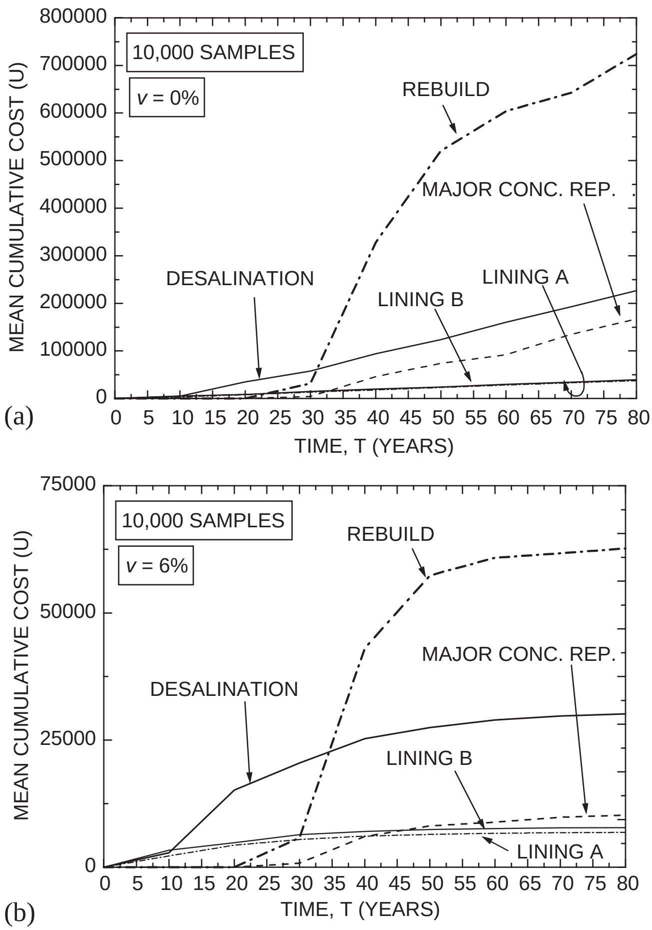

Figure 1. Mean cumulative cost under different maintenance scenarios and different discount rates: (a) 0% discount rate, (b) 6% discount rate.

necessary to apply proper maintenance actions to the highway structures in order to extend the remaining life of existing bridges, and minimize their repair and rehabilitation costs. The main objective of this study is to develop a framework for predicting the future deterioration of highway structures with emphasis on concrete superstructures based on the experience of a Japanese Highway Agency, namely the Nippon Expressway Company. Recent studies revealed that the most relevant mechanisms for deterioration of concrete structures are corrosion of reinforcement due to chloride attack and/or carbonation. In this study condition evaluation method for bridge elements under chloride attack is introduced using a quasi-quantitative grading method. Future condition states of deteriorating concrete structures under chloride attack can be predicted by the chloride ion content X (kg/m^3) and corrosion of reinforcement Y (%) at given points in time, which can be represented by a time-dependent probabilistic distribution. A probabilistic approach to predict the future condition states of the structure is presented. Monte Carlo simulation method is used efficiently to evaluate the effect of propagation of uncertainties on the future condition states of deteriorating structures. Based on the simulation results, the probability of transition of condition states and probability of being in each condition state at given points in time can be predicted. These results can be useful for bridge agencies to predict the number of bridges that need to be repaired or rehabilitated at any given point in time. A methodology to determine the optimum maintenance strategies is discussed based on the comparison of cumulative lifetime maintenance cost under different maintenance scenarios. To find the lifetime maintenance cost for each maintenance scenario, Monte Carlo simulation was performed. Various discount rates are used to help understanding the effect of time value of money on results. From the present value of the expected cumulative maintenance cost under different maintenance scenario, shown in Fig. 1, it is concluded that the preventive maintenance strategy associated with applying lining A is the most cost effective maintenance scenario. In addition, iterative application of preventive maintenance can also guarantee better condition states compared to the corrective maintenance in which maintenance actions are not applied until the onset of the corrosion in reinforcement. The results obtained in this study illustrate that the most attractive maintenance scenario is the one associated with cyclic application of preventive maintenance action with lowest unit cost. The results obtained prove that the preventive maintenance scenario is a better option than corrective maintenance scenario in order to minimize the expected cumulative life-cycle cost of deteriorating structures.

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 ISBN 0 415 40315 4

Life cycle cost optimization of a bridge superstructure considering maintenance history

Y.S. Shin, J.H. Park, Y.J. Ahn & H.S. Lee
Dept. of Construction and Transportation Engineering, Ajou University, Suwon, Korea

ABSTRACT: The optimal design considering life cycle cost is performed for a bridge superstructure consisting of steel box girders and concrete deck. Load carrying capacity curves for the steel girder and concrete deck are proposed to calculate life cycle cost. These curves are derived on the basis of bridge diagnostic results and condition grade curves. The service life of the superstructure is determined considering load carrying capacity curves and maintenance histories. The objective function is set to the total life cycle cost including initial cost, damage cost, maintenance cost, user cost, and etc for the service life. Damage cost is calculated based on the damage probabilities to consider the uncertainty of load and resistance and user cost includes traffic operation costs and time delay costs. The optimal design of the superstructure is performed for various service lives. The validity of the proposed method is verified and the effect of user cost is also investigated through the sensitivity analysis.

1 INTRODUCTION

This paper suggests load carrying capacity curves for the steel girder and concrete deck, as shown in figure 1 and 2, and performed optimal design of a bridge superstructure. These load carrying curves and repair and rehabilitation history are derived on the basis of bridge diagnostic results and condition grade curves suggested by the Korea Infrastructure Safety and Technology Corporation (KISTEC) and used to calculate life cycle. The optimization method is applied to design of a bridge structure with minimal cost, in which the objective function is set to life cycle cost and constraints are formulated on the basis of Korean Bridge Design Specification. Damage cost is calculated based on the damage probabilities to consider the uncertainty of load and resistance. Maintenance cost and cycle is determined by a stochastic method and user cost includes traffic operation costs, time delay costs.

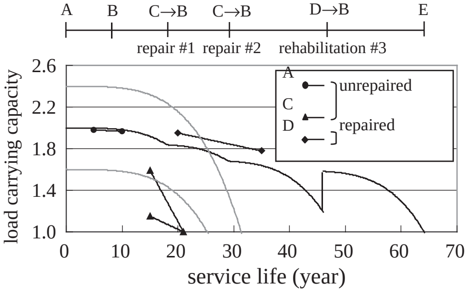

Figure 1. Load carrying capacity curve of concrete deck.

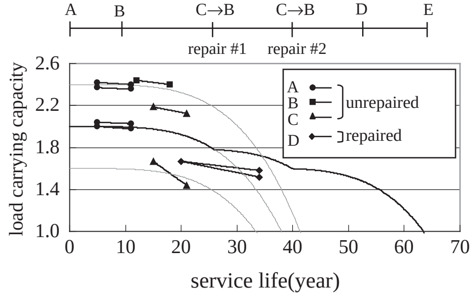

Figure 2. Load carrying capacity curve of steel girder.

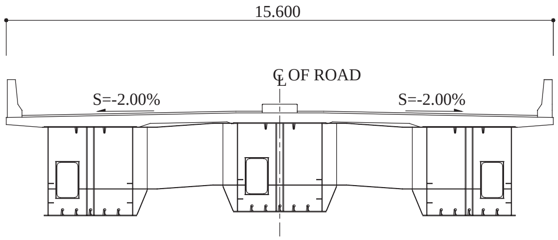

Figure 3. Bridge superstructure.

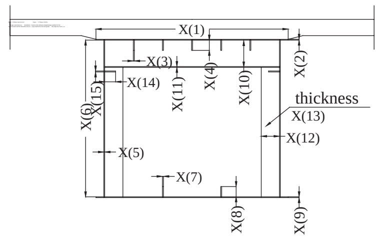

Figure 4. Design variables.

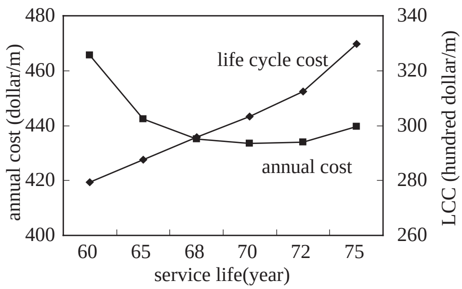

Figure 5. Optimal annual cost and life cycle cost for service lives.

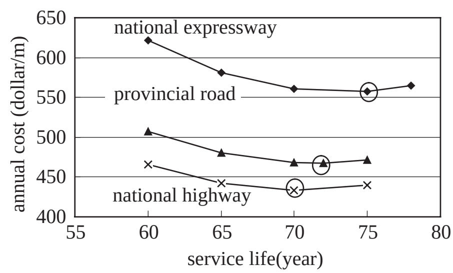

Figure 6. Annual costs for road types.

2 FORMULATION OF OPTIMAL DESIGN OF BRIDGE CONSIDERING LIFE CYCLE COST

The optimal design problem has been formulated as a non-linear mathematical programming problem. The bridge superstructure considered in this study consists of the steel box girders and the concrete deck, as shown in figure 3. The design variables for the concrete deck are slab thickness and area of steel reinforcing bar. Those for the steel box girder are dimensions of flange, web, and stiffeners.

Design constraints are formulated based on the ultimate strength design method for the concrete deck and the allowable stress design method for the steel girder. The objective function is set to the total life cycle cost.

$$C_t = C_I + P_D \cdot C_D + C_M + n \cdot C_R + C_U + C_P \tag{1}$$

Where, C_t is total life cycle cost; C_I is the initial cost; $P_D \cdot C_D$ is the damage cost; C_M is the maintenance cost; $n \cdot C_R$ is the repair and the rehabilitation cost; C_U is the user cost; C_P is the disposal cost.

3 CONCLUSION

In figure 5 the life cycle cost increases gradually while the trend of the annual cost is concave. The optimum service life and designs variables can be determined by use of annual cost. It is more reasonable to use annual cost, which contains initial cost, user cost, maintenance cost and etc, for better economical evaluation. Figure 6 represents the result of sensitivity analysis for various road types. It shows that the optimal service life and minimal annual cost are different for road types. Therefore, the user cost should be considered more appropriately, while the user cost is affected by the various environmental conditions and varies with road types.

In this study, the optimal design model is proposed and used to design a more economical bridge by considering life cycle cost. Also it can be used in optimization of the maintenance such as the time of repair rehabilitation and replacement of bridge members.

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The bridge management system in Osaka-City

M. Ishida & T. Iritani
Osaka Municipal Government, Public Works Bureau

M. Jido
Chuo Fukken Consultants Co., LTD, Osaka, Japan

1 CURRENT CONDITION OF BRIDGES

Osaka-City manages 762 bridges which total length is 47.6 km and which deck area is 722 thousand m^2. AS shown in Figure 1, most bridges were intensively constructed in 1920s when the city basis service progressed remarkably and in 1960s, 1970s of the high economic growth period. Therefore, in the near future, the rehabilitation/repair will increase rapidly due to deterioration of structures. There are no bridge which age is over 100 year now, but there will be aging time pf bridges because about 100 bridges will become over 100 years old in 30 years. If over 100 year bridges would be re-constructed for 30 years, there would be expected to amount about 5,000 million yen per year for re-construction. Therefore, the influence over the social economy activity is worried about, because of increase of financial load and closed traffic by re-construction works. Therefore, Osaka-City has to expand bridge life so as to maintain service level of bridges and, to smooth and decrease financial load. Osaka-City have established committee for Osaka-City Bridge Management System (OBMS) from 2003 (chairperson is professor emeritus Eiichi Watanabe) and constructed new maintenance management system. The outline of OBMS is described as follows.

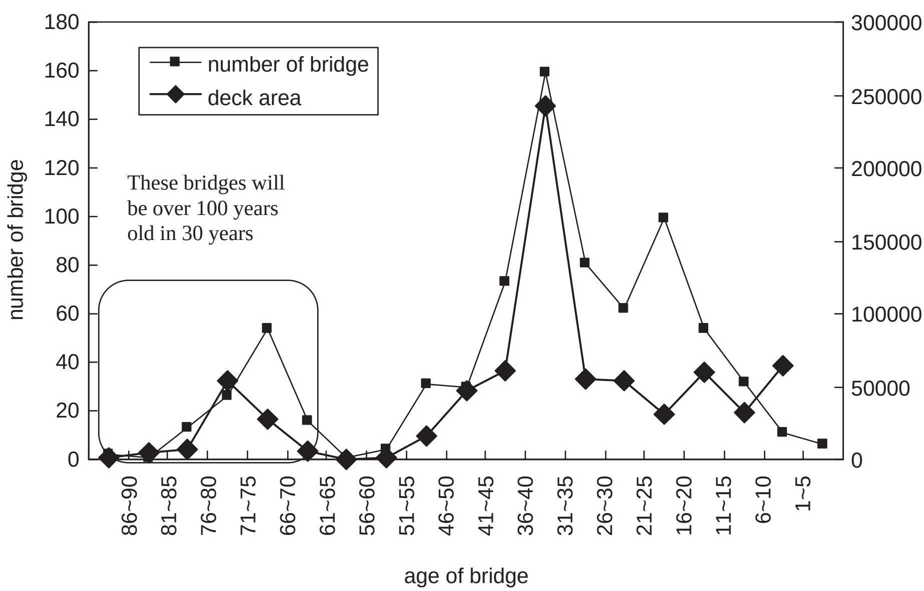

Figure 1. Distribution of age of bridges in Osaka-city.

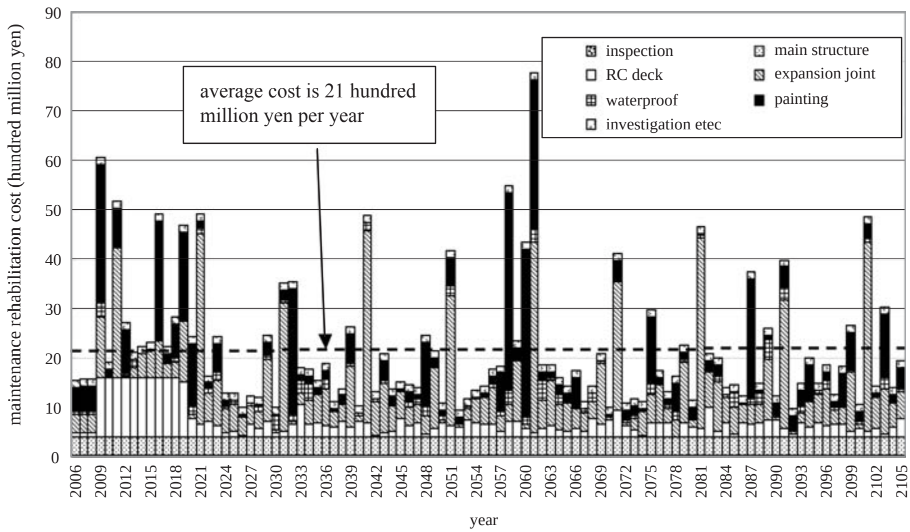

Figure 2. Transition of total maintenance cost.

2 OUTLINE OF OBMS

The basic policies of OBMS are the following four items

1) Expansion of bridge life by preventive maintenance
2) Construction of Preventive Maintenance Support System
3) Deterioration prediction based on inspection data
4) The maintenance management considering bridge characteristics

3 BRIDGE MAINTENANCE MANAGEMENT PLANNING

Figure 2 shows transition of total maintenance cost for 100 years when optimal repair policy is implemented. This cost is calculated by accumulating the result of each element. When service life of bridges are expanded by preventive maintenance which implements optimal repair minimizing LCC, then bridges are expected to be maintained at optimal service level with about 2.1 thousand million yen per year for 100 years. In case of replacing bridges which are over 100 years old, it is expected to cost about 5 thousand million yen per year with only replacement cost. Therefore, When attempting to convert for preventive maintenance, financial load will reduce substantially.

Repair budget in 2005 fiscal year is about 150 thousand million yen. In the future, optimal maintenance management is planned by reviewing this empirical study, and considering budget constraint.

4 THE POLICY IN THE FUTURE

This study describes the effort for constructing new bridge maintenance management system of Osaka-City.

Through this study, basic policy of OBMS is established, and basic flame work of bridge maintenance management support system for preventive maintenance is constructed.

In the future, accuracy of the bridge maintenance management support system is improved by accumulating of inspection and repair data, reviewing the bridge maintenance management support system, through implementation of management cycle of OBMS. Also Inspection manual, and decision making rule of bridge renewal will be prepared. After preparation of decision making rule of bridge renewal, management control system will be constructed so as to implement maintenance and renewal effectively and rationally by combining OBMS and the decision making rule for renewal.

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A bridge management system applied to a set of Portuguese bridges

J.O. Almeida
Instituto Politécnico de Viana do Castelo, Viana do Castelo, Portugal

R. Delgado
Faculdade de Engenharia da Universidade do Porto, Porto, Portugal

ABSTRACT: Portuguese people still haven't forgotten two important accidents occurred with two bridges recently, one in Entre-Os-Rios and another in Lisbon. In fact, the problem is that some other bridges can probably be near the end of their structural or functional period of life now. Therefore, it's urgent to identify those cases and act according to the urgency of intervention.

The guarantee of safety is hardly assured when the reparation works are only implemented if someone, more or less accidentally, notices some damages and alerts to them. In fact, it is urgent to implement a regular inspection on bridges and then treat all the gathered information in order to plan the needed intervention. Besides, it is necessary to do an exhaustive analysis that could assign the real urgent and essential actions.

The implementation of Bridge Management Systems is being recognized as a good way to systematize all that process and minimize the ratio cost/benefits during bridges' life time. To help in the process of inventory and appraisal of a group of highway bridges, a Bridge Management System tool, called *REGpontes*, was developed by Almeida (2003), using a methodology of classification from USA (FHWA 1995).

The referred methodology of classification includes the determination of four partial ratios: structural adequacy and safety (R1), serviceability (R2), essentiality for public use (R3) and special reductions (R4). Those ratios are then balanced, taking into account their relative importance (55% of R1, 30% of R2 and 15% of R3), to calculate a global efficiency ratio. That final ratio can then be very useful to analyze the global situation, to identify intervention priorities and to program the needing for repairing, strengthening and rehabilitation works.

The results of its application on a set of Portuguese highway bridges are presented in Figure 1 and in Figure 2, taking in consideration the relation between the gathered ratios (converted in a 0 to 100% scale, from worse values to better ones) and the main structural material or the major dimensions of the bridges. The global efficiency of the analyzed bridges, in average terms, is almost fifty percent. Analyzing the average obtained for each one of the partial ratios, it is possible to conclude that the selected bridges are better in structural terms and worse in serviceability terms.

The reason for the serviceability insufficiency is mostly related with deficiencies associated to traffic circulation, such as the narrowness of the road face, the volume of traffic, which mainly, results from the oldness of the selected bridges. The utility for public use, which represents the strategic value of the bridge and its importance because of higher daily traffic and/or a long detour length, is the ratio with a bigger standard deviation. In terms of structural adequacy and safety, the metallic bridges present a lower value, mainly because of deficiencies in maintenance.

In Portugal there is a lot to do and it is urgent to act. *REGpontes* is an informatic tool especially developed to guide the management of Portuguese highway bridges. From the presented example of its application it's possible to see that the methodology inherent to the program can be easily adopted with great benefits. With *REGpontes*, the technical and political decision can be objectively supported.

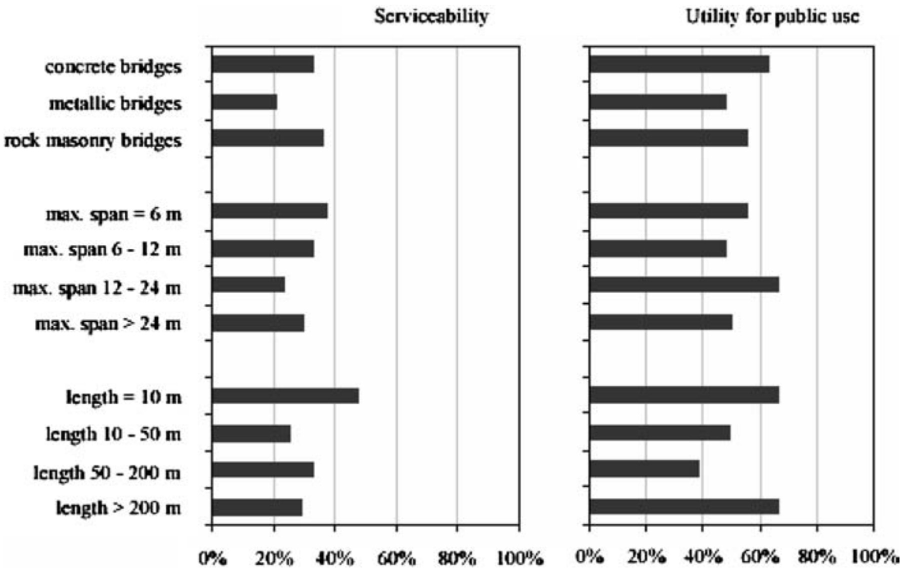

Figure 1. Average results for Serviceability and Utility for public use, in accordance to some main characteristics of the bridges.

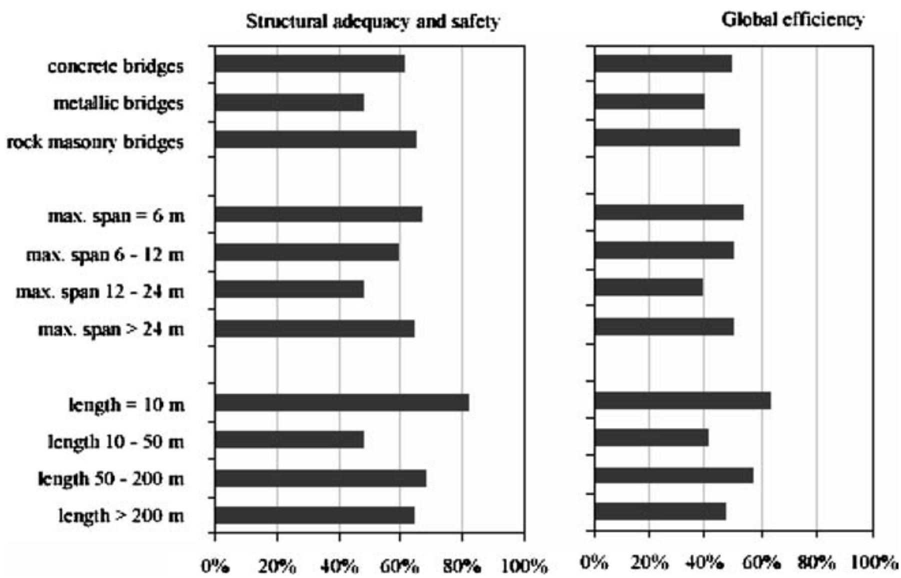

Figure 2. Average results for Structural adequacy and safety and for Global efficiency, in accordance to some main characteristics of the bridges.

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Optimal maintenance strategies for existing infrastructures under seismic risks

T. Koike & I. Aoki
Musashi Institute of Technology, Tamazutsumi, Setagaya-ku, Tokyo, Japan

ABSTRACT: This paper describes optimal maintenance strategies for existing infrastructures to prevent structural damage due to seismic disaster based on a risk management approach.

Many existing infrastructures which are deteriorated by corrosive and climate conditions are always threatened by various natural hazards including earthquake loads. Actually, those structures constructed prior to 1980 in Japan were designed for a particular seismic load which is smaller than the Level 2 ground motion. After the 1995 Hyogoken-Nanbu earthquake, revised guidelines were specified making old infrastructure design fall below acceptable limits. This means these structures are vulnerable to strong earthquake in the future.

Due to the shrinking population of Japan, in recent years, its economy and national budget have been affected. In view of this, a more rational decision making process is necessary for future investment on maintenance activities of infrastructures. Maintenance works therefore must be optimized under the constraint of a limited budget.

The original strength of a structure degrades during its service period, because of the aggressive environment which causes it to undergo various deteriorating effects. A time dependent deterioration model is introduced herein to describe the corrosion effects of the reinforcing steel bars in the RC column of bridge structures. Adding to this effect, the seismic impact provides a significant damage to the structure. So the repair works after the quake is conducted for both the seismic damage as well as the daily deterioration damage. It should be noted that many bridge structures in Japan have already been reinforced. Steel plate covers around bridge piers are for anti-seismic disaster preparedness. Therefore, this study discusses on the restoration strategy when a medium size earthquake causes certain amount of damage to the structure, and also discusses on which strategy can provide the optimal solution for a more severe earthquake of Level 2 ground motion.

Bridge structure is an important element in a transportation lifeline network system. This typical infrastructure is owned by a (recently privatized) highway company in Japan. So the risk management approach (Hoshiya et al, Koike, 2005) is developed herein to take into account the social benefit effect in the net present value estimation.

The maintenance cost includes periodic inspection/repair cost and additional repair immediately after an earthquake. It is understood that the maintenance strategy depends on the residual service period of a structure. If a structure has a residual long lifetime, the maintenance should be enough to cope for possible future earthquakes, while, if its lifetime is too short, the maintenance investment should be limited.

In order to obtain the optimal investment to the maintenance, several maintenance strategies are discussed and compared. Numerical results show that the optimal maintenance strategy strongly depends upon the restoration content at a near future earthquake.

A single column RC bridge pier shown in Fig. 1 is a schematic example of a bridge structure which is designed for resisting the failure due to bending moment and shear force to be produced by structural responses from earthquake effects. Applying the additional steel plate around the bridge column as shown in Fig.1, the reinforced shear strength can reach 1.4 times the initial shear strength at bending failure.

Fig. 2 shows a schematic illustration of the restoration work after a middle size earthquakes denoted by *EQ1* at time T_E. The T_R, T_P, T_X, T_D are the accomplished time of the seismic disaster

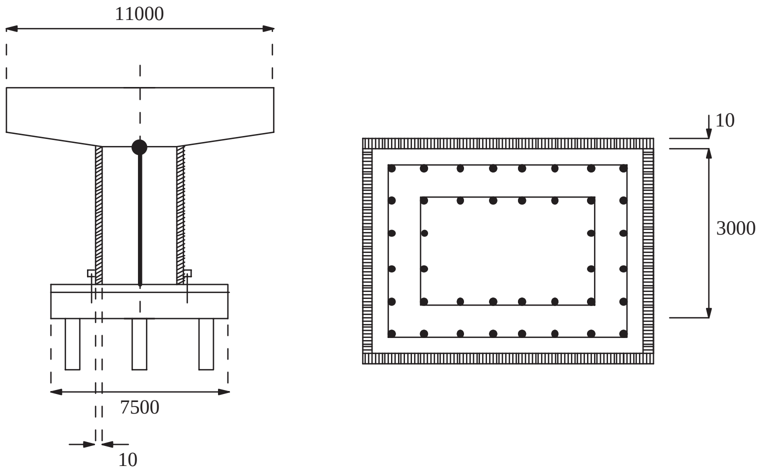

Figure 1. Reinforced bridge pier with steel plate.

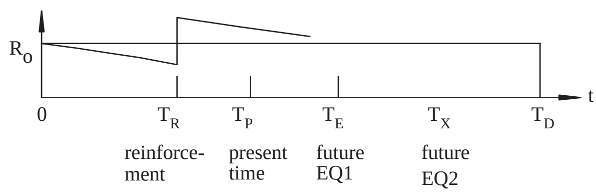

Figure 2. Profile of the residual strength during the life cycle.

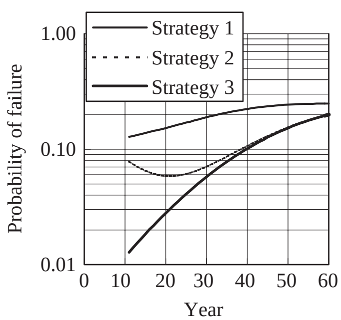

Figure 3. Probability of failure for various strategies.

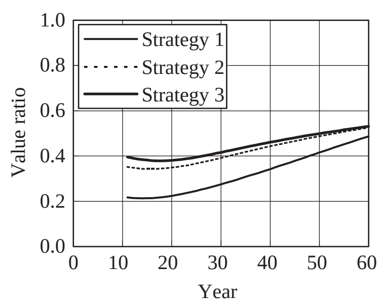

Figure 4. Net present value for various strategies.

prevention work, the present time, the uncertain time of *EQ2* occurrence which is supposed to be a Level 2 ground motion and the life cycle period of the structure, respectively. The structural strength R changes from the initial strength R_o to the residual strength under deteriorating process and inevitable hazardous process including seismic effects. The following three strategies are introduced to find the optimal restoration against more severe earthquake. Strategy 1 means that restoration is done to recover the present level, while strategy 2 aims to recover the initial level, and strategy 3 can provide the reinforcing level enough to resist against the Level 2 ground motion denoted by *EQ2*.

Numerical results for the three strategies are shown in Figs. 3 and 4 which show the probability of system failure and net present value when *EQ2* earthquake strikes the residual structure after the first earthquake *EQ1*. Numerical results suggest the restoration up to the highest grade (strategy 3) can provide the optimal solution among the three strategies.

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Small and medium size bridge maintenance sequence analysis by optimization technique

H.C. Hsu
Feng Chia University & Fong-Yi Construction Co. Ltd., Taichung, Taiwan

W.P. Chang
Feng Chia University, Taichung, Taiwan

R.D. Wang
River Basins Management Bureau, Pingtong, Taiwan

C.H. Cho & D.H. Jiang
Feng Chia University, Taichung, Taiwan

ABSTRACT: Bridge in traffic transportation system play an important role to solve the obstacle of topography and build up the elevated interchange. Due to the increasing of traffic flow, over weighted heavy vehicle and natural disaster from typhoon or earthquake bridge damage are getting more seriously threaten the transportation network. But the maintenance budget from the public authority is limited. How to build up the priority of maintenance sequence of the governed bridges to make the best cost benefit efficiency is one of the main problems for bridge authority. Local government usually lack of budget and expert but face to a lot of small & medium size bridge to keep the daily life and traffic going well between the small town and villages. This paper is intended to find a suitable weight value for priority analysis of maintenance the small & medium size bridge in a specified area.

At first, we use D.E. R&U Criterion to classify the bridge damage in Degree (D) and at what Extend (E). Then by considering the security and relevancy (R) the inspector will evaluate the Urgency (U) to make the sequence of maintenance for all the bridge in the considered region. This method is officially adapted by the Ministry of Transportation and Communication in Taiwan.

We have further developed the model by using Analytic Hierarchy Process (AHP) method through the interview with experts of more than 10 years experience in bridge maintenance, questionnaires to determine the appropriate weight of evaluation factors. Comparing the weight and priority orders with the Back-Propagation Neural Network model (BPN), AHP model and D.E. R&U we found BPN model as similar as the weight of evaluation factor of D.E. R&U method. The weight of evaluation factors with AHP model is higher than the other two models. We consider that because of the debris flow have frequently damage the bridge foundation thus the experts have giving more concentrated on the infrastructure of the bridge.

The case study is taken basically on the bridge inspection of Nanto County in Central Taiwan where most of the region is mountain. We have the case of totally 2230 bridges to analyze. But only 317 bridges inspection data are trained by BPN network.

To sum up, the effect of priority analysis that the BPN model is better than AHP and D.E. R&U models.

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Internet-based management of major bridges and tunnels using the Danbro+ system

J. Bjerrum
The Danish Road Directorate, Ministry of Transport, Denmark

F.M. Jensen
Ramboll, Copenhagen, Denmark

ABSTRACT: The Danish Road Directorate (DRD) has 30 years experience with systematic maintenance and management of 1450 bridges and tunnels on the Danish national road network, including 40 major bridges and traffic tunnels. The PC-based management system DANBRO+ has been successfully used to administer and manage the minor or small and medium-span bridges and tunnels in the last 15 years, but the system has shortcomings with respect to optimal management of major or long-span bridges and tunnels. This is due to the number of tasks on each bridge, the special structural elements and the comprehensive organization involved with the daily administration and management of e.g. large traffic tunnels, bascule bridges or suspension bridges. This experience with systematic maintenance and with DANBRO has been used in the development of DANBRO+, a holistic information and management system for the maintenance management of large bridges and other complex structures. The purpose is to support the handling of administrative data, structural and mechanical components, management tools, traffic management, inspections, repair/rehabilitation and special structure – relevant equipments/materials. The system has been specially designed to support all management levels of the bridge owner's organization as well as consultants, contractors/suppliers and others participating in the operation and maintenance of the structures.

The systematic maintenance and management of major bridges and traffic tunnels involves an extensive list of technical, traffic and administrative activities. Therefore, an IT management system must be able to include a number of different disciplines such as maintenance, organization management, surveillance, inspections, rehabilitation, traffic control, priority-ranking and budget and quality control. The purpose of the management system is to optimize and support the daily administration of the major bridges and tunnels including systematization and quality assurance of the management of the structures carried out by many engineers and technicians from DRD and consultants/contractors.

To support the handling and coordination of these activities DRD has developed the information and management system DANBRO+. DANBRO+ is an Internet-based client-server system, tailor-made for handling all activities such as routine, principal and special inspections together with all maintenance, rehabilitation and repair activities. Furthermore, the system includes a complete document-handling system with advanced search and navigation facilities.

More than 30 years' experience with systematic maintenance and management and more than 15 years of experience with the DOS and Windows-based DANBRO+ systems for maintenance and management of smaller bridges forms the background for the DANBRO+ system.

The decision to develop a tailor-made management system – and not to adapt an existing standard system – was based on an evaluation of several standard administration and management systems. The conclusion of this survey was that no standard system covers more than approximately 60% of DRD's needs and requirements.

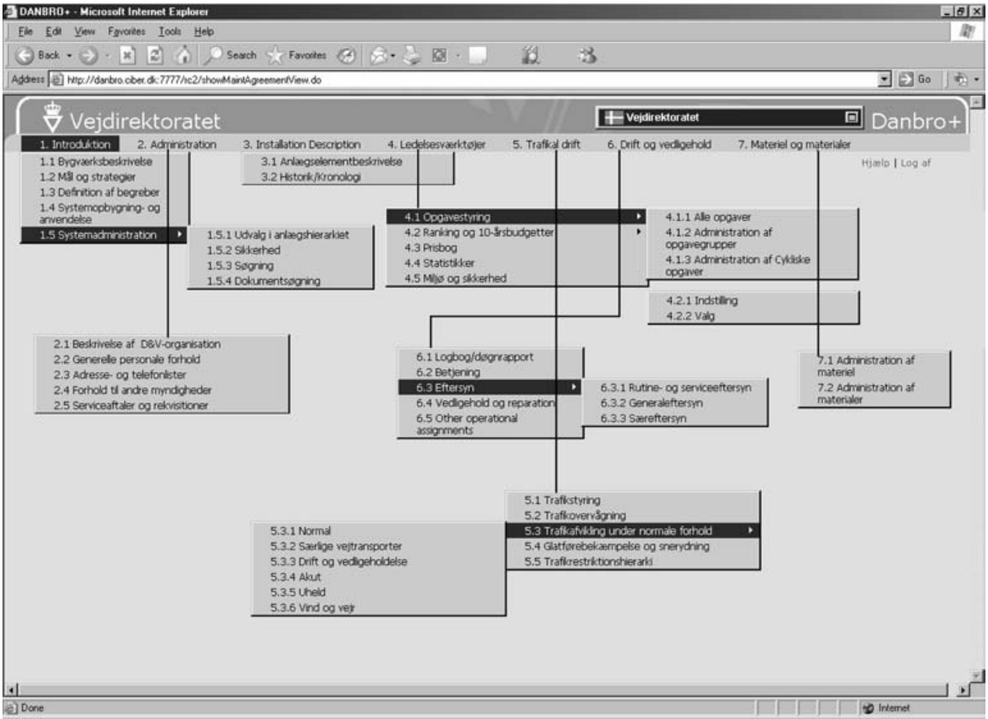

Figure 1. Overview of modules and sub-modules within DANBRO+.

The DANBRO+ system is divided into main and secondary modules that meet the daily requirements and support the needs for overview and easy access to information (data and documents). The modules implemented into DANBRO+ is shown in the figure above.

The DANBRO+ system was delivered by year-end 2005 and will be tested in the spring of 2006. During and after the test period bridge data and documents will be transferred from various older systems to DANBRO+ including the complete digitized archive consisting of complete documentation of all inspections, all repairs, all drawings and all management related documents. This alone consist of more than 20,000 documents and drawings for the 35 bridges and tunnels initially to be implemented in the DANBRO+ system.

The DANBRO+ system will be introduced to all users: administrators, consultants, contractors etc. in April 2006. At that the all tasks for 2006 have been created and bridge relevant data inserted into the DANBRO+ database. The users will therefore get a "kick-start" using DANBRO+. All tasks for year 2006 will therefore be managed within DANBRO+.

The DANBRO+ system will also be used as a political tool to achieve better understanding that yearly appropriations cannot be fixed since repairs on major bridges and tunnels are singular costly events which are executed with large intervals. DANBRO+ will be used to support the understanding of requirement-based appropriations as opposed to more or less fixed yearly appropriations.

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Use of genetic algorithms for optimal policies of M&R in a bridge network

F.A. Alonso & J.R. Casas
Civil Engineering School, Technical University of Catalonia, Barcelona, Spain

M. Nazar
Civil Engineering School, Autonomous University of Chiapas, Mexico

ABSTRACT: In available Bridge Management System (BMS) there are different models to optimize the funds that each Agency dedicates to those matters. The paper presents a model developed for the BMS of the Chiapas State in Mexico, using a joint optimization of the maintenance and rehabilitation policies. The specific transition probabilities of the Markov matrices are estimated for the particular conditions of the Chiapas bridge stock condition state and traffic loading and environmental loads in the area. The optimization problem is solved via a computer application using genetic algorithms (GA) to find the minimum costs for the different maintenance and rehabilitation policies generated by the model. It is shown how the application of the proposed model leads to better budget allocation and less total cost when compared with the standard method of maintenance used till now by the Chiapas State Agency.

In bridge engineering, computer simulations have proven to be an important tool for studying some aspects in bridge management. In the last years, several researches have used Genetic Algorithms (GAs) in optimal models for decisions-making process (Liu et al. 1997, Furuta et al. 2001, Malioka & Onoufriou 2002, Neves 2005). This method implement the idea of goal-oriented design was the application of a search and optimization technique borrowed from the field of artificial intelligence. Gas is based on Darwinian notions of survival of the fittest, where selection and recombination operators are used among candidate solutions to look for the optimal one. They have obtained optimal solutions for their proposed models using this optimization technique. In a bridge, the service life is limited by several factors. Deterioration in different components can be caused by problems during construction or in service stages. Problems such as corrosion, cracking, scours, concrete spalling, vegetation, settlement and others, may be a risk for users and produce a traffic interruption. Application of maintenance or repair activities can extend the service life. The question now is where, when and which bridge of the network to apply the specific and necessary maintenance activities? In the last years, many agencies have needed to implement a bridge management system (BMS) that assists them in an optimal decision-making process. Basically all BMS used by the different Transportation Agencies have a module for the election of maintenance and rehabilitation (MR) policies (BRIME 2003). Researchers in this area have developed theories and models to solve this problem and algorithms to provide solutions (Furuta et al. 2001, Tam & Stiemer 1996, Smilowitz & Madanat 1999, Morcous et al. 2003, Yanev et al. 2003). A model has is presented in this paper to optimize the costs of maintenance and rehabilitation policies of the Chiapas State in Mexico. The model uses the current condition of the bridges stock together with maintenance or rehabilitation policies in different periods of time. The evolution of the condition state of the bridge is taken into account by using transition matrices (Markovian model) and the optimal policy is obtained using Genetic Algorithms.

Chiapas has approximately 300 bridges in its state highway network. During 2004, an inspection campaign was carried out to know the current condition state of the bridges. Since 1980, no inspection campaign was carried out and so no regular funds for maintenance were available by the Agency during the past years. The decisions on maintenance and/or rehabilitation actions were made on a case by case basis, when the bridge was thought to become a danger for the users and traffic. The cost of the interventions was attended with extraordinary Federal or State funds only.

The model developed in this paper uses the main basis outlined in Guignier & Madanat (1999), where is proposed an approach for the joint optimization of maintenance and rehabilitation of the components of a network of infrastructure facilities under a Markov decision model. The present model adapts the joint optimization and develops a new and specific approach for the different conditions of the bridge network in the Chiapas State. The decision to develop a model based in a joint optimization was taken because others models were shown inadequate to the conditions of Chiapas bridge stock. These conditions are mainly: Absence of a regular periodic inspection campaign, absence of a monitoring technology, no plans for maintenance priorities and no ordinary budget for maintenance and/or rehabilitation actions.

For the joint optimization, first the model proposes maintenance or rehabilitation policies for each component in a random way. These policies may be "to act" or "do nothing" in a component, if the policy is "To Act" a maintenance or rehabilitation action will be applied to the structure according to its actual condition state. These policies determine the new condition state of the component for the next time period. If the random policy is to act, then the component will have a good condition state in the next time period, otherwise the new condition state of the component for the next time period will be obtained by multiplying the transition matrix by the condition state vector. There are two different costs for each bridge component. They are related to the decision to apply a maintenance/rehabilitation actions or do-nothing. Do-nothing cost is only the user cost.

To minimize the cost of the maintenance/repair actions in the network, a random policies series that will optimize the cost using genetic algorithms (GAs) are used. The objective function is presented in equation (1) and the minimization will be subject to different constraints:

$$MIN\left[\sum_{k=1}^{c}\sum_{j=1}^{p}\sum_{i=1}^{n} C_{k} * ECC_{ijk} * MPC_k\right] \tag{1}$$

By the nature of the objective function (equation 6) and by the complexity of the problem (because when the number of bridges increases, the number of random policies increases too), the model uses GAs to solve the problem outlined previously. There are not universal rules how to use a GAs to solve a problem. Its performance will usually depend on details such as the solution codification methods, the operators, the parameter values and the particular criteria to measure the algorithm success.

The model generated good results. The maintenance and repair policies obtained from the model provide an optimal cost when applied to one or various components of the each analyzed bridge that require maintenance or rehabilitation actions. In this work, GA was applied to find an optimal cost, the results obtained showed that GA is a good search algorithm and optimal technique, although it was applied to a small number of bridges, but the same procedure can be applied to the complete bridge network.

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Optimization of reinforced concrete bridges maintenance by Markov chains

André Orcesi & Christian Cremona
Laboratoire central des Ponts et Chaussées, Paris, France

In France the management and the rehabilitation of bridge stocks are concepts that were really taken into account in public policies only after the seventies. Indeed despite an institutionalization of French bridges maintenance in 1716 with the establishment of the "Ponts et Chaussées" corps, the bridge maintenance was progressively neglected after the Second World War to the advantage of building policies. The aim was then to build rather than to rehabilitate. But lacks of service levels due to the increase of traffic, the collapse of deteriorated bridges like these of the bridge of Elme in 1977, or these of the bridge of Tours in 1978 led to the awareness of the importance to assess bridge stocks. This is how was written the "Instruction Technique pour la Surveillance des Ouvrages d'Art" on the nineteenth of October in 1979 which was a rationalization of the maintenance actions and that is still used nowadays.

The management of bridge stocks associated to scarce capital resources led in the last years to the development of bridge management systems. Those models are based on life cycle costs analysis and try to optimize maintenance costs. Several management policies are generally compared to maintain the level of the bridge stock above a certain limit acceptable for the bridges manager. The knowledge of how the bridges deteriorate is essential in those decision processes. Some models like Pontis or Bridgit in the United States use probabilistic methods to simulate bridges ageing. Those models are particularly interesting because unlike deterministic methods they include all the possible cases of failure and make it possible to consider all the hazards that can occur in the life of a facility.

Such models do not exist in France for the moment. Decision-making tools only permit to have an idea by hand of the way the facility stock evolves with time. Consequently it is difficult to optimize maintenance. Moreover probabilistic models, which were used until today, consider the bridges only individually. For a bridge it is the ageing of its constitutive elements that is modelled. There is no real probabilistic model that considers the bridges as elements and predicts the evolution

Figure 1. IQOA inspection.

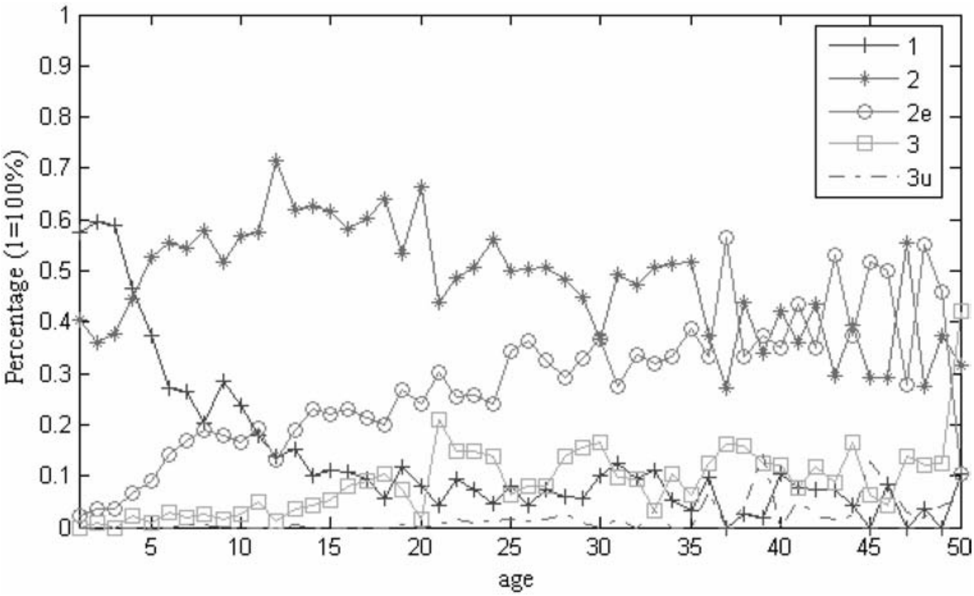

Figure 2. Repartition of reinforced concrete bridges per age and per state in 1998.

of the bridge stock. The IQOA programme (Quality picture of bridges) was developed to give an image each year of the global state of the bridge stock managed by the French Highway Agency (Fig. 1).

The access to the corresponding data for reinforced concrete bridges (Fig. 2) is an opportunity to apply a probabilistic model on a bridge stock in a global way and not on individual elements of each bridge.

A possible approach is to model the transition from a state to another with matrices by using Markov chains (Markov Process). The use of Markov chains on the IQOA database restricted to reinforced concrete bridges is studied in a first time then the transition matrices are calculated in several ways to determine the most realistic predictions of the bridge stock evolution. Then after having estimated the transition matrices, several management policies are compared. An objective of this study is to include maintenance costs to lead an economical analysis. Simulations of evolutions associated to an economic approach permit to determine optimal maintenance strategies, i.e. those that maintain the bridge stock in a good state and which control the repair costs. The final objective is to be able to appreciate the optimal allocation of resources for the management of a bridge stock.

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Bridge management system – GOA

T.P. Mendonça, A.R. Vieira, V.R. Brito & P.P. Paulo
Betar Consultores, Lisbon, Portugal

ABSTRACT: The Bridge Management System (GOA) has been changing throughout the years in order to fulfil the requirements of our different clients and the characteristics of their bridges. During the past years these changes were mainly related with the definition of the basic modules that enable the storage of different data (Inventory, Principal and Routine Inspection, Special Transportation, etc). As these first steps have been taken in the implementation, our attention is now focus on the implementation of priorization criteria that enables the client to choose the more beneficial investments

As the GOA system encloses a large number of data in its different modules (Inventory, Principal and Routine Inspection, etc), its very important to determine those that are essential to a priorization strategy.

1 INTRODUCTION

The bridge management system (GOA) has been implemented during the past years in several road and train owners, with the objective of centralizing the relevant information about their bridge set.

In order to set a perspective, the BMS GOA is actually implemented in the following clients , which have to account for the number of bridges shown it the table below:

Bridge owner	Bridge number
AEA	322
VLT	199
VEP	123
LUS	69
AEN	114
SCV	237
REFER	2200
EP	≈6000

As seen in the table, the client of the GOA BMS, are very different, both in scale and in characteristics, thus requiring different approaches regarding the implementation of a BMS.

2 BMS APPROACH

The implementation of the BMS GOA, done during the past years has mainly been focused in storing relevant data regarding the characteristics of each bridge (component by component) and also the inspection data.

This preliminary step is a very important one in the implementation of a full BMS system, as it enables the client and consultant to have a perspective about the influence of the environment and traffic in the degradation rate of the bridges. As the link between the various parameters involved in the degradation of each bridge is very complex, the evaluation period of the degradation rate has to be extent. In this evaluation is very important to cross check link between the pathologies detected with the involved parameters, as well as the consequences of their existence.

2.1 *Knowing the bridge set*

To set a perspective the following pathologies detected in inspections done during the past years of implementation of the GOA BMS, show a huge variety of defects. These are directly related to either the structural type, environmental location of the bridge, construction period, traffic loads, etc.

2.2 *Priorization approach*

As shown before the various defects result from a large set of parameters, which define the environmental and the loading of the bridge.

During the past years a great effort has been done throughout the world in order to achieve degradation models capable of quantifying each parameter. The evaluation of each parameter and even their interaction has been proven possible, but the reflection of each result in the all bridge is however more difficult.

In the mean time, while degradation models are being further analyzed, BMS systems have maintained course, working with more tangible parameters, even though they contain some subjective evaluation.

The priorization of investments in BMS such as GOA is made through the evaluation of both discontinuous and continuous parameters stored in the database or in linked databases. These parameters can be group in the following items:

- Structural safety
- Road Safety
- Serviceability and Functionality
- Cost benefit of investments
- Other factors

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Current maintenance management practice for highway bridges in Vietnam

Dinh Tuan Hai, Hitoshi Yamada and Hiroshi Katsuchi
Yokohama National University, Yokohama, Japan

ABSTRACT: The paper reports a research to investigate current maintenance management practice of highway bridges in Vietnam. Results show clear picture of poor maintenance management for existing bridges. Suggestion is stated at the end to improve the shortcomings.

1 INTRODUCTION

Due to external impacts, bridges in Vietnam now have poor condition and require substantial expenditures to overcome these deficiencies. A local research identifies corrosion, fatigue, aging, human invasions, missing elements, etc., as current defects. Meanwhile, research on bridge maintenance management of Vietnam is unavailable. The aim of this paper is to identify current system in Vietnam to find shortcomings and to suggest possible ways to improve the system.

2 GENERAL OVERVIEW ON HIGHWAY BRIDGES IN VIETNAM

General description of highway bridges in Vietnam is stated as different construction time and applied standards, multiple forms, under strong impacts of wars and adverse climates. There are differences from geographical conditions among bridges due to scattering through mountains, deltas and coasts; differences from northern and southern areas; and differences from temperatures of day and night, winter and summer. Increasing vehicles transported are moreover adversely impacting on bridge conditions. There is an assumption that bridges built before 1995 are impossible for current transported overloads, especially those were impacted by the war.

3 MANAGEMENT PRACTICE FOR HIGHWAY BRIDGES

3.1 *Structural organization of bridge owners and its agencies*

In Vietnam, bridge owner and its agencies are organized in top-down hierarchy. All bridges are owned by the government who provides finances for initiation, operation and maintenance. The ministry of transport is the forefront representative in the field and divided into specific sections such as policy formations, new construction and maintenance management. The road administration is intermediate agency to be in-charge of highways and bridges. Direct-agencies are the management and repair companies directly involving in site maintenance and management.

3.2 *Management of bridge inventory data*

The owner wishes to record this data in consecutive orders for subsequent uses. The practice is however far from requirement. In-deep surveys at direct-agencies confirm this data is manually recorded on paper-forms, kept in disorder storages and verbally communicate among relevant parties. Meanwhile, computerized database is not applied yet for managing existing bridges.

3.3 *Management of bridge physical sites*

The owner wants properly managing bridge sites to prevent human invasions, ensure smooth and convenient traffic, preserve bridge quality, and reduce accidents. Several activities are (a) management of safe corridors, (b) management of safe facilities and (c) bridge guards.

3.4 *Bridge inspections and evaluations:* as shown in figure 1

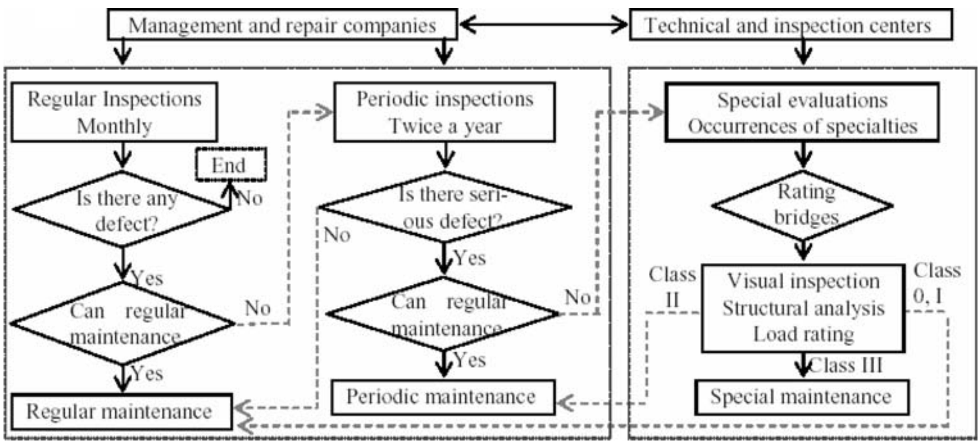

Figure 1. Procedure of regular and periodic inspections and special evaluation.

4 MANAGEMENT PRACTICE FOR HIGHWAY BRIDGES

4.1 *Management of maintenance data*

It is to manage all maintenance data of bridges. This data should be carefully store to preserve them for subsequent uses. The practice is far from requirement as direct-agencies do not systematically manage, but scatter and manually keep data in disorder forms. It moreover confirms that there is no computerized database available in Vietnam to store data for future uses.

4.2 *Site maintenance:* as shown in figure 2

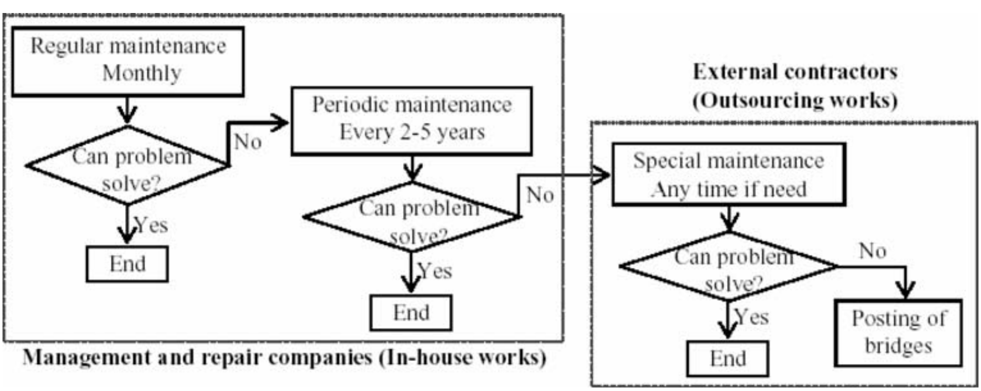

Figure 2. Three categories of site maintenance for highway bridges in Vietnam.

5 CONCLUSION

Existing bridges are in poor physical condition and functionality. In management aspect, good management system that encompasses appropriate computerized database is not yet available in Vietnam. For site management, shortage of the fund cause delays and cancellation in site works. Missing and disorder of inventory data additionally cause dilemmas for field practitioners. The paper proposes firstly setting up an appropriate maintenance management system with good computerized database to store all inventory data and to manage bridges in best manners. Annual fund should be increased from currently 30–50% to at least 50–70% of actual requirement.

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Cruz, Frangopol & Neves (eds)*

Proposal of maintenance management system for existing bridges

D.T. Hai, H. Yamada, H. Katsuchi & E. Sasaki
Yokohama National University, Yokohama, Japan

ABSTRACT: The paper proposes a model to include three interlinked management, assessment and maintenance for bridges. A computerized database is additional suggested in this model.

1 INTRODUCTION

Emphasis has recently placed on the new constructions, but not for maintenance management through bridge lifespan. Inappropriate maintenance causes bridges to be degraded by lack of durability and serviceability. Present practice moreover highlights on site-works, but not for management and assessment. Thus, the research is initiated with objective to propose an appropriate bridge maintenance management system. It is to balance between limited resource and high maintenance demands so bridges can be maintained at reasonable quality and functionality.

2 BACKGROUND

Effective bridge maintenance management has been implementing. Chase and Gaspar (2000) mentioned the use of the Pontis and the BRIDGIT in U.S. The J-BMS is proposed for Japan (Kawamura et al., 2001). In Vietnam, the BridgeMan is under considering (Dac et al., 2004).

3 PROPOSAL OF BRIDGE MAINTENANCE MANAGEMENT MODEL

Paper proposes model consisting of management, assessment and maintenance (figure 1).

3.1 *The management module*

The bridge management consists of regular and repeated activities with direct involvement of in-house staffs. It is divided into five subgroups (figure 2). The policy may vary due to external factors and should be established to guide relevant people in accomplishing target.

Bridge condition: the quality should be inspected by various methods. Inspection and load rating are compulsory while structural analysis and nondestructive test are optionally proposed.

Computerized database: is proposed to systematically store and update bridge data. The proposed database includes three sub-modules of asset management, assessment and maintenance.

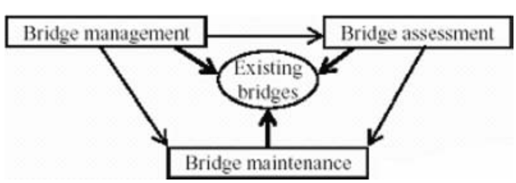

Figure 1. Proposed model of maintenance management.

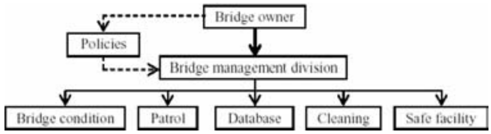

Figure 2. Bridge management.

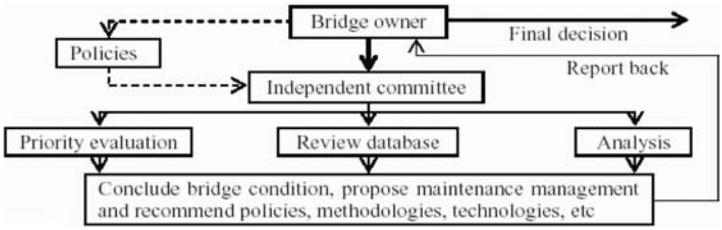

Figure 3. Bridge analysis.

Figure 4. Procedure of bridge maintenance.

3.2 *The assessment module*

The bridge assessment utilizes maintenance management data to assess bridge condition, to propose suitable activities and to recommend suitable method, technology, etc (figure 3). Condition rating: classifies in degree 0: no defect; 1: minor defects, not weaken on quality; II: moderate defects, impact on quality; III: serious defects, adverse impact on quality. Priority index (BP): maintenance priority is based on the value of the bridge important and the bridge health. Bridges with highest BP are in priority for maintenance as soon as possible.

3.3 *The maintenance module*

The result gained from the management and the assessment is used for efficient maintenance (figure 4). The policy may vary to reflect many requirements as safety must be emphasized. Maintenance assessment: if budget is low, only prioritized bridges and elements are maintained, basing on priority index, budget, LCC, cost-benefit analysis, degradation prediction, etc. Site maintenance: should outsource for external contractors to ensure better work quality at reasonable prices. It consists of preventive, progress and emergency maintenance.

4 CONCLUSION

The paper proposes a maintenance management system for existing bridges. It should include three interlinked modules of management, assessment and maintenance. Management module is to control all bridge inventory data since initiations until postings. The module of assessment can be used to analyze quality condition, life cycle cost, cost-benefit, priority maintenance index. The maintenance module that gets results from two other modules for conducting efficient site works. A proposed database system is introduced in this paper to facilitate in-house works. Further study is planned to introduce this model in actual maintenance management practice.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Toward maintenance of old stone bridges in Korea

N.K. Hong & H.-M. Koh
Korea Bridge Design & Engineering Research Center, Seoul, Korea

S.G. Hong
Department of Architecture, Seoul National University, Seoul, Korea

B.S. Bae
National Research Institute of Cultural Heritage, Daejeon, Korea

ABSTRACT: Most stone arch bridges in Korea are older structures and are becoming short of safety. This lack in safety is a result of natural deterioration, of the materials used in construction, and of earlier design consideration that no longer accommodate the speed, dimensions, loads, and volume of modern traffic demands. However, the older bridges must be evaluated for historic significance in addition to safety to advocate for the preservation of our cultural and engineering landmarks-monuments to the people and communities. Therefore, engineers must balance preservation principles demanding authenticity of materials and visual characteristics with code requiring safety, strength and stability and historic significance for the maintenance of historic bridges. In order to enable bridge owners to weigh many factors in deciding to maintain, repair, or replace, a method of assessing the physical condition of the bridge and its historic significance. This paper focuses on issues of how to assess historic significance by considering the information about our cultural heritage including architectural uniqueness, innovations in engineering, and evolution of the transportation system.

1 INTRODUCTION

Recently, the importance of cultural heritage has been greatly raised and concerns both about the conservation and maintenance are growing among preservationists in recognizing and protecting the cultural values worldwide. There are several cases which affect the degradation of cultural significance of cultural heritage including the following cases:

Case 1: Urban development without a comprehensive consideration for cultural heritage, which moves or buries it into an unrelated site.

Case 2: Restoration of cultural heritage without systematic maintenance management plan, which replicates the outer appearance only.

Case 3: Rehabilitation considering only few aspects such as structural safety impending at hand, which causes the replacement of the original material with a new one.

In order to address the above issues, a review of the conservation and general maintenance management literature including the renewal project of Korea was conducted. Through the examination, it is known that a planned and comprehensive approach to the maintenance programming for old bridges is essential by addressing both safety and preservation issues involved in the restoration of historic bridges in a balanced way. This study mainly focuses on the development of maintenance process and tasks before the bridge material gets to the stage where repairs are needed, thus preserving historic material as possible. Moreover, the maintenance plan for structural safety will be balanced with the degree of cultural significance.

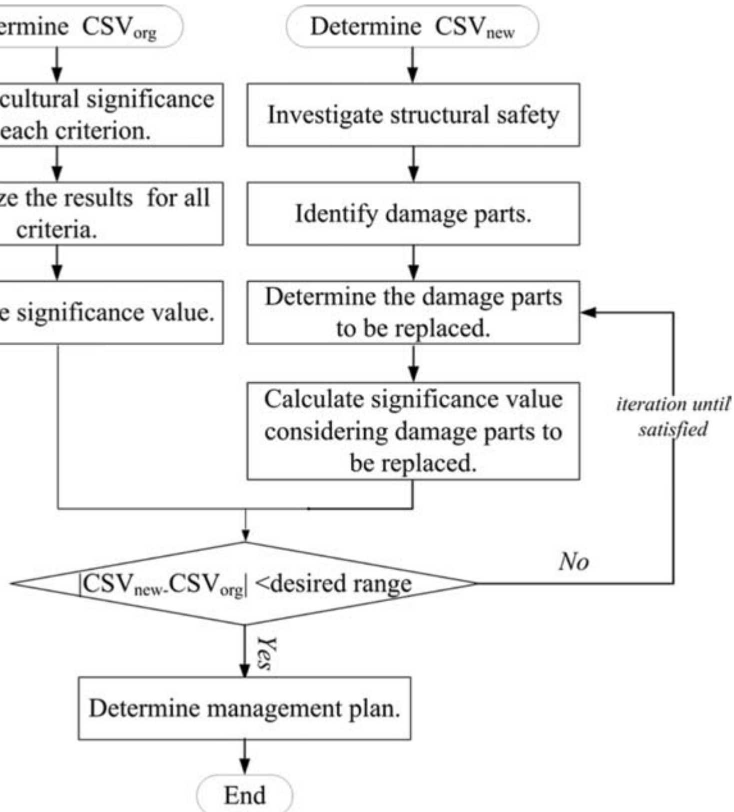

Figure 1. Maintenance programming procedure for the conservation of cultural heritage.

2 MAINTENANCE PROGRAMMING PROCEDURE

The primary aim of preservation and conservation is to maintain the historical authenticity and integrity of the cultural heritage. Each intervention should therefore be based on proper studies and assessments. Problems should be solved according to relevant conditions and needs with due respect for the aesthetic and historical values, and the physical integrity of the historic structure or site. In this relation, maintenance programming is the most important action to protect cultural significance because, if properly taken, it is the least destructive of all the 'interventions' which inevitably occur in the process of conserving historic bridges.

Figure 1 shows an overview of maintenance programming procedure proposed in this study as a planned approach. The procedure consists of two distinct sub-processes: (1) assessment of cultural significance value (CSV_{org}) of cultural heritage before modification and (2) assessment of cultural significance value (CSV_{new}) of cultural heritage after required modification.

One of the important features of the proposed approach is to use cultural significance value as a strong constraint in decision-making process of management action. Generally old heritage bridges are exposed to experience great threat which is associated with time passing or destruction of cultural site. For example, when an aged heritage bridge is becoming damaged and needs of retrofit for structural safety, the bridge must be repaired to a certain degree and thereby this repair affects the cultural significance value. Then, a management action must be cautiously planned not to change the cultural significance value estimated before the repair. This may be done by simulating various repair methods to find out the best solution which enables to keep the cultural significance value at a desired level. Thus degrees of 'repair' would be determined depending on the cultural significance value within the maintenance plan.

3 CONCLUDING REMARKS

The age of a bridge is not a major reason for decay of old heritage bridges. More systematic care with an appropriate maintenance programming is both fundamental and effective to better conservation. Especially, a rational decision-making for determining management programming is of importance. In this relation, this study proposed the comprehensive maintenance programming approach for old bridges which addresses both the structural safety and preservation issues within an integrated framework. This paper presents an example of the on-going study with stone-slab bridges. The proposed maintenance programming approach could also be applied to other bridge types such as stone-arch bridges while the detailed information which may vary widely among different bridge types is being developed.

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An outline of the APT bridge management software

D. Zonta, R. Zandonini & F. Bortot
Department of Mechanical and Structural Engineering, University of Trento, Italy

R. De Col & P. Nicolussi Paolaz
Department of Transportation, Autonomous Province of Trento, Italy

ABSTRACT: This paper presents the main features of the software for bridge management developed by Trento University for Autonomous Province Trento (APT). The system operates entirely on the web, and includes modules for (i) condition state (CS) evaluation, (ii) safety assessment, and (iii) prioritization. CS is evaluated on the basis of a procedure that acknowledges the general rules of the AASHTO Commonly Recognized (CoRe) Standard Element System, in order to conserve compatibility with PONTIS evaluation and deterioration models. Elements are characterized by up to five discrete CS, which describe the type and severity of deterioration mostly in visual terms. Normally, the system conservatively estimated a prior reliability index β for each bridge on the basis of the inspection data. Where the condition of the bridge is cause for concern, the system foresees a formal safety evaluation based on a 5-step procedure that acknowledges the general guidelines resulting from BRIME project.

The system operates with a number of separate servers which are conceived to be hosted on different machines and can be managed by different providers. In its initial configuration the system includes: (i) a database server, (ii) a web server application and (iii) the data analysis application server. Most of the data information is stored in an SQL database. The web application serves as the main interface to system users, providing access to data management through a user-friendly portal. Users include the system managers, who have full control over the stock information, inspectors and evaluators, whose access is limited to bridges and inspections under their responsibility, and guests. Network-level tools are implemented in the form of an application server. The web application acts as a client respect to the analysis server, and all data is exchanged through the database. The main advantage of this software architecture is that it allows flexible system development towards a multi-agency scheme.

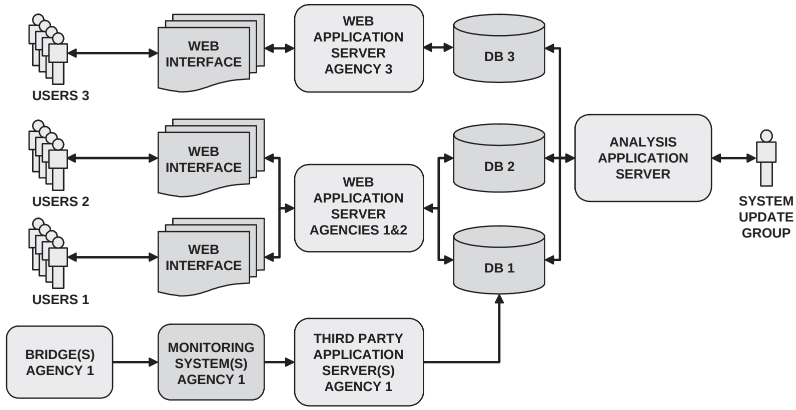

Figure 1. Architecture of the PAT-BMS in a multi-agency configuration, including link to monitoring systems.

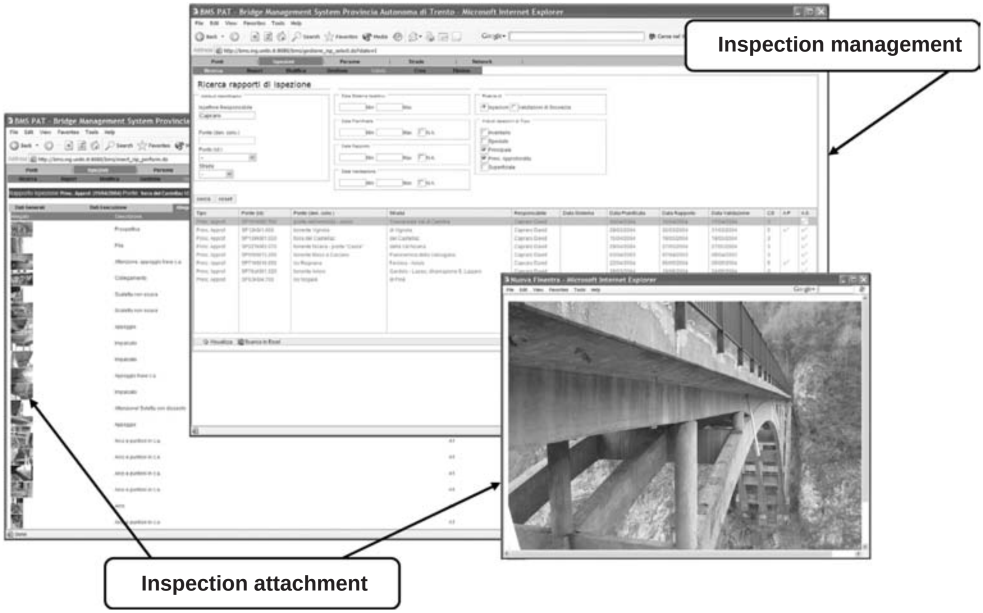

Figure 2. Display of the APT-BMS web-interface.

The actors involved in system operation send queries to and modify the database through a web-based user-friendly interface. Using the main menu, the user browses the various sections of the site, including: *Inventory*, *Inspections*, *People*, *Roads* and *Network*. From the same web application, inspectors and evaluators download the appropriate procedures, and upload the data resulting from condition assessments or from safety evaluations, including attachments, if any, such as pictures, documents, FEM models, AutoCad files. Appropriate restrictions to the system are given so that each user can access only the information necessary for carrying out his or her specific task.

In principle, the system can also automatically receive data streams from a permanent monitoring system, although this option has not been fully exploited by APT so far. The choice of substituting in the near future all or part of the manned work with automatic processes depends on issues that are technological (the actual availability of a technology, in the broad sense, capable of reproducing manual labor) and economic (the cost of instrumentation and its operation with respect to the cost of inspection). Network-level analysis is performed in real-time by a stand-alone application, currently hosted and maintained at the University of Trento. As this tool is also Internet-based, the manager can access the result of the analysis on a real-time basis through the same web-interface used for browsing the database. Although access to the official APT-BMS is obviously password protected, a mirror of the web application and of the database, including all of the bridge data inventoried up to December 2005, is maintained on a separate server for research and dissemination purposes. The reader can appreciate the operation of the software by accessing this mirror through the website http://bms.ing.unitn.it:8080/bms, using *guest* as username and *dims425* as password.

The system is continuously updated by an operative group at Trento University. Updating includes: procedure detailing, refinement of network-level models, monitoring and evaluation of the work of inspectors and evaluators, software usability optimization and debugging. All these operations are remotely performed and are totally transparent to the users of the system.

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Development of a reconstruction strategy for the Angolan bridge network

M. Alves
ISQ, Lisbon, Portugal

J. Sebastião
INEA, Luanda, Angola

ABSTRACT: The development of a reconstruction strategy for the Angolan bridge network is presented. The paper begins with an evaluation of bridge management situation in Angola and the characterization of the bridge stock in order to access the most urgent needs. It is followed by the description of the use of some of the Pontis® tools to set up reconstruction programs.

1 INTRODUCTION

The Angolan Road Network was constructed from the late 50's to mid 70's. In 1974 it assured the connection to almost all Angolan territory in good conditions. Nowadays the road network is in a very poor condition as consequence of the civil war (1975–2002) and the lack of maintenance in the past 30 years. The Angolan Government has established as a top priority the reconstruction of the road Network, in order to assure the safe circulation of people and goods.

The Pontis® bridge management system will be adapted in a first stage as a reconstruction management tool in order to develop a straightforward strategy that can be applied on a short-term. Most of the work done in the first stage can be used as a basis for a bridge management system in the future.

2 INVENTORY AND INSPECTION DATA

Until the beginning of the implementation of the bridge management system the situation regarding inventory and management of the nation's bridges was the following: 1. Non existent inventory; 2. Non existent or miss-placed construction documents and as-built drawings; 3. No one responsible for the maintaining bridges in-service (reconstruction of bridges after they collapse; 4. Lack of qualified technicians to perform bridge inspections and to elaborate bridge reconstruction plans.

2.1 *Inspections' program*

Reliable and up to date data are a key factor in any decision making process. In that sense, a special training program was developed to enable INEA's technicians in performing bridges inspections assuring: 1. Standardization on data collection and condition classification; 2. Evaluation of collapse risks.The first part of the inspections' program consisted on the inspection of all bridges in three different corridors (considered crucial to the country's development):The inspection procedures follow the FHWA Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges (Coding Guide), so the data collected is suited to respond to the Pontis® bridge management system needs. At this stage a total of 92 bridges and over 250 culverts were inspected.

2.2 *Characterization of the bridge stock*

As stated before, most of the bridges were built from 1955 to 1974. This means that many structural solutions, materials and construction methods are similar, which represents a great advantage in implementing a systematic approach for establishing a reconstruction strategy.

Almost all bridges cross rivers or small waterways. Over 65% of the bridges have a total length below 50 meters and only 10% have a total length over 100 meters. The superstructure consists in almost all cases of reinforced concrete simple spans. The infrastructures' material is reinforced concrete or masonry in the older bridges. As a consequence of the 30 year civil war many bridges were blown up. A small part was reconstructed; the remaining was replaced by portable temporary bridges, which assured circulation. Some of these temporary bridges do not comply with any sort of bridge standards and aren't fitted for the usual traffic loads.

3 DEVELOPMENT OF A RECONSTRUCTION STRATEGY

At this first stage the use of Pontis® is limited to the inventory and inspection data collection module and the project planning module.

Pontis® inspection module has a feature that allows inspectors to introduce work candidates Inspector work candidates may be entered as part of the inspection data entry process. These work candidates are used in the Project Planning module. Their impacts on bridge and element conditions can be simulated, and they can be used to create projects. All information entered about a work candidate is carried over if it is selected to become part of a project.

In order to establish a projects plan it's important to define priorities. The priority is set according to the gravity of the bridges' anomalies.

After entering all inspector work candidates and the respective priority rankings and estimated costs the Project Planning module is used to help to develop actual projects. The Project Planning module provides a flexible set of tools for planning and scheduling project work.

The policy established by INEA for defining and scheduling project is the following:

1. All bridges that have high priority works assigned should be scheduled for reconstruction first.
2. If the sum of all high and medium priority works to be done on a bridge is 40% higher than replacing the bridge, then the bridge should be replaced.

4 CONCLUSIONS

The strategy developed is very simple and straightforward and is based on quality of inspections and engineering judgment. The extensive use of Pontis® optimization modules is dependent on the capability of collecting cost and deterioration data and the stabilization of Angola, political and economically in order to allow a reliable long-term simulation.

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Dynamic programming for optimal bridge maintenance planning

Ming Liu & Dan M. Frangopol
Department of Civil, Environmental and Architectural Engineering, University of Colorado, Boulder, CO, USA

ABSTRACT: Bridge management systems currently face the great challenge to balance the limited available funds and increasing needs for bridge maintenance, repair and rehabilitation activities. This provides an excellent opportunity for applying advanced mathematical programming techniques in this field. This paper presents a dynamic programming (DP) procedure for identifying optimal bridge maintenance plans that are associated with the minimum total maintenance costs and satisfy both bridge safety and condition requirements for a targeted lifetime period. Since the effective service lifetime of a bridge without maintenance may not be long enough to reach a target level, it can be extended by applying sequential maintenance actions. The effect of any maintenance action on bridge safety and condition profiles can be classified as (a) improvement of current safety and/or condition indices, (b) delay in deterioration occurrence, (c) reduction of deterioration rates and (d) combinations of the three previously mentioned effects. Any combination of the maintenance action candidates, which can extend the bridge effective service lifetime to the targeted level, can be regarded as a feasible bridge maintenance plan. These feasible maintenance plans may

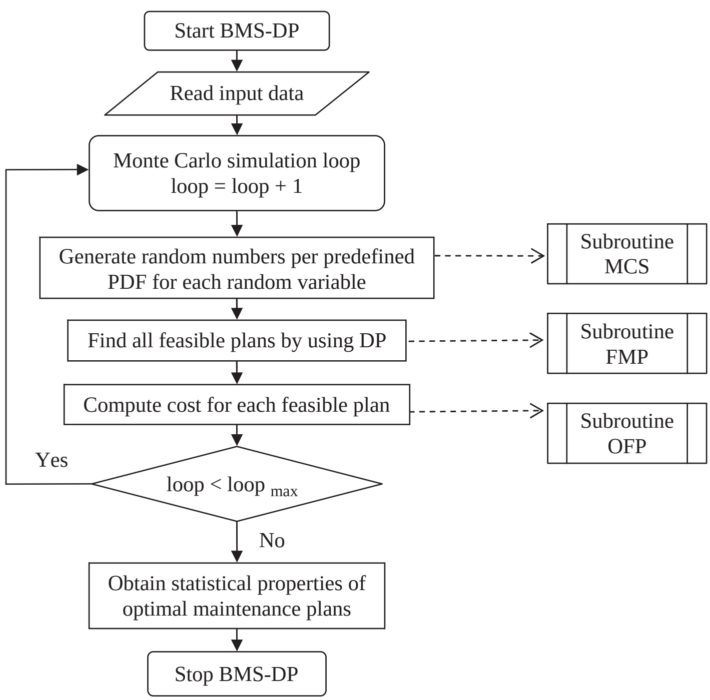

Figure 1. Flow chart of the computer program BMS-DP with its three subroutines.

require performing different combinations of the maintenance action candidates at different application times, resulting in different life-cycle maintenance costs. In order to find an optimal plan, the life-cycle maintenance cost for each feasible maintenance plan must be converted to the net present value (NPV), using a reasonable discount rate. An optimal maintenance plan in this study is the feasible plan that has a minimum life-cycle maintenance cost in terms of NPV. Theoretically, an optimal bridge maintenance plan can be obtained by comparing the life-cycle maintenance cost for all feasible plans, using the enumeration methods. However, the advanced mathematical optimization techniques such as DP and genetic algorithms (GAs) can provide more efficient approaches particularly for stochastic systems that involve random variables in computations.

In this paper, the effects of four different maintenance actions (*i.e.* minor concrete repair, silane treatment, cathodic protection and rebuild) on bridge safety and condition over time are reported. These four maintenance types include both preventive and essential maintenance actions with actual cost data. Then, all possible feasible maintenance plans for the targeted lifetime period are identified by a DP procedure, based on the allowable safety and condition requirements. The total maintenance cost of each feasible maintenance plan is converted to the net present value (NPV) using a discount rate ranging from 2% to 8%. Finally, Monte Carlo simulations are integrated with the proposed DP procedure for sensitivity studies, considering the probability distributions of all random variables and parameters in the proposed procedure. For this purpose, the FORTRAN computer program BMS-DP has been developed at the University of Colorado at Boulder. The flow chart of BMS-DP is presented in Figure 1. A numerical example is included for demonstration in this paper. As a result, the probabilities that each of the four maintenance actions may be applied at a certain time (year) can be obtained. This provides a solid base for bridge annual maintenance budget predictions, particularly for bridge network maintenance management.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Optimal long-term single stage intervention strategies for road bridges

B.T. Adey & R. Hajdin
Infrastructure Management Consultants, Zürich, Switzerland

E. Brühwiler
Division of Maintenance and Safety, Swiss Federal Institute of Technology (EPFL), Lausanne, Switzerland

1 INTRODUCTION

Road network managers are currently being faced with the task of approximating mid to long term financial needs for their infrastructure, as well as short term financial needs. This task is particularly complicated for bridges due to the lack of relevant information. Due to this lack of information and the need for relative immediate prediction of long-term financial needs it is often necessary to make simplified approximations of costs. The method discussed in this article, that can be used to make simplified approximations, is based on the condition of the entire structure, rather than its individual elements. This is done to circumnavigate the lack of data on the element level which will persist for many highway agencies over the upcoming years. It should in no way be understood as a preferred method over approximations based on element level data.

2 METHODOLOGY

The methodology used is to first estimate the cost of one time interventions on each type of bridge to be investigated and then to determine the cost of single stage intervention strategies for each of these interventions.

3 BASE COSTS

Since bridge interventions occur due to the deterioration of the bridge, the deterioration processes and their speed play an important role in determining intervention strategies and therefore costs. As deterioration processes vary based on the construction materials used, the primary division of bridges in the estimation of intervention costs is based on construction material. In this research work bridges were divided into four types, 1) metal bridges, 2) masonry bridges, 3) concrete bridges, and 4) composite bridges.

The intervention costs are approximated based on expert opinion per bridge type, and include only the rehabilitation and replacement costs of the agency. The intervention costs are approximated based on newly built structures, without construction defaults. Experts were asked the cost to repair the bridges in terms or their total costs when each of the bridges were in one of three general condition states, to restore them to a "like new" condition. The three general condition states for each of the bridges are associated with key times in the life of the bridges. The "like new" condition was defined to be one where the time to the next intervention and the cost of the next intervention would be approximately the same as for a new structure. Experts were asked to predict likely time periods within which they expected each new bridge to fall into each of these condition states, if no interventions were performed.

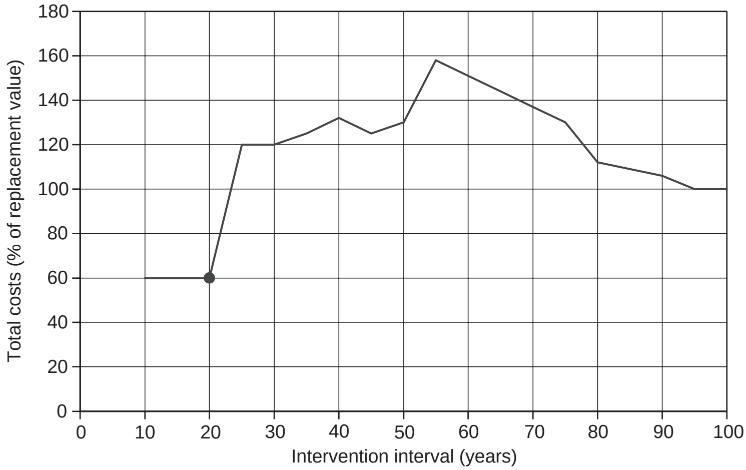

Figure 1. Costs of intervention strategies: Metal bridges.

4 INTERVENTION STRATEGIES

The intervention strategies used in the approximation of mid-to-long term costs were assumed, for simplification, to be single stage intervention strategies. The time intervals between interventions for these single stage intervention strategies were assumed to be the average time intervals and are given in five year intervals. In order to ensure that each intervention strategy is compared equally it is assumed that an intervention is performed in the last year of each investigated time period to restore the bridge to a "like new" condition.

As an example, the costs of the intervention strategies for metal bridges are shown in Figure 1. The costs are given in terms of bridge value if an intervention is done at every x years. For example, a 20 year single stage intervention strategy, when the intervention restores the bridge to a "like new" condition, is expected to cost 60% of the bridge value over a 100 year period. If the interventions are done every 40 years it is expected to cost 130% of the bridge value. No discount rate was used in the approximations of intervention cost.

5 CONCLUSION

It is found that it is optimal for each type of bridge (metal, masonry, composite, and concrete bridges) to ensure that it is continually protected against their respective deterioration processes. The interventions should be performed at the last possible moment prior to the onset of the general deterioration of the entire structure. Such single stage intervention strategies can save the infrastructure owner, when compared to an intervention strategy where the bridge is replaced in the first year that replacement becomes more expensive than bridge rehabilitation:

– 100% of the value of metal bridges over a 100 year period
– 80% of the value of masonry bridges over a 200 year period
– 70% of the value of composite bridges over a 100 year period
– 85% of the value of concrete bridges over a 100 year period.

There is little difference in the costs of intervention strategies that allow the general deterioration for metal and masonry bridges. This means that if it is optimal to allow the general deterioration of a bridge, and there are user costs associated with bridge intervention, that it will be better to have longer intervals between interventions rather than shorter. This trend is not observed for composite bridges or concrete bridges where even if the intervention strategies allow a general deterioration of a structure it is still better to intervene sooner rather than later.

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Optimization of preventative maintenance strategies for bridges

E.A. Tantele, T. Onoufriou & M. Mulheron
School of Engineering C5, University of Surrey, Guildford, Surrey, UK

ABSTRACT: The use of preventative maintenance (PM) has been acknowledged as an effective way of reducing the whole life cost of maintaining bridges in good, safe and functional condition. However, there are many uncertainties associated with the need and effectiveness of PM. Such uncertainties if not addressed can lead to incorrect decisions and wastage of resources. In addition optimization procedures are necessary to produce strategies with minimum cost while maintaining the reliability of the bridge at an acceptable level. This paper examines the issue of effectiveness of different PM on reinforced concrete bridges deteriorating due to the corrosion of reinforcement from chloride ions. It investigates the applicability of an optimization methodology, using the principles of genetic algorithm (GA) to identify optimum PM strategies based on their effectiveness and cost.

1 METHODOLOGY

The effectiveness of PM on reinforced concrete elements can be estimated based on the ability of PM measures to keep the reinforcement bar free from chlorides ions (Cl^-). PM is carried out usually at specified time intervals and can be either proactive or reactive. Proactive PM (waterproofing, surface treatment, etc.) involves carrying out works before a problem arises, while reactive PM (concrete replacement, cathodic protection, chloride extraction, etc.) is undertaken after a deterioration cause (e.g. initiation of corrosion) is observed.

To enable the prediction of the effectiveness of different PM actions a procedure using probabilistic techniques was developed (Tantele et al., 2005a). Uncertainties that influence the PM degree of effectiveness such as the amount of diffusivity of chloride in the concrete treated with PM are incorporated. The outcome of the analysis is expressed as a probability of failure (p_f) and depends on the proposed limit state. The limit state (margin) is set to satisfy the condition, that the chloride concentration at the surface of the reinforcement does not exceed the threshold value i.e. there is no corrosion initiation.

Furthermore, an optimization methodology was developed by the authors for identifying optimum PM strategies, using the principles of genetic algorithm. The GA method was invented by John Holland. Other researchers that used this approach for maintenance optimization are Furuta et al. (1997), Liu & Frangopol (2004), etc.

In the GA based methodology developed in this paper, the effectiveness of various PM measures is linked with their cost. The objective is to identify optimum PM strategies that have minimum possible cost while the p_f of the bridge element to which the actions are applied to, is kept below a target p_f; where p_f represents the probability of corrosion initiation on the top level of the reinforcement.

2 NUMERICAL CASE STUDY

A computer program using Excel XP and Visual Basic 6 was developed for the implementation of the probabilistic PM effectiveness modeling and the GA optimization methodology. In the case study presented in this paper, a range of PM measures are considered as options to form the optimum PM

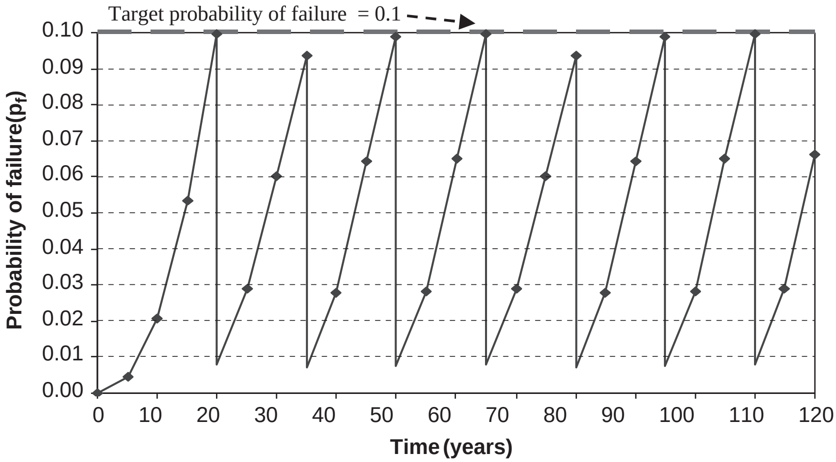

Figure 1. Probability of failure profile after application of proposed strategy.

strategy. The proactive measures examined here are silane, sealer and P-m coating and the reactive are cathodic protection, electrochemical chloride extraction and concrete replacement. The element performance when a recommended optimum PM plan is proposed can be seen in Figure 1 which shows the p_f profile with time. A combination of proactive PM measures is selected for a period of time until p_f reaches a critical value at which point a reactive PM is applied. This cycle is repeated several times during the lifetime of the element.

3 CONCLUSION

To obtain optimum or near optimum PM strategies a proposed GA methodology was developed to link the effectiveness of PM measures with their cost and it was successfully implemented for the case study presented here. The methodology developed here is a valuable tool for engineers, enabling them to gain better understanding of the relative effectiveness of PM and applying them more successfully in maintaining the safety and performance of bridges at acceptable level.

ACKNOWLEDGEMENT

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Service life design in concrete bridges

T.P. Mendonça
Betar Consultores, Lisbon, Portugal

ABSTRACT: The total cost of a Bridge is the sum of the construction cost and the maintenance and repair costs during service life. Whole life cost analysis is a simple and most common way to deal with this problem. It is normal that the Net Present Value of a Bridge can be 2 to 3 times the construction cost. This means that the most economical solution for a bridge may not be the one with the lowest construction cost.

In order to reduce the huge maintenance and repair costs that are nowadays spent in most European countries, it is mandatory to develop design recommendations in order to reduce future costs, improve durability and serviceability.

1 INTRODUCTION

In some countries the economic growth of the past years, with large investments on infrastructures, has centred the attention of bridge specialists on the construction phase, very often leaving out maintenance and its teachings.

In other countries, large sums of money are spent in repair and maintenance of bridges. In bridge tenders for construction, bridge owners in Europe only take into account the construction cost and not the whole life cost (total cost), construction cost plus maintenance costs. The most economical solution for a bridge maybe not the one with the lowest construction cost.

Many of the problems that bridges have during their life could be minimized in design phase. It's mandatory to develop design recommendations to reduce future cost and improve durability and serviceability.

In this paper we list the main recommendations for bridge design in order to attain these goals.

2 MAINTENANCE ENGINEERS ON DESIGN PHASE

Small details in design or construction phase can lead to big problems and high investments during service life of a bridge maintenance engineers deal with these problems everyday. A way to reduce problems is to involve them in design phase. It should be mandatory that all design should be check by maintenance engineers.

3 INTEGRAL BRIDGES

The use of Integral Bridges (bridges without joints or bearings), can reduce the maintenance cost of bridges, this type of structural solution can easily be adopted for bridges of total length up to 70 ∼ 80 m.

To compare the total cost of an Integral Bridge with a Non–Integral solution, we used an Integral Over Pass built in the 1989 and compared with a non Integral alternative.With the construction cost and all the maintenance costs the Net Present Value (NPV) methodology as used to compare the Total Cost of both solutions.

From the example analysed the following conclusions can be drawn:
The **NPV** is from **1.98 to 2.81 times** the construction cost depending on the discount rate.
The **construction cost** of the non integral solution is **10% higher** than the integral solution.
The NPV of the non integral solution is **13% to 15% higher** than the integral solution.
It's not the same to compare the construction cost or the total cost (in this case NPV) of a bridge.

4 INSPECTIONABILITY AND ACCESSIBILITY

During their life, bridges need to be regularly inspected and maintained in order to have a good performance. In the design phase it should be ensured easy access to all parts of the structure.

5 BRIDGE EQUIPMENT

Expansions joints, bearings, safety barriers, guard-rails, are normally responsible for the majority of maintenance costs.

5.1 *Expansion joints*

In order to have better joints some recommendations must be taken in design and installation such as Good detailing, correct management of water in the bridge deck, installation made by specialized works, etc. The detailed design of an expansion joint should include the maintenance plan, expected life time and warranty period.

5.2 *Bearings*

Immediately after the expansion joints, the bearings are the components that spend more financial resources during bridge life. They need a preventive maintenance to prevent possible changes in their function. The design recommendations shall include the maintenance plan and replacement procedure.

6 CONCLUSIONS

As we have shown in this paper, service life design in bridges is a sum of small things. A change in attitude is needed, bringing bridge maintenance specialists into the design phase, developing more and better structural solutions for integral bridges, taking great care in detailing expansion joints and bearings, preventing water to accumulate on bridges and last, but not least, always use low permeability concrete with a "nice" cover for reinforcement bars. What can be simpler?

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A practical bridge management system using new multi-objective genetic algorithm

H. Furuta
Department of Informatics, Kansai University, Takatsuki, Osaka, Japan

T. Kameda & M. Erami
Graduate School of Informatics, Kansai University, Takatsuki, Osaka, Japan

ABSTRACT: Recently, maintenance work is becoming more and more important, because the number of structures requiring repair or replacement increases in the coming ten years, in Japan. In order to establish a rational and economical maintenance program, the concept of Life-Cycle Cost (LCC) has gained great attention, which minimizes the total cost of whole lives of structures.

So far, the authors have developed LCC based bridge maintenance systems for existing concrete bridge structures. The concrete bridges are deteriorating due to the corrosion of reinforcing bars and neutralization of concrete. Then, it is necessary to achieve an optimal maintenance plan that can provide appropriate methods and times of repairing or replacement. However, the optimal maintenance problem is very difficult to solve, because it is one of combinatorial problems with discrete design variables and discontinuous objective functions. Furthermore, the problem may become tougher, when it becomes larger and more complex.

Although low-cost maintenance plans are desirable for bridge owner, it is necessary to consider various constraints when choosing an appropriate actual maintenance program. For example, the minimization of maintenance cost needs to prescribe the target safety level and the expected service life time. The predetermination of requirements may loose the variety of possible maintenance plans.

In this paper, it is intended to discover many alternative maintenance plans with different characteristics by introducing the concept of multi-objective optimization. When selecting a practical maintenance plan, it is desirable to compare feasible solutions obtained under the various conditions. This process is inevitable and effective for the accountability by the disclosure of information. Furthermore, another attempt is made to develop a new multi-objective genetic algorithm for the bridge management problems that have a lot of constraints. Several numerical examples are presented to demonstrate the applicability and efficiency of the proposed method.

Maintenance management planning for ten consecutive piers and floor slabs (composite structure of steel girders and reinforced concrete (RC) slabs) is considered here. Each bridge has the same structure and is composed of six main structural components: upper part of pier, lower part of pier, shoe, girder, bearing section of floor slab, and central section of floor slab Environmental conditions can significantly affect the degree of deterioration of the structures and may vary from location to location according to geographical characteristics such as wind direction, amount of splash, etc. In order to prevent deterioration in structural performance, several options such as repair, restoring, and reconstruction are considered. Each repair method has different applicability and effects. Life-Cycle Cost (LCC) is defined as the total maintenance cost for the entire bridge group during its life.

As mentioned before, the formulation of bridge maintenance planning has several constraint conditions. In usual, it is not easy to solve multi-objective optimization problems with constraints by applying Multi-Objective Genetic Algorithm (MOGA).

In this study, an improvement is made on the selection process by introducing the sorting technique. The selection is performed using so-called sorting rules which arrange the list of individuals in the order of higher evaluation values. Then, the fitness values are assigned to them by using the

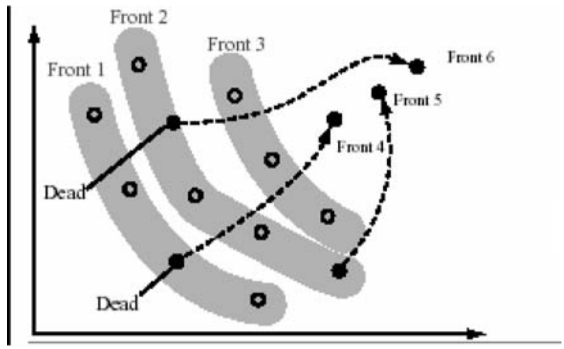

Figure 1. New sorting rules.

linear normalization technique. In usual, if the fitness values are calculated directly according to the evaluation values, the differences among every individuals decrease so that the effective selection can not be done. On the other hand, the linear normalization technique enables to keep the selection pressure constant so that it may continue the selection well. In this study, the selection procedure is improved coupling the linear normalization technique and the sorting technique. Using the evaluation values, the individuals are reordered and given the new fitness values. Figure 1 presents the process of the selection proposed here. The individuals of satisfying the constraints are arranged first according to the evaluation values and further the individuals of unsatisfying the constraints are arranged according to the degree of violating the constraints. Accordingly, all the individuals are given the fitness values using the linear normalization technique.

Then, objective functions are defined as follows:

$$\begin{aligned} \textit{Objective function } 1 : C_{total} &= \Sigma LCC_i \rightarrow min \\ \text{where } LCC_t &= LCC \text{ for bridge } i \end{aligned} \tag{1}$$

$$\begin{aligned} \textit{Objective function } 3 : P_{total} &= \Sigma P_i \rightarrow max \\ \textit{Constraints} : P_i &> P_{target} \\ \text{where } P_{target} &= \text{Target safety level} \end{aligned} \tag{2}$$

In this paper, an attempt was made to formulate the optimal maintenance planning as a multi-objective optimization. By considering LCC and safety level as objective functions, it is possible to obtain the relationships among these two performance indicators and provide bridge maintenance management engineers with various maintenance plans with appropriate allocations of resources. Based on the results presented in this paper, the following conclusions may be drawn:

1. Since the optimal maintenance problem is a very complex combinatorial problem, it is difficult to obtain reasonable solutions by the current optimization techniques.
2. Although Genetic Algorithm (GA) is applicable to solve multi-objective problems, it is difficult to apply it to large and very complex bridge network maintenance problems. By introducing the new multi-objective genetic algorithm, it is possible to obtain efficient nearoptimal solutions for the maintenance planning of a group of bridge structures.
3. The Pareto solutions obtained by the proposed method show discontinuity.
4. In the examples presented, the relation between safety level and LCC is non-linear. The increase of LCC hardly contributes to the improvement of safety level.
5. LCC can be reduced by adopting simultaneous repair works. The proposed method using linear normalization technique and sorting technique can provide many near-optimal maintenance plans with various reasonable LCC values and safety levels.

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Novel management system for steel bridges in Korea

J.S. Kong & S.H. Park
Korea University, Seoul, Korea

S.H. Kim
Infra Asset Management Co. Seoul, Korea

K.H. Park
Korea Institute of Construction Technology, Goyang-Si, Gyeonggi-Do, Korea

D.M. Frangopol
University of Colorado, Boulder, Colorado, U.S.A

ABSTRACT: In Korea the importance of management of bridges has been recognized over a couple of decades, resulting in the development of database and various bridge management assistant tools by both government and private sectors. However, none of them has truly included the expected features of the Bridge Management System (BMS) for the next generation such as the quantification of the effect of maintenance interventions. As a result, life-cycle cost analysis has been performed within the limits of analyses including uncertainties in terms of application time of maintenance interventions and of overall costs for bridge maintenance only. Recognizing the problem, a new research project has been launched to construct a bridge management system which has enough flexibility and extendibility for the next generation BMS. This system has been constructed based on new theoretical background and included many features which were not included in the conventional BMS. The system has been developed for steel bridges first. In this paper, background and important aspects of the novel management system developed in Korea for steel bridges is represented.

The fundamental difference between management systems is due to the difference of assessment system. Moreover the difference of assessment is often originated from the difference in indexing system. Fig. 1. shows the characteristics of different assessment indexing systems. Each system has its own advantages and disadvantages. In the developed system, multi-indexing system has been implemented to gather available information from all kind of sources.

Quantification of the effect of degradation of members is a difficult and tedious process because of the diversities of bridges depending on designer's selections and loading condition. Therefore, it is very important to develop a simple but reliable enough method to compute the lifetime performance of the target structure. Solution may lie on a simple regression model established based on enormous analysis results obtained by considering expected possibilities of loading conditions and structural properties. The Response Surface Method (RSM) has been proposed and used successively to

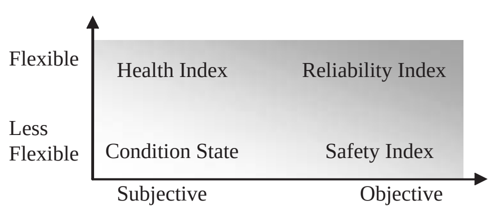

Figure 1. Characteristics of different Indexing Systems.

construct the performance profile of deteriorating bridge members. Among 1,954 steel box bridges in Korea, 633 steel box bridges maintained by the Ministry of Construction and Transportation has been analyzed to decide the design variables.

Based on the results, bridge types have been classified depending on length of bridges, number of spans, span length, width of bridge, different combinations of different span lengths, number of lanes, number of steel boxed, height of steel boxes, variation in thickness of steel plate, support conditions and so on.

Dependency between design variables was investigated to reduce the number of unimportant design variables. Lots of dependent variables could be eliminated and important design variables are identified. For instance, it is concluded that only three design variables, the length of bridge, thickness of concrete slab, and the width of girders are dominant to the behavior of simply supported one-span steel box girder bridges. For two-spans bridges two variables, the length of the first span and the web thickness at the middle of span are added to create the response surface models. Time dependent performance of the three-spans bridges can be analyzed by using the similar variables as of the two-spans bridges except that the length of the first span is replaced by the ratio of the length of the second span to the total length of bridges. It is denoted as explicit model because the explicit performance profile is obtained from numerical analyses.

Quantification of degradation (and also quantification of maintenance effect) based on the direct analysis and the response surface models are not always possible to be established. To mitigate this problem, two additional methods have been applied. One is to use the historical record storing in KOBMS, a database which has been reformed from the BMS accumulating data since 1989. Another method is to use experts' opinions even though this method has to deal with subjective information. In this case, the performance profile is assumed as a linear model and denoted as the implicit model.

Quantification of maintenance effect is basically the same to the problem of quantification of degradation. For maintenance interventions without any direct relationship to the structural performance, such as cleaning drainage, are computed based on the frequencies of occurrences based on historical records. The other maintenance interventions causing variation in structural performance, such as essential maintenance interventions, are analyzed and their effect on structural performance is quantified by the response surface model. Number of design variables to construct the response surface model changes depending on the characteristics of maintenance interventions. For instance, two more variables, time of application and average daily truck traffic volume (ADTT) have added to the initial design variables to compute response surface equations.

There are many elements to be considered to make an efficient but comprehensive bridge management system. Among them the items which will be discussed in the following sections have been being considered extensively. Eventually, cutting edge technologies combined with advanced assessment and analysis methods will produce the smart structural management system for the next generation.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Egnatia Motorway bridge management systems for design, construction and maintenance

A. Liolios
Egnatia Odos A.E., Thermi – Thessaloniki, Greece and Professor of Civil Engineering, Democritus University of Thrace, Xanthi, Greece

D. Kotoulas, F. Antoniou & D. Konstantinidis
Egnatia Odos A.E., Thermi – Thessaloniki, Greece

1 INTRODUCTION

Egnatia Motorway is one of the largest civil engineering projects currently under construction in Europe connecting the major cities in Northern Greece, serving five ports and linking six airports. When completed it is expected to enhance the cohesion of the regions through which it crosses and will have a total length of about 1000 km. The Motorway runs through various mountain ranges and valleys necessitating the construction of a number of major bridges with relatively tall piers and/or long spans/lengths. Arachthos Bridge with a length of 1036 m is the longest bridge on the main axis, whilst Metsovitikos and Votonosi Bridge with the main spans of approximately 235 m will be among the longest span balanced cantilever bridges in Europe.

This paper presents the management systems employed by the client organization Egnatia Odos A.E. (EOAE) responsible for the design, construction and maintenance of the Motorway's bridges. The management systems for the design phase, including procedures for the award of design contracts, internal review and external independent checking of the procured designs are discussed. For the supervision of bridge construction, EOAE enforces a strict quality control and quality assurance system, including application of relevant procedures and testing at every stage of construction. These as well as the bridge maintenance program are also described. In addition, the project monitoring systems employed to provide immediate access to design information and facilitate effective planning of the works, as well as the Quality Assurance System used to ensure that the quality of the bridges meet state-of-the-art standards, are presented. Finally, a seismic risk assessment software tool being developed by EOAE is described.

2 EGNATIA ODOS DESIGN MANAGEMENT SYSTEMS

Throughout its nine year life, EOAE's design department's engineering staff was responsible for managing over 900 design contracts of total value over 190 M€ and carrying out over 12000 technical reviews. In order to manage the vast amount of design work a number of innovative management techniques were employed. Uniform design performance is achieved by the implementation of the EOAE's design guidelines, the OSMEO and the OSAT. Significant design and checking time is saved due to the standardization of culverts and retaining walls, which allows the immediate start of work on site as well as the implementation of fast track principles. The OSMEO and OSAT design guidelines and the standard designs for culverts and retaining walls permitted a systematic approach to defining detailed Scopes of Works for designers, which simplified the approval procedures, as all engineering staff were trained to review submissions based on the requirements defined in the Scopes of Work and the guidelines.

Due to the strict time schedules in order to meet targets set and funding constraints, international competitions were held for a series of call up contracts for all eight disciplines to minimize the time required for contract award for both the above procedures. In this way, following one major competition for each discipline, required design activities are assigned to those designers with relevant call up contracts via work instructions, thus saving a significant amount of time and allowing instant responses in case of emergency.

Submitted designs are reviewed by EOAE staff or external design managers to ensure that the OSMEO Guidelines are adhered to, the relevant codes of practice are employed and sound engineering principles are implemented with durability, future maintenance and whole life cost in mind. To ensure the safety of completed structures EOAE employed national and international design firms following international competition to carry out detailed checking of bridge, non-standard culvert and tunnel designs. Finally, the employment of external consultants by EOAE to carry out technical reviews, independent checking and management of designs resulted in optimised productivity of human resources.

3 EGNATIA ODOS CONSTRUCTION MANAGEMENT SYSTEMS

The management of the construction contracts is carried out by five Regional Services based in Ioannina, Metsovo, Kozani, Thessaloniki, and Komotini who refer to one of the three Regional PM's (West, Central and East). Between 1996 and 2004, day to day construction supervision on behalf of EOAE was carried out by Construction Management Consultants who were employed following international competition. Since 2004, EOAE has set up its own in-house Construction Management Teams (CMT). Bridge construction supervision is based on the construction contract documents, which include EOAE's Technical Conditions of Contract, specifications and detailed designs.

The CMTs ensure that required quality standards are met by:

- applying a specially designed Quality Assurance System for construction management and supervision which is included in the company's Total Quality Management system and includes a series of 34 Operational Procedures.
- evaluating, approving and auditing the Quality Management Systems implemented by the contractors.
- evaluating the Contractors' laboratories and carrying out tests at independent laboratories at each stage of construction (soil tests, concrete cube strength etc.).
- checking the management of suppliers by the contractors.
- ensuring that all plant and equipment are properly maintained.
- checking and accepting "As Built" drawings.
- keeping electronic data bases for all testing and auditing results.
- setting up of schedules and monitoring progress and reporting by using Primavera Project Planner.

4 EGNATIA ODOS MAINTENANCE PROCEDURES

The Operations and Maintenance Guidelines that have been developed by EOAE are for routine maintenance of the motorway, routine maintenance of tunnels' mechanical and electrical installations, main maintenance of pavements, winter maintenance, health and safety and operation of the motorway.

Those guidelines relating to the maintenance of bridges and structures are the guidelines for routine maintenance of the motorway, winter maintenance and operation of the motorway. The Guidelines for Motorway Routine Maintenance describe the requirements for maintenance of the road and its structures, the inspection techniques as well as the use of a Routine Maintenance

Management System (RMMS), which is a computerised database that records all highway assets, inspection and maintenance data. Winter maintenance refers to the required actions for dealing with ice and removing snow for the road surface. The purpose of the Egnatia Motorway Operation Guidelines are to define the relationship of the Company with the road users with the aim at providing users with high quality infrastructure, within reasonable financial ranges while assuring high level of service, comfort and safety. It must be noted that EOAE outsources all operations and maintenance work by employing operations and maintenance contractors following international competitions.

5 SEISMIC HAZARD RISK ASSESSMENT

The seismic hazard risk assessment software tool being developed is expected to allow the prediction of the most likely extent of damage to the bridges of the motorway in the case of a serious earthquake, thus permitting the planning of alternative routes around predetermined weak links.

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East river bridges preventive maintenance program

M.S. Hershey
Hardesty & Hanover, LLP, New York, NY, USA

M. Sharif
New York City Department of Transportation, New York, NY, USA

ABSTRACT: The East River Bridges Preventive Maintenance Program has evolved since it's inception in 1997 to a fully functional program ensuring that the rehabilitated East River Bridges of New York will not be neglected and fall into a state of disrepair as happened in the latter part of the 1900's.

Four major bridges, namely the Brooklyn, Manhattan, Williamsburg and the Queensboro, as a group, are known as the East River Bridges. The rehabilitation of these essential components of the NYC transportation infrastructure has been ongoing for more than 20 years. The ERB Preventive Maintenance Program will preserve the nearly $3 billion investment that has been made at the federal, state and local levels.

This program is funded by FHWA and NYSDOT and administrated by staff engineers employed by NYCDOT. The paper will describe the history and current application of the program to the 4 East River Bridges including:

– ERB-PM Task Force Makeup (FHWA, NYSDOT, NYCDOT)
– Overall Description and Brief History of the Program
– Current Funding, Organization and Staffing Levels
– PM Activities and Frequencies for standard and special components
– Accomplishments to Date including Methods, Data and Quantities
– Future Funding and Planned Contracts

Although the four East River Bridges were essentially built during the same era (late 18th to early 19th century) they are of different design; three are suspension bridges and one (Queensboro) is a cantilever span. As a result the maintenance program for the four structures has tasks that are common to all as well as tasks that are unique to each bridge. All four bridges utilize complex under deck moveable inspection platforms which are essential to maintain in proper working order; however no two are of the same design. Also there are a number of special and unique components such as the solid rod suspenders on the Brooklyn Bridge and the multi-rotational bearings on the intermediate towers of Williamsburg Bridge to be maintained.

The maintenance program is a combined effort of New York City Division of Bridges in-house staff and outside contractors. Certain specialized maintenance activities are performed in-house because of the institutional knowledge and experience of the City workers. Other generalized maintenance activities are contracted out resulting in a successful collaboration of the public and private sectors. This division of effort and responsibilities between the public and private sector results in a healthy combination of resources in which each side wishes to demonstrate their efficiency and cost effectiveness.

This program is the first application of a maintenance program funded by FHWA in New York City. It is anticipated that due to the accomplishments and success of the existing program

that the ERB-PM Task Force will recommend a similar PM program for the NYC movable bridges.

As Director of Bridge Maintenance at the time Mr. Hershey was instrumental in creating the initial proposals for staffing, funding and required PM activities. Mr. Mo Sharif is presently overseeing the program for NYCDOT.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

The potential applicability of the Life-Quality Index to maintenance optimisation problems

M.D. Pandey
Department of Civil Engineering, University of Waterloo, Waterloo, Canada

J.M. van Noortwijk
HKV Consultants, Lelystad, The Netherlands, and Faculty of Electrical Engineering, Mathematics and Computer Science, Delft University of Technology, The Netherlands

H.E. Klatter
Civil Engineering Division, Ministry of Transport, Public Works, and Water Management, Utrecht, The Netherlands

1 INTRODUCTION

The Life-Quality Index (LQI) is a tool for the assessment of risk reduction initiatives that would enhance safety and quality of life. The application of the LQI to maintenance decision problems helps determining an optimal trade-off between the risk and consequences of traffic accidents and the reduction in maintenance efforts. The LQI is applied to quantify the societal willingness to pay – which is an acceptable level of public expenditure in exchange for a reduction in the risk of death that results in improved life quality – and to obtain an optimal balance of the costs of maintenance and risk. The feasibility of this approach is investigated using a realistic example as a case study. The influence of maintenance efforts on road-surface quality and traffic safety are assessed.

2 LIFE-QUALITY INDEX

To avoid failure, preventive maintenance is performed. In order to determine an appropriate maintenance strategy, we need to balance the costs of maintenance and failure. The consequences of failure can be subdivided into monetary and non-monetary losses. Monetary losses are relatively easy to value. Non-monetary losses represent losses suffered by individuals or groups of individuals. The "costs" associated with these consequences are much harder to estimate. Non-monetary cost can be dealt with using the LQI. If maintenance improves the life quality, it must be quantifiable in terms of money or life years and it can be included in the LQI. On the other hand, if the improvement is purely abstract, then the LQI or any other measure is of no help. An advantage of the LQI is that a direct estimation of the value of human life is avoided. Instead, the LQI focuses on wealth, long life in good health, and leisure.

The Life-Quality Index at the societal level is defined as

$$L = G^{\frac{c}{1-c}} E, \tag{1}$$

where G is the gross domestic product (Euro/person/year), E is the life expectancy in the country, and c is a constant, denoting the annual fraction of work time per person required for producing G. The components of the LQI relate to the key human concerns: creation of wealth, duration of life and the time available to enjoy life in good health. The derivation of LQI is formally described in the full paper. This formulation links the LQI to concepts generally understood by practitioners in decision analysis, economic modelling, cost-benefit analysis and risk assessment.

One important goal in managing risks to life safety is to determine an acceptable level of expenditure that can be justified on behalf of the public in exchange for a small reduction in the risk of death without comprising the life quality. This value can be considered as a fair estimate of the Societal Willingness to Pay (SWTP) for safety.

3 APPLICATION OF THE LQI TO MAINTENANCE AND SAFETY

To optimise the maintenance of structures and infrastructures, the Netherlands Ministry of Transport, Public Works, and Water Management (Rijkswaterstaat) has defined the so-called Basic Maintenance Level (BML) that should be satisfied against lowest life-cycle costs. The relation between the state of maintenance and the traffic safety should be an important aspect of this optimisation. In practice, this aspect is treated qualitatively. A potential application of LQI to the field of maintenance in quantifying this relation is explored using a specific example. This example assesses the effect of maintenance on the road surface quality and the number of accidents and fatalities. However, data on the relation between maintenance and traffic accidents are scarce. For this purpose, a case has been constructed using data from different sources and some simplifying assumptions to estimate the relations between the quality of the road surface and the occurrence frequency of traffic accidents and the costs of traffic accidents and maintenance.

The total number of traffic fatalities on the main road network was 155. The total costs of road accidents – *including* medical cost, loss of productivity, material cost, clearance cost, traffic-delay cost and *excluding* immaterial cost due to deaths – are estimated at 6,000 million Euro. An estimate of the fraction of traffic accidents related with the quality of the road surface is 7.5%. Applying the same percentage to the number of traffic fatalities on the main road network, 12 deaths are assumed to be related to the road surface quality. From the total costs of road accidents, about 50 million Euro is related to the surface quality of the main road network. This provides an approximate estimate of the benefit of better maintenance.

A reduction of quality standards for the quality of the road surface – e.g., in terms of skid resistance and rut depth – by 20 to 40% leads to a 20 million Euro reduction of cost of pavement maintenance. The effect of this reduction on the number of fatalities and accident costs is roughly estimated to be a 50% increase in both the number of fatalities (an increase of 6 to a total of 18 fatalities) and the cost of traffic accidents excluding fatalities (an increase from 50 to 75 million Euro).

Using the LQI, we can now determine the SWTP of *not* reducing the maintenance and preventing 6 additional fatalities. The LQI is calibrated using the specific economic and demographic data for the Netherlands. The total benefit of current maintenance is the sum of SWTP (14 million Euro) and averted cost of traffic accidents (25 million Euro). Thus, the total benefit of 39 million Euro is much greater than the reduction in maintenance cost of 20 million Euro. In conclusion, the LQI method supports the decision of ongoing road maintenance.

4 CONCLUSIONS

The paper concludes that the LQI can be promising in supporting decisions for structure and infrastructure maintenance. The application of the LQI to maintenance decision problems helps determining an optimal trade-off between the risk and consequences of traffic accidents and the reduction in maintenance efforts. In traditional life-cycle costing analyses of maintenance programs, the cost assessment is often performed quantitatively and the risk assessment qualitatively. A quantitative risk assessment looks promising from a conceptual point of view, which is accomplished in the paper using the LQI method.

The example with a preliminary cost-benefit analysis of the current road maintenance program with respect to a possible decrease in the road quality standards shows that the application of the LQI is feasible and useful. A detailed analysis of the relation between the traffic accidents and the quality of road and maintenance actions is needed to refine the LQI analysis presented in the paper. In this respect, the use of geographic information systems can be helpful. The LQI framework can be applied to assess the impact of other investment in safety-related programs, such as road and bridge safety, in The Netherlands.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Optimal cost allocation for improving the seismic performance of road networks

Hitoshi Furuta
Department of Informatics, Kansai University, Takatsuki, Osaka, Japan

Koichiro Nakatsu
Graduates School of Informatics, Kansai University, Takatsuki, Osaka, Japan

Dan M. Frangopol
Department of Civil, Environmental, and Architectural Engineering, University of Colorado, Boulder, CO, USA

ABSTRACT: Japan has been exposed to many natural hazards such as typhoons, tsunamis and earthquakes. However, road networks have not been designed to protect against all such natural hazards. Moreover, even the most modern design specifications can not guarantee the absolute safety due to economic constraints. Therefore, it is necessary to develop a comprehensive disaster prevention program based on the recognition that road networks may be unavoidably damaged when very strong earthquakes occur. After earthquake disaster, road networks play important roles in rescue, evacuation activities, extinguishing fires, and disaster-relief activities. The damage of the road network is associated with severe effects on the daily life and economic activities of people. In this study, the seismic performance of road network is evaluated based on the following three factors: (a) the characteristics of road network such as existence or non-existence of detour, (b) importance of bridge such as existence or non-existence of a medical center near a particular bridge of the network, and (c) road traffic condition. The purpose of this research is to provide a framework for optimal allocation of strengthening cost for increasing the seismic performance of road networks. In this paper, an attempt is made to discuss the relationship between strengthening cost of an entire road network and safety level of this network by using Multi-Objective Genetic Algorithm (MOGA). Namely, the following two objective functions are considered: (a) strengthening cost of road network is minimized; (b) safety level of road network is maximized. There are trade-off relations among these two objective functions. For example, safety level decreases when strengthening cost decreases. Multi-objective optimization can provide a set of Pareto solutions that can not improve an objective function without making other objective functions worse. The objective functions are defined as follows:

$$SC = \sum_{i \in J^0} SC_i \rightarrow \min$$

$$SL = \sum_{i \in J^0} B_i \times S_i \rightarrow \max$$

where SC is the strengthening cost of the road network, SC_i is the strengthening cost of the i-th bridge of the network, SL is safety level of the road network, B_i is the importance of the i-th bridge and S_i is the safety level of the i-th bridge.

In the implementation of MOGA, the GA parameters considered are number of individuals = 200, crossover rate = 0.60, mutation rate = 0.05, and number of generations = 5000. Figure 1 shows the results obtained by MOGA. Comparing the Pareto solutions D and E with

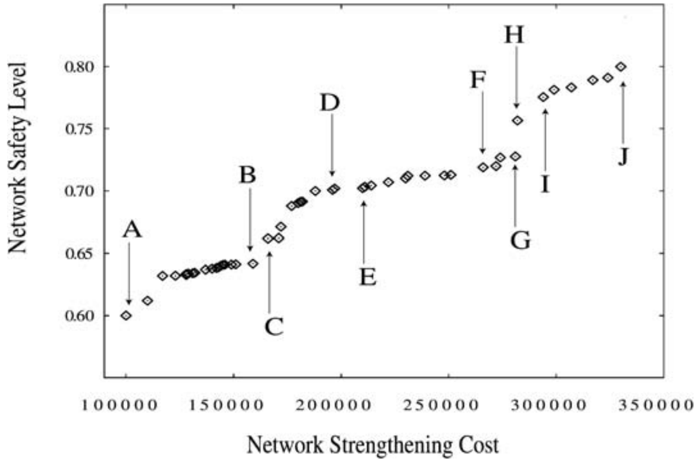

Figure 1. Pareto solutions obtained by MOGA.

respect to safety level, there is no significant difference. However, considering strengthening cost, solution E is much more expensive than solution D. Conversely, the Pareto solutions G and H are similar with respect to the associated strengthening cost, but they are very different with respect to their associated safety level. With respect to safety level, the Pareto solutions D and F are similar. However, there is a significant difference between the strengthening cost associated with these two solutions. Therefore, when selecting a strengthening program, the proposed method enables to compare feasible optimal solutions associated with different values of the objectives.

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Development of bridge maintenance planning support system using multiple-objective genetic algorithm

Hitoshi Furuta & Yasuhiro Takenaka
Department of Informatics, Kansai University, Takatsuki, Japan

Takahiro Kameda & Isao Tsukiyama
Graduate School of Informatics, Kansai University, Takatsuki, Japan

ABSTRACT: In Japan, maintenance work is becoming more and more important, because the number of existing bridges requiring repair or replacement increases in the coming ten years. In order to establish a rational maintenance program, it is necessary to develop a cost-effective decision-support system that can provide us with a practical and economical plan. Although low-cost maintenance plans are desirable for bridge owner, it is necessary to consider various constraints when choosing an appropriate actual maintenance program. For example, the minimization of maintenance cost requires to prescribe the target safety level and the expected service life time. The predetermination of requirements may loose the variety of possible maintenance plans. Namely, it may be possible to find out a better solution that can largely extend the service life if the safety level can be sensitively decreased even with the same amount of maintenance cost.

In this paper, it is intended to discover many alternative maintenance plans with different characteristics by introducing the concept of multi-objective optimization. When selecting a practical maintenance plan, it is desirable to compare feasible solutions obtained under the various conditions. This process is inevitable and effective for the accountability by the disclosure of information. Thus, an attempt is made to develop a decision support system for the bridge maintenance that can provide us with several alternative plans by applying Multi-Objective Genetic Algorithm (MOGA). However, it is not easy for the decision maker to choose an appropriate solution from many Pareto solutions. In order to help the decision maker, a 3D graphical system is developed using JAVA. It is important to find the appropriate repair methods and the branching points of cost effectiveness. Several numerical examples are presented to demonstrate the applicability and efficiency of the system proposed here.

In order to find out several useful solutions from the set of Pareto solutions, a 3D graphical system is developed in this paper. The system aims to help the decision maker to select several solutions that satisfy some requirements through checking their constraint conditions by using JAVA3D. The system consists of three subsystems; 1) 3D representation, 2) general representation, and 3) graphical representation. Each representation is implemented using JAVA.

The general representation system can list the solutions obtained by the 3D representation system. The solutions can be arranged in the order of evaluation values. The solutions are listed up, corresponding to the range defined by the 3D representation system. While the 3D representation system is useful to grasp the relations and tendencies of solutions, the general representation system is useful to show the characteristics of each solution.

The graphical representation system can provide us with the detail of repair methods calculated by MOGA. The system can play a role in checking the appropriateness of the obtained repair methods and in finding out the tendency or pattern of repair program. Observing and comparing the pattern obtained, it is possible to discover the branching points of short, medium and long term repair plans.

Figure 1 shows the representation of the graph by the proposed 3D representation system. For example, it is possible to find out a gap among the solutions, which may be caused by the

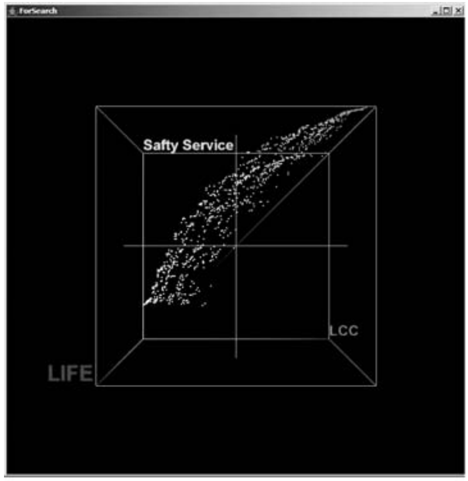

Figure 1. Java 3D Application for the make decision system 1.

cost reduction by the common usage of scaffold. Apparently, it is possible to obtain more useful information by using the 3D representation instead of the usual 2D representation.

In this paper, an attempt was made to formulate the optimal maintenance planning as a multi-objective optimization. Furthermore, a 3D graphical representation system was developed to find out several useful solutions from the set of Pareto solutions obtained by the optimal maintenance planning system using Multi-Objective Genetic Algorithm (MOGA).

By considering LCC, safety level, and service life as objective functions, it is possible to obtain the relationships among these three performance indicators and provide bridge maintenance management engineers with various maintenance plans with appropriate allocations of resources. Since the optimal maintenance problem is a very complex combinatorial problem, it is difficult to obtain reasonable solutions by the current optimization techniques. Although Genetic Algorithm (GA) is applicable to solve multi-objective problems, it is difficult to apply it to large and very complex bridge network maintenance problems. By introducing the technique of Non-Dominated Sorting GA-2 (NSGA2), it is possible to obtain efficient near-optimal solutions for the maintenance planning of a group of bridge structures. However, it is not easy for the decision maker to choose an appropriate solution from many Pareto solutions. In order to help the decision maker, a 3D graphical system is developed using JAVA. It is important to find the appropriate repair methods and the branching points of cost effectiveness.

The Pareto solutions obtained by the proposed method show discontinuity. This means that the surfaces constructed by the trade-off relationships are not smooth so that an appropriate long term maintenance plan cannot be created by the simple repetition of short term plans. In the examples presented, the relation between safety level and LCC is non-linear. The increase of LCC hardly contributes to the improvement of safety level. On the other hand, LCC and service life are almost linearly related. Therefore, service life can be extended if LCC can be increased. However, there is no distinct relation between safety level and service life. LCC can be reduced by adopting simultaneous repair works. The proposed method using NSGA2 can provide many near-optimal maintenance plans with various reasonable LCC values, safety levels and service lives.

Reliability analysis and optimal design of deteriorating structural systems

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Lifetime nonlinear analysis of concrete structures under uncertainty

Fabio Biondini
Department of Structural Engineering, Politecnico di Milano, Milan, Italy

Franco Bontempi
Department of Structural and Geotechnical Engineering, University of Rome "La Sapienza", Rome, Italy

Dan M. Frangopol
Department of Civil, Environmental, and Architectural Engineering, University of Colorado, Boulder, Colorado, USA

Pier Giorgio Malerba
Department of Structural Engineering, Politecnico di Milano, Milan, Italy

Reliability based analysis and design procedures must account for the uncertainties affecting the structural performance. However, for concrete structures the number of uncertain parameters involved in design is usually large, and probabilistic evaluations may become very complex. This aspect is particularly emphasized when full nonlinear analyses are required to investigate the structural performance. For this reason, in design practice, the complexity of the reliability problem is reduced by means of simplified probabilistic methods in which only a few uncertain parameters are considered as random variables, typically only those which mainly affect the structural response. In concrete design such variables are generally associated with the material properties, i.e. concrete and steel strengths, while for the other mechanical and geometrical parameters the deterministic nominal values are assumed.

This design approach is calibrated by the codes to be on the safe side and is proven to be effective for practical purposes. However, such calibration mainly refers to undamaged structures and does not account for the effects of uncertainty associated with the unavoidable sources of structural damage which during time may strongly modify the structural reliability level. In fact, for concrete structures immersed in aggressive environments the structural performance must be considered as time-dependent, mainly because of the progressive deterioration of the mechanical properties of materials which makes the structural system less able to withstand the applied actions. In this context, a design procedure aimed to achieve the required level of structural performance not only at the initial time, but over the whole expected service life of the structure, must consider that the relative importance of the uncertainty effects associated with each design parameter may significantly vary during time. To this aim, the role played by both the deterioration process and the corresponding evolution of the structural performance on this variation needs to be investigated and clarified. In this way it would be possible to calibrate the design procedures with respect to the random variables which actually affect the lifetime structural response.

This paper presents the results of a preliminary investigation aimed to highlight the time evolution of the uncertainty effects associated with the different parameters which define the structural problem. The study is developed by using a novel methodology for time-variant reliability analysis of concrete structures subjected to diffusive attacks from external aggressive agents, recently proposed by the authors (Biondini et al. 2004*a*, 2004*b*, 2006). Based on this procedure, a probabilistic time-variant nonlinear analysis of a concrete box-girder cross-section undergoing diffusion is carried out by Monte Carlo simulation and the corresponding time evolution of the structural performance is investigated with reference to suitable indicators on the nonlinear behavior.

The effects of the uncertainty associated with each random variable on the performance indicators are finally quantified and compared by means of suitable time-dependent sensitivity factors. The obtained results show that:

– The cracking moments in the undamaged scenario mainly depend on the concrete strength. Such dependency quickly decreases during time, and after about 20 years the damage rate of concrete becomes the more important parameter. However, this contribution tends to progressively disappear. At the end of service life an important role is played by the damage rate of steel and, for the positive cracking moment, by the geometrical dimensions of the cross-section.
– The resistant moments in the undamaged scenario mainly depend on the steel strength. Such dependency quickly decreases during time and become negligible after about 25 years. At this point, for positive resistant moment the damage rate of steel becomes the more important parameter for the whole remaining service life. On the contrary, for negative resistant moment a clear dependency no longer holds until the last years of service life, when the damage rate of steel begins to give a significant contribution.
– The curvature ductility in the undamaged scenario mainly depend on the steel strength. For positive bending behavior such dependency is maintained along the whole service life. On the contrary, for negative bending behavior more complex correlations emerge after about 15–20 years, when the role played by the location of the steel bars and the damage rate of concrete becomes more important. However, these contributions tend to disappear at the end of service life, when a significant contribution is given by the area of the steel bars.

The results of this investigation proved that for concrete structures in aggressive environments the relative importance of the uncertainty effects associated with each design parameter may significantly vary during time. In particular, the classical view in which the main role in concrete design is played by the uncertainty associated with the material strengths needs to be reviewed to account for the effects of the unavoidable sources of structural damage which during time may strongly modify the structural performance and the corresponding reliability level.

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Probabilistic lifetime assessment based on limited monitoring

F. Biondini
Department of Structural Engineering, Politecnico di Milano, Milan, Italy

D.M. Frangopol
Department of Civil, Environmental, and Architectural Engineering, University of Colorado, Boulder, Colorado, USA

E. Garavaglia
Department of Structural Engineering, Politecnico di Milano, Milan, Italy

In an aggressive environment, the uncertainties associated with the occurrence of environmental attacks on structures and structural materials require studying their deterioration from a probabilistic point of view. Each probabilistic approach needs a reliable experimental data set and a coherent and meaningful definition of the physical aspects of the phenomenon under investigation.

The prediction of the time taken to reach a given structural damage level is an important issue in planning strategies for the maintenance and repair of existing structures. A valuable set of useful data can be found from monitoring activities, but these activities are often limited in space and time. Limited observed data might be available through monitoring. However, the data recorded by monitoring can be uncertain. Therefore, of paramount importance is the accurate prediction of the main parameters defining structural performance and its evolution over time based on limited monitoring activities.

In this paper, an original approach is applied on sets of data concerning the deterioration of concrete structures in aggressive environments with the aim to investigate which parameters are more influential on the system reliability and its deterioration over time. The proposed procedure is also applied to study possible maintenance strategies and their effect on the time-dependent system reliability.

The application refers to a bridge pier with cellular cross-section. The structure is considered to be immersed in an aggressive environment. The diffusion process of the agent is simulated by using cellular automata. Structural damage is modeled by introducing a degradation law of the effective resistant area for both the concrete matrix and the steel bars.

A Monte Carlo simulation is carried out and the results of this simulation will be assumed as a large sample of experimental data that fully characterize the actual evolution of the structural performance. In this way, a set of simulated monitoring data will be extracted from such database in order to apply the proposed procedure. On the basis of a simulated monitoring composed by three recording instants (10, 20 and 30 years), the fragility curves have been built, they are able to predict the exceedance probability for the system connected with a given damage level.

The proposed model has been also used as a tool to plan maintenance and/or rehabilitation interventions. The restoring of the resistant bending moments in the two principal directions can be approached in two different ways: *essential* or *preventive*; it is then possible to create mixed scenarios of maintenance and to identify optimal maintenance solutions.

Some strategies based on the cost minimization and associated to mixed scenarios of maintenance have been analyzed. With reference to a target reliability level $R^*=0.97$, Figure 1 shows the reliability function $R(t)$ for four different scenarios of *preventive* maintenance. In all scenarios, when the target threshold R^* is reached from one of the two parameters a maintenance is made in order to increase the structural performance. The maintenance can lead the resistant bending moments to reach the

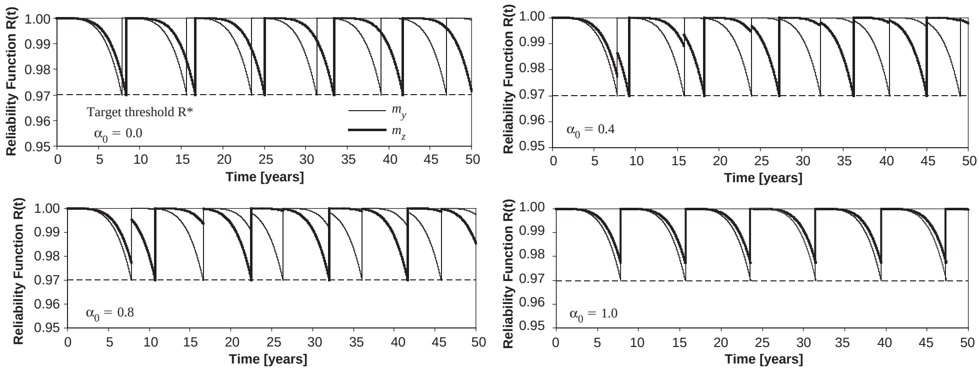

Figure 1. Time evolution of reliability function $R(t)$ and possible maintenance scenarios leading to a structural lifetime $T \geq 50$ years.

initial level of performance or a percentage of it. By denoting α_y and α_z the parameters which represent the corresponding percentage of restoring, it is assumed:

$$\begin{matrix} \alpha_y = 1 \\ \alpha_z = \alpha_0 \end{matrix} \quad \text{if } R_y = R^* \qquad\qquad \begin{matrix} \alpha_y = \alpha_0 \\ \alpha_z = 1 \end{matrix} \quad \text{if } R_z = R^*$$

where $0 \leq \alpha_0 \leq 1$ is a suitable parameter defining the maintenance scenario.

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Structural response evaluation of two-blade bridge piers subjected to a localized deterioration

L. Sgambi & F. Bontempi
University of Rome "La Sapenza", Rome, Italy

E. Garavaglia
Polytechnic of Milan, Milan, Italy

EXTENDED ABSTRACT: Two-blade bridge piers represent today an innovative solution to optimize the structural behavior of long viaducts subjected to seismic actions (Biondini 2001, Calzona & Bontempi 2001). In fact, a proper design of the structure provides a correct transmission of vertical loads to the foundation structures and the possibility to allow horizontal displacements without the use of kinematics mechanisms. Due to the particular configuration of the pier (a rigid box section in the bottom part and two flexible blades at the top connected with the viaduct deck), the structural behavior can be achieved only using refined analysis formulations and effective numerical tools. In particular, during the nonlinear response, the shear effects are very important and can govern the structural behavior. Nonlinear models have to be able to consider the shear damaging with sufficient accuracy otherwise, the numerical response would be affected by heavy errors. In addition, uncertainties on material characteristics can affect the numerical response. To handle these important uncertainties involved in the problem, the use of probabilistic or fuzzy approaches can be suitable.

In this paper, to show the influence of the degradation on the structural response a new and a damaged two-blade pier will be analyzed. Probabilistic evaluation of the material properties are considered to improve the reliability of the numerical analyses. The material properties on the numerical model are not considered in deterministic way, but having a probabilistic distribution. Physical consideration has suggested to model the material properties with a Lognormal distribution. To reach a significant value of the solution in the probabilistic approach, 30 piers are considered with different mechanical properties.

To study the influence of the position of a generic deterioration, one has introduced in the model a portion of the pier deteriorated having 4 *m* of height. Clearly, the quota of the deteriorated section

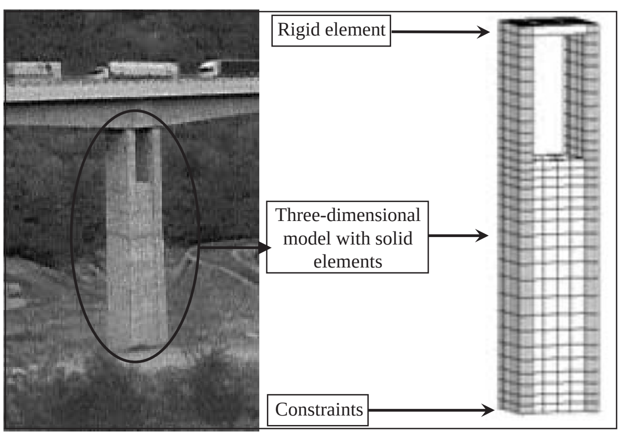

Figure 1. A pier image and the three-dimensional model used in the analyses.

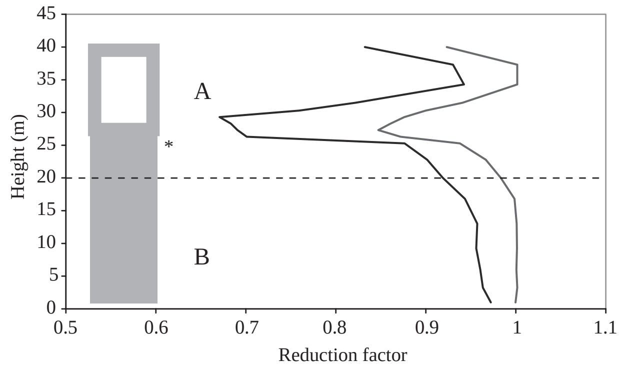

Figure 2. Influence of the deterioration on the ultimate behavior of the pier. On the left, the curve of the displacement reduction factor; on the right, the curve of the strength reduction factor.

influences the global response of the pier. Therefore, one has considered 19 different quotas in order to study the structural sensitivity at the deterioration. For each different quota, 30 different piers with initial mechanical properties, assumed as stochastic random variables having a Log normal distribution, were considered.

To summarize the results, it is possible to define a reduction factor like:

$$r = \frac{\langle f \rangle}{\langle f_{und} \rangle} \tag{1}$$

where $\langle f \rangle$ is the expected value of f greatness, for the 30 evaluations performed, and $\langle f_{und} \rangle$ is the expected value of f for the 30 not degraded piers. In this way, the reduction factor r is 1 if the pier is not damaged, and assumes values among 0 and 1 in the other cases. The left curve in the Figure 1, represent the reduction factor of the displacement while the right curve represent the reduction factor of the strength. It is possible to note that the reduction factor increase quickly when the deteriorated portion is close to the section (*). Therefore, it is possible divide the pier in two zones. The zone A) in which the structural behavior is strongly influenced from a deterioration and the zone B) where the deterioration has not effect on the nonlinear behavior of the structure.

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Stiffness matrices and genetic algorithm identifiers toward damage detection

L. Faravelli & F. Marazzi
Department of Structural Mechanics – University of Pavia, Pavia, Italy

ABSTRACT: The existing civil and architectural facilities would take advantage from the availability of non-destructive investigation techniques, whose outcome is the evaluation of the current state of a structure. Different approaches have been pursued, either in a targeted way or integrated by analytical models able to drain the necessary information on the structural response. Usually, a comparison of current versus initial values of the main dynamic parameters of the system drives the diagnostics, which consists of detecting the presence and of identifying the entity and the location of some local deterioration or damage.

The research effort reported in this paper aims to identify the changes between undamaged and damaged structural configurations by the application of Genetic Algorithms (GAs). In particular, GAs are used to evaluate the stiffness components of a multi degrees of freedom structure. For this purpose, the following steps are taken. The response measurements are collected and the eigenvalues of the system are determined from them, in real-time. A specific genetic algorithm estimates the corresponding stiffness matrix of the structure. From signals recorded during different periods of the structure lifetime, one obtains a sequence of stiffness matrices whose comparison allows to detect the presence of damage, if any, and to localize it.

Evolutionary Algorithms (EA) are adopted in order to detect the presence of a damage and to specify its location.

The evolutionary algorithm is applied to solve the so-called inverse problem, i.e., the identification of the stiffness parameters, with the required accuracy in view of the identification of possible damage. The parameters to be identified undergo an optimization process that can be either approached in the frequency domain or in the time domain.

The so-called Differential Evolution (DE) technique is used as algorithm of optimization. In the frequency domain, two approaches can be envisaged. The first procedure is based on a cost function which compares the vector of the exact structural frequencies ($\boldsymbol{V}_{\text{exact}}$), computed by a numerical method or derived from the experimental data, with the vector of structural frequencies obtained at the current step of the DE ($\boldsymbol{V}_{\text{gen}}$). Such an approach requires the complete knowledge of the natural frequencies of the system. The second procedure will require the knowledge, from the experimental information, of m_1 eigenvectors and m_2 natural frequencies of the system.

In this framework, the inverse problem is solved starting from the time histories of acceleration measured at the three levels of the structure in Figure 1.

The figure case of study was assembled at the ELSA laboratory of the Joint Research Centre (JRC) in Ispra, Italy, and it consists of three levels supported by six steel columns, in a two bays by one scheme. The floors are made of reinforced concrete slabs laying on nerved steel plates. The dimensions show a total height of 6 m along the z axis, while each bay is 4 m by 2.5 m in the x and y directions, respectively. The two longitudinal bays are along the x axis. The height of each floor is 2 m. The columns are HEB140 and they are made of steel Fe360. The beams are IPE180, also made of steel Fe360. The structure is anchored at the base.

The target of this paper is the generalization of the process from the two-dimension schematization adopted in the previous works, to the exploitation of a three-dimensional sensors information, so that the eigenvalues of the system are globally estimated.

Figure 1. The “Baby-Frame” specimen at the ELSA laboratory of the JRC in Ispra, Italy.

A preliminary numerical study is carried out, and the simulated time histories are used to approach the inverse problem solution. The EA procedure is successively applied to the time histories recorded during the laboratory tests.

By using the set of experimental results collected at the ELSA laboratory, this paper shows how a genetic algorithm (GA) process allows to pursue the identification of the stiffness values.

As a further application of this result, the comparison of the response records collected during successive periods of time can be exploited to highlight the presence of damage in terms of stiffness reduction, and also allows for its localization.

The issues related to the convergence on local minima could be overcome by adopting a more sophisticated formulation of the cost function, which includes also the eigenvectors computed from the dynamic measurements of the structure. This possible development is currently under investigation.

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Influence of the corrosion damage scenarios on the residual life of bridge grillages

T. Albanesi
University of Rome, Rome, Italy

Z. Rinaldi
Dep. of Civil Engineering, University of Rome "Tor Vergata", Rome, Italy

C. Valente
PRICOS, "G. D'Annunzio" Chieti-Pescara, Italy

L. Pardi
Manutenzione e Standard, Autostrade per l'Italia Spa, Rome, Italy

ABSTRACT: The reinforcement corrosion is nowadays considered one of the primary causes for the decrease of the load carrying capacity of r.c. structures. This phenomenon is particularly felt for bridge structures very often subjected to aggressive environment and/or to thaw salts, as witnessed by the need for continuous maintenance and important repairs or strengthening due to structural deterioration. The rebar diameter reduction, induced by the corrosion, and particularly the pitting effects, seems to influence both the steel yield stress and the ultimate steel strain. In fact, as the load increases, the plastic deformation localizes in small zones and the consequent reduction of yielding spread can lead to a brittle failure.

Aim of the paper is the analysis of the behavior of a reference r.c. bridge grillage, representative of a wide class of this structural typology, subjected to corrosion. Different damage scenarios are considered and the most critical situation is highlighted. The analyses are performed with numerical procedures; the corrosion effects on the structural elements are calibrated on the basis of the results of experimental tests carried out by some of the authors.

The study has been conducted on a r.c. bridge grillage representative of a wide class of Italian viaducts, selected through a critical analysis of different geometrical parameters.

The chosen case presents a static scheme of simple supported beams, and this very common peculiarity allows a clear interpretation and understanding of the phenomena related to the rebar corrosion. The system is composed of 4 main beams with a 6 meter spans and 4 transverse beams characterized by 3 m spans, completed with a r.c. slab with constant thickness of 22 cm, with load amount and distribution in agreement with the Italian code.

The simulation of the grillage response is carried out with the well-known program SAP9-NL, by performing a static non-linear analysis, through an accurate modelling of the moment-rotation capacity of the plastic hinges.

In this preliminary case, in order to have a clear interpretation of the results, the corrosion effects are simulated only with a reduction of the yielding and post-yielding strengths.

In order to investigate different damage scenarios, localized corrosion phenomena are considered (pitting corrosion). This kind of attack is particularly dangerous for the structure, due to its local nature, that causes, even if in smaller zones, highest strength reductions, if compared with an "equivalent" uniform corrosion.

On the basis of the results obtained by the experimental tests, related to a uniform corrosion, highlighting a reduction of 30% of the ultimate bending moment, for a pre-fixed corrosion level (assumed as 100%), it appears reasonable to consider a 50% of strength reduction in pitting attack. The critical zones to be subjected to corrosion are detected on the basis of a linear elastic analysis

of the grillage, allowing the definition of the most stressed sections and with the aid of indications provided by the Italian Highway Society on the most recurring damage zones. As a result 5 damage scenarios have been assumed, with corrosion localized in defined sections of the edge beam or near the transverse joints.

For each corrosion distribution a series of non-linear static analyses are performed, by increasing the applied loads, while the corrosion amount in the critical sections is kept fixed. The obtained results are interpreted and discussed mainly through capacity curves. The evolution of the plastic hinges up to the collapse is also analyzed and the behavior of the structures is judged in the framework of performance criteria based on different performance levels of the plastic hinges.

The safety level respect to the failure is evaluated as the ratio between the maximum collapse load and the load provided by the current Italian code.

The obtained results allow to point out the safety of the damaged structure respect to the sound one, as a function of the corrosion distribution, and then the most dangerous damage scenario for the structural scheme. In particular the following aspects are highlighted:

1. the strength supply of the sound grillage;
2. the strength supply of the damaged grillage as a function of the corrosion scenario;
3. the stress redistribution, after yielding, in the sound and corroded grillage;
4. the most dangerous damage scenario.

The obtained results have highlighted that the global behavior is strongly affected by concentrated damage on the longitudinal beams, while the corrosion of the transverse beams leads to almost negligible effects.

For the analyzed geometry and load distribution the most critical situation is related to the corrosion in the edge beam midspan. In this case the loss of safety level is equal to the loss of local strength (about 50%) that mean that no redistribution occurs in the structure.

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Optimal design of deteriorating structural systems

Lorenzo Azzarello, Fabio Biondini & Alessandra Marchiondelli
Department of Structural Engineering, Politecnico di Milano, Milan, Italy

ABSTRACT: The optimum structural design is generally aimed to find the minimum cost solution under prescribed performance requirements. In the classical approach, the time evolution of the structural performance induced by the progressive deterioration of the system properties is not properly considered, since the attention is focused on the initial configuration only, in which the structure is fully intact. However, this approach is not consistent with the actual nature of the design problem, which should lead to concept structures able to comply with the desired performance not only at the initial time of construction, but also during the whole expected service lifetime, despite of the deteriorating effects induced by the unavoidable sources of mechanical damage. In addition, the effects of maintenance interventions on the actual cost optimality of the selected design need to be consistently accounted for by a proper formulation of the maintenance cost.

To overcome the inconsistencies involved in the classical formulation of the design problem, this paper presents a new conceptual approach to the minimum lifetime cost design of deteriorating structures under multiple loading conditions. To this aim, the structural damage is modeled by introducing a proper material degradation law and the structural analysis is carried out at different time instants in order to assess the time evolution of the system performance. The design constraints are related to both the time-variant stress and displacement state, as well as to the amount of structural damage. The objective function is formulated by accounting for both the initial cost of the structure and the costs of possible maintenance interventions, that are properly discounted over time and assumed to be proportional to the actual level of structural damage.

The results of the lifetime structural optimization of the truss structure shown in Figure 1.a are presented to highlight the effectiveness of the proposed formulation. Three alternative loading conditions are considered: (a) $F_z = F = 1$ MN and $F_y = 0$; (b) $F_z = 0$ and $F_y = F$; (c) $F_z = F_y = F$. A circular cross-section with diameter d_i is assumed for each member $i = 1,\ldots,10$. The structural shape is defined by the coordinates (z_j,y_j) of each node $j = 1,..7$. The design model is based on the following geometrical constraints: $d_1 = d_4$, $d_2 = d_5$, $d_3 = d_6$, $d_7 = d_8$, $d_9 = d_{10}$, $z_1 = z_4 = 0$; $y_1 = y_4$, $(z_2,y_2) = (z_5,y_5)$, $(z_3,y_3) = (z_6,y_6)$, $y_7 = 0$, $z_7 = L = 3$ m. In this way, the structural optimization problem is defined by the $n = 10$ design variables $\mathbf{x} = [d_1\ d_2\ d_3\ d_8\ d_9 | y_1\ z_2\ y_2\ z_3\ y_3]^T$, for which the following side constraints are assumed: $0.05\,\text{m} \leq d_i \leq 0.50\,\text{m}$; $0.10\,\text{m} \leq y_j \leq 1.00\,\text{m}$; $0.10\,\text{m} \leq z_2 \leq 2.80\,\text{m}$; $0.20\,\text{m} \leq z_3 \leq 2.90\,\text{m}$. Time-variant behavioral constraints on the acting stress in each member i and loading condition ℓ are defined by assuming the initial allowable stress $\bar{\sigma}^+_{0,i,\ell} = -\bar{\sigma}^-_{0,i,\ell} = 160$ MPa in tension and compression, respectively, and by considering the critical threshold of structural stability in compression.

In case damage is not considered, the optimal solution described in Figure 1.b and Table 1 is achieved. This solution is compared with the structures shown in Figure 2, obtained by solving the lifetime optimization problem for a prescribed damage scenario where a maintenance intervention, aimed to partially or totally restore the initial performance of the damaged structure, is carried out at the end of each design monitoring period $T_D = \{1,\ 50\ \text{years}\}$ during a service lifetime $T_S = 100$ years. This comparison proves that the optimal design solution $\mathbf{x}^*$ is strongly influenced by damage with respect to both the dimension of the cross-sections and the location of the nodal points. In addition, it significantly depends on the parameter T_D, which is directly related to the frequency of maintenance interventions. Such results show the fundamental role played by both the time-variant performance and the maintenance plan in the selection of the optimal structural design.

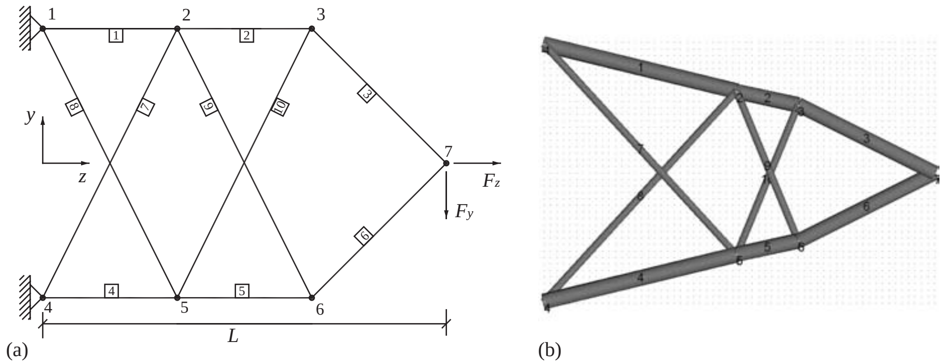

Figure 1. Truss structure. (a) Design model. (b) Optimal solution without damage.

Table 1. Truss structure. Optimal value of the design variables without damage.

d_1 [mm]	d_2 [mm]	d_3 [mm]	d_8 [mm]	d_9 [mm]	y_1 [mm]	z_2 [mm]	y_2 [mm]	z_3 [mm]	y_3 [mm]
120.5	116.5	115.7	57.7	57.8	988.9	1478.6	627.2	1952.2	522.2

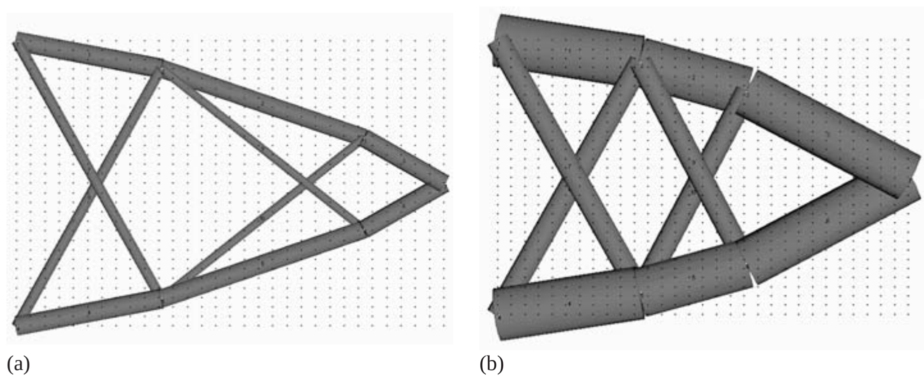

Figure 2. Truss structure. Optimal solutions obtained for a prescribed damage scenario where a maintenance intervention is carried out at the end of each design monitoring period (a) $T_D = 1$ year, and (b) $T_D = 50$ years, during the whole service lifetime $T_S = 100$ years (discount rate of money $v = 3\%$).

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Design, operation and maintenance of high speed railway bridges

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Design issues for dynamics of high speed railway bridges

José M. Goicolea, Felipe Gabaldón & Francisco Riquelme
Grupo de Mecánica Computacional,
Depto. de Mecánica de Medios Continuos y Teoría de Estructuras,
E.T.S. Ingenieros de Caminos, Universidad Politécnica de Madrid, Spain

ABSTRACT: This work discusses the influence of some issues arising from the dynamic resonant response on the design of railway bridges. These issues are related to dynamic analysis methods, composition of trains, and type of bridges. The evaluation of different response magnitudes is also addressed, showing that for calculation of bending moments or support reactions a much larger number of modes is needed in the model. Finally, a case study is presented with dynamic calculations for a large three-dimensional model of a special bridge.

1 RESONANCE AND METHODS FOR DYNAMIC ANALYSIS

The main concern in dynamic response of railway bridges is the risk of resonance from railway traffic loads. When such risk is relevant ($v > 200$ km/h) a dynamic analysis is mandatory.

An application of dynamic calculations using moving loads and simplified interaction models is shown in Figure 1a. A considerable reduction of vibration is obtained in short span bridges under resonance by using bridge-train interaction models. This may be explained considering that part of the energy from the vibration is be transmitted from the bridge to the vehicles. However, only a modest reduction is obtained for non-resonant speeds. Further, in longer spans or in continuous deck bridges the advantage gained by employing interaction models will generally be very small. This is exemplified in Figure 1b, showing results of sweeps of dynamic calculations for three

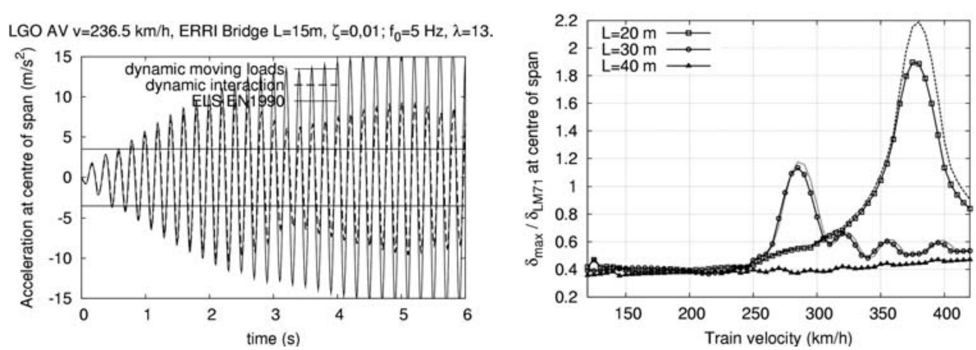

Figure 1. (*a*) Calculations for simply supported bridge from ERRI D214 (2002) ($L = 15$ m, $f_0 = 5$ Hz, $\rho = 15000$ kg/m, $\delta_{\mathrm{LM71}} = 11$ mm), with TALGO AV2 train, for resonant (236.5 km/h, bottom) speed, considering dynamic analysis with moving loads and with train-bridge interaction.
(*b*) Normalised envelope of dynamic effects (displacement) for ICE2 high-speed train between 120 and 420 km/h on simply supported bridges of different spans ($L = 20$ m, $f_0 = 4$ Hz, $\rho = 20000$ kg/m, $\delta_{\mathrm{LM71}} = 11.79$ mm, $L = 30$ m, $f_0 = 3$ Hz, $\rho = 25000$ kg/m, $\delta_{\mathrm{LM71}} = 15.07$ mm and $L = 40$ m, $f_0 = 3$ Hz, $\rho = 30000$ kg/m, $\delta_{\mathrm{LM71}} = 11.81$ mm). Dashed lines represent analysis with moving loads, solid lines with symbols models with interaction. Damping is $\zeta = 2\%$ in all cases.

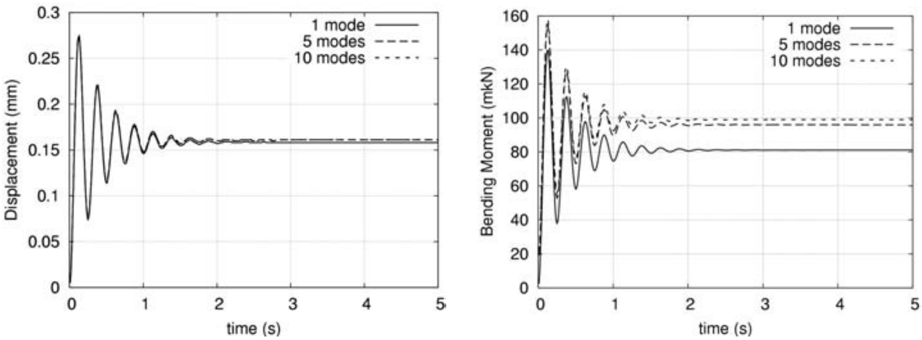

Figure 2. Response of simply supported bridge ($L = 20$ m, $f_0 = 4$ Hz, $\rho = 20000$ kg/m) under step load $P = 20$ kN, with damping $\zeta = 10\%$. Results for displacement and for bending moment at centre of span as a function of the number of modes considered in model [Goicolea et al. (2003)].

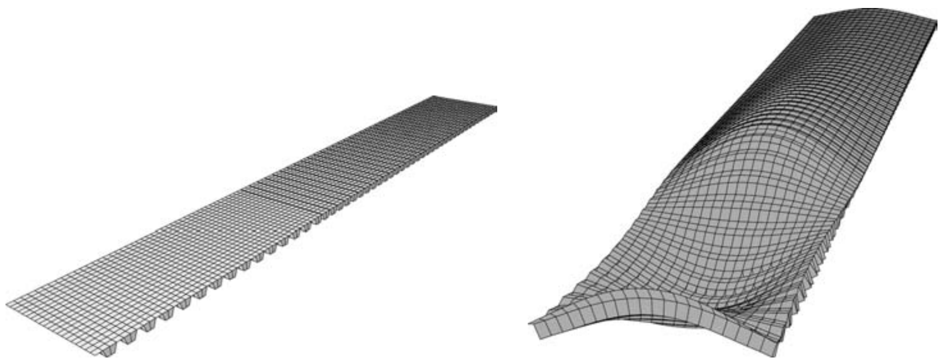

Figure 3. Finite element model of the "pergola" bridge and eigenmode #3, corresponding to frequency $f_3 = 4.51$ Hz. The model included 552 eigenmodes in order to consider all frequencies lower than 30 Hz, one of the modes is shown in Figure 3. The traffic actions correspond to the ten trains of the HSLM-A model. The calculations are performed for the range of velocities of 120–300 km/h every $\Delta v = 5$ km/h. The highest velocity is 20% higher than the design velocity $v_{\text{design}} = 250$ km/h. The impact coefficient Φ_{real} was evaluated from the basis of displacement amplitudes. Further checks were done for accelerations and other ELS magnitudes. A total of 370 calculations were performed for this sweep of velocities. In order to employ a reasonable lapse of time a cluster of 24 PENTIUM machines (2.6 GHz and 512 Mb RAM) was used, running in parallel processes.

bridges of different spans. As a consequence it is not generally considered necessary to perform dynamic analysis with interaction for design purposes.

In some situations specific dynamic response magnitudes are required directly from the analysis model. This arises when a more precise evaluation is required than what would be obtained by using an overall factor Φ_{real}, computed from say a displacement response. We call the attention to the fact that the model to be employed, for instance the number of modes considered in the integration, need not be the same for all cases. To illustrate this we present a model problem, a sudden step load P at the centre of a simply supported span. A closed form solution may be obtained for the response of each mode, obtaining the total magnitude as sum of a series. The results for a typical railway bridge are shown in Figure 2. For displacements at centre span only the first mode gives an excellent approximation; however, for the bending moment 10 modes must be considered for similar precision.

Finally, a case study of dynamic behaviour for a special bridge is presented. The mesh and one of the modes of vibration are shown in Figure 3. In the full paper results are shown for the impact coefficient Φ_{real} obtained for each velocity and each train, which are always lower than unity, indicating that for the range of velocities considered dynamic effects will not surpass those of the static LM71 train.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Fatigue verification for railway bridges including resonance effects due to high speed trains

M. Muncke, L. Bagayoko, E. Koch & S. Crail
Deutsche Bahn AG, Germany

ABSTRACT: Fatigue strength is for railway bridges in certain cases of equally great importance as the load carrying capacity. A bridge structure may fail due to fatigue without having reached its maximum design strength.

For railway bridges it is therefore required to check not only the ultimate limit state for static loads etc. but also the fatigue strength. This fatigue assessment has to be done for all bridges irrespective of the type of bridge and varies accordingly.

The calculation required in the design codes allow for the effects of the normal rail traffic at low or medium speeds. For high speed rail traffic resonance phenomena are possible and these need to be analysed more closely.

At present, design codes do not offer detailed rules regarding fatigue verification in the case of resonance. This paper presents a method as to the fatigue verification for resonance phenomena at high speed rail traffic.

Structural elements may fail under fatigue loads. Fatigue strength of structural members depend on many parameters such as the materials used, structural design details, temperature paths of welded structures, etc.

It is assumed that in theory with each passing train the structure is subject to some partial damage which accumulates in the course of time until the structure sustains serious damage. This so-called cumulative damage hypothesis forms the basis of the fatigue assessment in the Eurocodes.

The structural design of a railway bridge is usually based on Load Model 71. Effects due to rail service and track condition are allowed for by dynamic factors which are part of the design equations. The fatigue behaviour of a structure depends on the stresses relevant to the damage. For the damage assessment it is therefore necessary to have exact knowledge of the Real Trains and the traffic mix.

Because all these effects cannot be ascertained theoretically, fatigue strength can only be assessed by means of tests. This was already recognise by August Wöhler who carried out first systematic investigations into fatigue strength. For that reason extensive experimental investigations were carried out with the aim to determine for all common detail categories the so-called S-N-curves (Wöhler curves). They give for known upper and lower stresses (known stress ranges) the corresponding number of cycles up the failure. With the fatigue-strength-curves and a cumulative damage hypothesis (for example the Palmgren-Miner-Rule described below) a fatigue assessment is possible. A simple application of the S-N-curves (Wöhler curves) is only possible for simple periodic loading. For complex loading it is necessary to reduce the time history to a sequence of simple cyclic loadings from which a histogram of cyclic stress can be created where each stress level has a contribution to the cumulative damage.

The rainflow-counting method is the most popular technique used to convert a complex stress history into a histogram of stress ranges. Fatigue can be defined as progressive damage and each stress range can be associated with a partial damage. The Miner's-rule is the most frequently used and probably also the most criticised damage hypothesis. The shortcomings of the Miner's rule are well known. Nevertheless this simple rule is preferably used for almost all damage assessment calculations because of its simplicity. The counting method alone can lead to a different collective for the same applied stress which makes the comparison of results from different sources very difficult. Especial mention needs to be made of the scatter of the material properties, which is no longer recognises from the equation of the S-N-curves after the regression analysis. The classical approaches for fatigue assessment based on the S-N-curves uses as the initial parameter for actions the applied stress which is deterministically found for the intended service life. For the resistance the S-N-curves are derived from the simple cyclic tests. The method uses suitable transformations to facilitate a comparison between actions and member resistance.

A criterion for fatigue failure is the exceedance of the permissible loading cycles, which can be read from the S-N-curves for a known stress range. This criterion cannot be used directly for randomly occurring action effects as they exist under service conditions. Counting methods need to be devised, which extract from any loading spectrum the information relevant for fatigue, i.e. the stress amplitudes as well as the corresponding number of cycles. This information can be shown in a histogram or as cumulative frequency. In a second step an amount of damage is allocated to each loading cycles which can possibly show different stress amplitudes and mean stresses. Assumptions need to be made which form the basis of a cumulative damage hypothesis and which allow an estimate of the damage caused by the stress collective.

Resonance occurs, if the frequency of the applied loads equals the undampened natural vibration frequency of the bridge. For such cases the stress ranges must be calculated using a dynamic analysis. The simplified procedures in the design rules do not apply. A complete dynamic analysis according to EN 1991-2 is first required. With certain assumptions, fatigue verification according to the general rules (i.e. computation of the cumulated damage) can be performed. Such a procedure is briefly described in EN 1991-2. However, the parameters needed are not all given.

More information about fatigue assessment including resonance can be found in the design code of the Deutsche Bahn AG for railway bridges (Richtlinie 804). As mentioned above the fatigue assessment in such cases is extremely complex. For this reason substantial details are given in the "Richtlinie 804" for which cases additional calculation is not required: If these conditions are not fulfilled, fatigue verification according to the procedure described below is necessary. However, due to the probably considerably reduced fatigue life, it is recommended to change the bridge parameter or the operating speed.

The first step of the fatigue verification including resonance effects is to specify the fatigue loading, i.e. the trains and the corresponding speeds. The train types and the traffic mix should be the most realistic estimation of the current and future traffic. For verifications of ultimate limit states, it is required to consider speeds up to $1.2 \cdot$ Maximum Line speed. Since fatigue failure occurs in case of high numbers of cycles, fatigue verification may be carried out for speeds only up to $1.0 \cdot$ Maximum Line speed.

The second step is to compute the stress history $\sigma(t)$ for all train types. For this step, a dynamic analysis is required. The bridge parameters (damping, mass and stiffness) should be determine according to the design codes. Generally the trains are represented by series of moving point loads, neglecting the vehicle/structure mass interaction effects. The equations of motion can be solve using standard numerical procedures. Unlike the verifications at ultimate limit state, which require only extreme values (design values), the complete stress history must be considered for fatigue verification. Even the free vibration can be relevant.

The third step is the application of rainflow counting method (see chapter 2.3) to calculate the stress ranges and the corresponding number of cycles, taking into account the annual traffic volume and the service life.

In the last step the damage D_{Ed} is calculated using the Miner's rule (see chapter 2.3) and the S-N-curves defined in the design codes. The damage should satisfy the condition $D_{Ed} \leq 1{,}0$.

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Dynamic behavior of high speed railway bridges in interoperable lines

R. Delgado, R. Calçada, I. Faria, D. Ribeiro, J.R. Pinto & H. Figueiredo
Faculty of Engineering of the University of Porto, Porto, Portugal

ABSTRACT: The dynamic behavior of different types of bridges in interoperable lines, in which the Technical Specifications for Interoperability (TSI) are applicable, has been performed for both the actual high-speed real trains and the HSLM load schemes. The response of the bridges was assessed in terms of structural safety, traffic safety and passengers comfort, based on the most recent criteria established in the Eurocodes.

Four different types of bridges were analyzed, the first existing in a conventional line and the others part of high-speed railway lines: a metallic bowstring arch bridge, a prestressed concrete voided slab bridge, a prestressed concrete box girder bridge and a composite twin girder bridge.

The bowstring metallic arch bridge, the Alcácer do Sal bridge, consists of 8 simply supported spans, ranging from about 14 to 50 meters in a total of 291.25 m, and only one track. This bridge belongs to a conventional line that has been subject of a number of interventions for allowing the passage of new trains traveling at speeds higher than those to which it had been originally designed for. The main goal of the study was the evaluation of the dynamic effects resulting for the passage of the high-speed "Alfa Pendular" tilting train, in order to conclude about the performance of the structure up to the maximum speed reached by these trains, which is 220 km/h, complemented with the investigating of the dynamic behavior of the bridge for the passage of other trains, currently in use in the European high speed railway network. The dynamic analyses were performed for the passage of the high-speed trains VIRGIN, TALGO, TGV, EUROSTAR, THALYS, ICE2 and ETR-Y, with a range of speeds from 140 km/h to 420 km/h (1.2×350 km/h), and, for the "Alfa Pendular" up to a speed of 265 km/h (1.2×220 km/h). The maximum displacement, equal to 2.65 cm, was obtained for the TGV train at a speed of 365 km/h, significantly lower than that resulting from the application of the loading model LM71, majored by the respective dynamic factor, equal to 3.50 cm (3.35 cm $\times$ 1.04).

The prestressed concrete voided slab bridge, in the Córdoba-Málaga high-speed railway, has a total length of 415.0 m, comprising a continuous deck with seventeen spans. From the structural safety point of view table it was concluded that the bending moments obtained using the dynamic analysis are considerably lower than those resulting from the application of the LM71 and SW/0 load models. The relation between these values is of 2.8 for the 1st span, 2.9 for the 2nd span and 4.3 over the support element.

The prestressed concrete box girder bridge, one of the most frequently used solutions for bridges with spans around 40 m to 60 m, included in the Córdoba-Málaga high-speed railway line, has simply supported spans of 46 m. The dynamic analysis enables to concluded that the highest effects were obtained for the passage of the VIRGIN train at a speed of 270 km/h, the resonance speed corresponding to the excitation of the structure with a frequency of the first mode shape of the bridge. The maximum values of displacement and acceleration were obtained for the passage of the different HSLM-A loading schemes, with values slightly higher than the ones obtained with real trains. It was, also, concluded that, in general, the response is dominated by the contribution of the first mode shape. For a speed of 270 km/h, to which resonance occurs in the deck, the contribution of the first mode shape represents about 97% of the global response, both in terms of displacements and accelerations.

The composite twin girder deck, used in continuous schemes with span lengths ranging from 40 m up to 65 m, a very competitive solution in France, consists of a 333 m long composite twin

girder deck located on the French TGV Nord line. The bridge is continuous over its entire length, comprising 7 intermediate spans of 40 m and 2 end spans of 28 m and 25 m. From the analysis performed it can be concluded that the effects induced by the HSLMs are greater than those produced by the real European high speed trains, thus demonstrating the ability of the new models to reproduce the effects of high speed traffic on this type of structures. The highest vertical acceleration levels were registered on the two end spans, reaching a maximum value of 4.29 m/s^2 on the 9th span for a speed of 405 km/h (HSLM-A1). This peak values are associated with local modes of vertical bending and torsion.

From the track safety point of view it was concluded, in the bowstring metallic arch bridge, that the limit of 5 m/s^2 ($\approx$0.5 g) was exceeded at diverse speeds. For the other three bridges the vertical accelerations of the deck obtained from the analysis of the Real Trains are lower than the limit value of 0.35 g ($\approx$3.5 m/s^2) in all situations, except for the composite twin girder deck where this value is higher on the 5th span with HSLM-A3 at a speed of 330 km/h (3.75 m/s^2) and on the 9th span for speeds over 400 km/h (4.29 m/s^2).

For the bowstring metallic arch bridge it was observed that the level of comfort would be very good for speeds up to 280 km/h, and good for higher speeds.

Very good level of comfort for the passengers was obtained for all the others bridges.

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Design and construction of structures for high-speed railway lines

P. Ramondenc
SNCF Engineering Department

ABSTRACT: The purpose of this file is to describe how the SNCF approaches the design and construction of large bridges for high-speed railway lines. This approach based on close cooperation with other partners, such as architects, landscapists, subcontractors and design offices has enabled the SNCF to successfully carry out major projects for high-speed railway lines in France.

All bridge projects, whether for large viaducts or smaller ones, begin with extremely thorough architectural studies and also environmental impact studies in order to ensure a consistent design while also meeting the functional and technical specifications for high-speed railway lines. This complementarity of architectural and environmental concerns has enhanced the overall quality of the projects.

The reliance on innovative techniques and implementation of a quality policy: quality design, quality architectural conception and quality construction on the one hand and close collaboration between all of the project partners on the other hand (the first level customer, the prime contractor and particularly, the architects, the engineers and contractors) makes it possible to build structures which are outstanding in terms of their beauty, technical sophistication and performance but in particular in terms of their reliability and robustness.

The aim of this article, among others, is to describe procedures which should be implemented during design and construction phases for large bridges for high-speed lines. These procedures and the close collaboration referred to above make it possible to successfully complete major projects.

The bridges for the new lines should undergo very thorough architectural studies in order to ensure that they are consistent with all of the lines while meeting the functional and technical requirements for high-speed railways such as suitable safety measures for specific risks.

A well constructed bridge should give the impression to anybody looking at it, that it is perfectly suited for the loads it will have to bear. This does not require any particular knowledge of the characteristics of materials but can be seen at a glance.

A railway bridge should support heavy loads by opposing a certain resistance and rigidity. If so designed, this will be obvious to a casual observer while calculations will also demonstrate that it is suitable. On the other hand, one might have trouble imagining a train crossing a road bridge or walkway and the calculations will demonstrate this as well.

The same is true for high-speed technology though in a more striking way. We just have to remember how rails are tapped to detect defects to understand that when speed increases, vibrations will occur and that they have to be dealt with. When a part vibrates, often not much is needed to stop the movements. The same is true for bridges: the higher the speed of the train which crosses them, the more they are likely to vibrate. Simple devices can be used to damp these vibrations. The engineer's art involves designing them and implementing them.

To do this, he first has to understand the structure and its function. Until now, all of the calculations done were static calculations with the bridges being inert. Since high-speed trains make the bridges vibrate, dynamic calculations have become necessary. Engineers now have to imagine not only the moving bridge but also have to calculate this in order to better understand the structure than in the past. Perhaps one of the consequences of high-speed trains is to help engineers better understand their structures! In order to do this, these structures have to be stripped of all unnecessary additions so that they are as natural and as simple as possible: high-speed technology has no place for complicated or artificial elements.

High-speed as with all technical progress, thus obliges engineers to become better builders. That mastery should not be equated with complication but with understanding and simplifying is a basic necessity of which one has to be aware, especially at a time in which computers give the impression of controlling matter down to the finest details; computing can in fact only help improve engineering but is not a guarantee in itself.

Those who built the cathedrals had no other means at their disposal than communing with the forces and the materials, experiencing and understanding them. We sometimes tend to forget this. If we too often forget the need to understand our bridges then they way they react when crossed by high-speed trains will quickly bring us to our senses. High-speed reveals the behaviour of bridges and they have to be designed to take this into account.

Application of structural system identification methods

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Structural health monitoring using dynamic responses with regularized autoregressive model

J.S. Kang & H.S. Lee
School of Civil, Urban and Geosystem Engineering, Seoul National University, Seoul, Korea

Over the last few decades, there has been a significant increase in the health monitoring and safety management field of the complex structure. The primary goal of the structural health monitoring is to find changes of system parameters and to decide its soundness at earliest possible stage. There are two categories in structural health monitoring and damage assessment whether structural model, such as stiffness, damping and mass information exist or not. One is model based scheme and the other is non-model based scheme. In model based scheme, system parameters are estimated by inverse analysis based on the sensitivity method from a mathematical model. In non-model based scheme, structural soundness is evaluated by pattern recognition and statistical approach from only measured signals without a structural model.

Model based system identification problem is a type of inverse problems, which are usually ill-posed problem. An ill-posed problem is characterized by the non-uniqueness, non-continuous and instability of solutions. Various regularization techniques have been developed to overcome this ill-posedness of inverse problem. In spite of ill-posedness can be alleviated by regularization techniques successfully, model-based system identification schemes are not applied in real situation because of modeling error that difference between mathematical model and real structural model. Recently, a lot of structural health monitoring researches with statistical pattern recognition using purely measured signals has been attempted in the center of Los Alamos national laboratory in USA. Autoregressive model is widely used in time series pattern analysis (Box, 1994).

The sequence of non-model based structural health monitoring system is divided into six steps that Data acquisition, prediction modeling, feature extraction, control distribution construction, monitoring and decision making. Measured signals are obtained from sensors and a prediction model is made of the autoregressive model. Coefficients of the autoregressive model and residual errors are estimated by a prediction model. Statistical treatments of obtained residual errors must be done for more reliable structural health monitoring. Finally, the decision making of soundness of considered structure in real time by monitoring residual errors continuously will be performed.

Various algorithms for structural health monitoring using static or dynamic responses are proposed. But the main problem of structural health monitoring system is how to handle noises, whereas measured signals contain a mix of information related to both the damage in the structure and the perturbations due to the environment. A new structural health monitoring algorithm with time window technique is employed. In time window technique, the residual errors are predicted sequentially within a finite time period which called time window. The time window advances forward at each time step to predict residual errors repeatedly. Perturbations of environment are commonly changed gradually during long time period and time window size is relatively very smaller than environmental perturbation period so it is assumed that perturbation of environment can be neglected within the time window.

Decision whether the considered structure is sound or not using residual errors in every time step is also very important. Extreme value distribution (Castillo, 1988) is utilized for making decision boundary of soundness of the target structure. Extreme value distribution is utilized to detect outliers because damage information almost lie on the tail of distribution and extreme value distribution is well established in tail distribution. A generalized extreme value distribution(GEV)

(Park et al, 2005) which unify three known extreme value distributions, Gumbel, Weibull and Flechet is utilized for simplicity.

The validity and accuracy of the proposed algorithm is demonstrated through a numerical simulation studies on a two-span truss bridge. The numerically generated acceleration data with noise under Kobe earthquake ground acceleration are utilized as measured signals for the numerical simulation example.

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Estimation of stiffness and mass properties from measured modal information

S.-B. Shin
Dept. of Civil Eng., Inha University, Incheon, Korea

S.M. Lee
Dept. of Civil Eng., Dong-A University, Busan, Korea

A frequency-domain system identification method is proposed to estimate both stiffness and mass parameters using measured modal information, which is different from usual system identification algorithms estimating stiffness parameters only. The structural parameters are estimated by minimizing the output error between measured and computed modal displacements at the limited degrees-of-freedom. The output error vector $\mathbf{e}_{mi}(\mathbf{x},\mathbf{y})(N_m \times 1)$ of mode i is defined by Equation 1.

$$\mathbf{e}_{mi}(\mathbf{x},\mathbf{y}) = \mathbf{B}_m\boldsymbol{\phi}_i(\mathbf{x},\mathbf{y}) - \boldsymbol{\phi}_{mi} = [\lambda_{mi}\mathbf{B}_m\Xi_m^{-1}(\mathbf{x},\mathbf{y})\mathbf{M}_m(\mathbf{y}) - \mathbf{I}_m]\boldsymbol{\phi}_{mi} \tag{1}$$

where $\mathbf{B}_m(N_m \times N)$ = Boolean matrix composed of 0's and 1's to select measured degrees-of-freedom, $\Xi_m(\mathbf{x},\mathbf{y}) = \mathbf{K}(\mathbf{x}) - \lambda_{mi}[\,\mathbf{0}_m\ \ \mathbf{M}_u(\mathbf{y})\,]$, $\mathbf{I}_m(N_m \times N_m)$ = identity matrix, $\boldsymbol{\phi}_{mi}(N_m \times 1)$, $\boldsymbol{\phi}_{ui}(N_u \times 1)$ = measured and unmeasured parts of the ith modal displacement vector, and N_m, N_u = number of measured and unmeasured degrees-of-freedom, respectively. A nonlinear constrained optimization problem was formulated and solved for optimal parameters.

Simulation studies were carried with a frame structure to examine the effects of mass error on the model verification and also on damage assessment. When mass properties were fixed with misjudged values and only the stiffness parameters were estimated, mode shapes of some high modes deviated from exact shapes as shown in Figure 1. When both mass and stiffness parameters were estimated, all mode shapes could be matched well.

To investigate the influence of mass error on damage assessment, a frame structure as shown in Figure 2 has been studied. For the simulation study, damage was imposed as a reduction of sectional height at the member indicated in the figure. It was assumed that accelerations were measured to the horizontal and rotational degrees-of-freedom at all nodes. When damage is relatively mild with 10% reduction in the sectional height, the identification from the proposed algorithm is superior as shown in Figure 3. When only the stiffness parameters were identified with fixed mass error, mild damage could not be identified as shown in the left-hand figure of Figure 3.

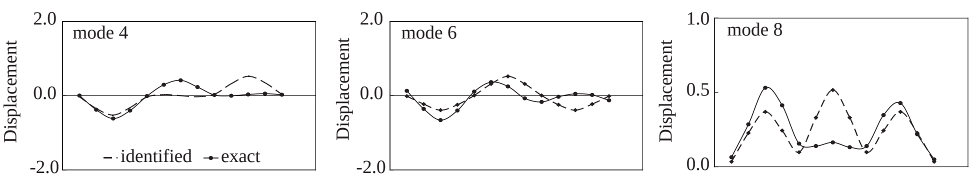

Figure 1. Comparison of mode shapes of mode 4, 6, 8 when mass properties were fixed as misjudged.

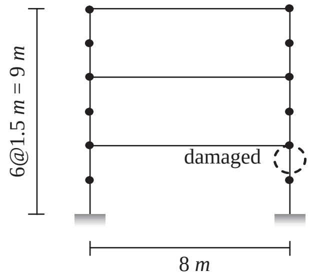

Figure 2. Sample frame for damage assessment.

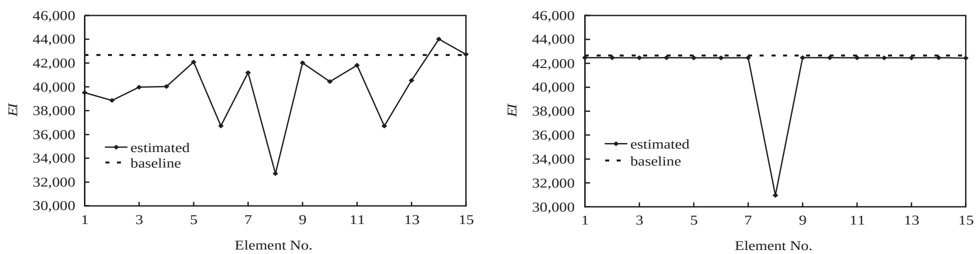

Figure 3. Identified *EI*'s from the conventional and the proposed algorithms.

From the studies on model verification and damage assessment, the importance of identifying mass in addition to stiffness parameters could be well demonstrated.

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Assessment of the dynamic displacements using acceleration data measured on bridge superstructures

B.S. Jung
Department of Civil Engineering, Dongeui University, Korea

N.S. Kim
Department of Civil Engineering, Pusan National University, Korea

1 INTRODUCTION

The displacement response is one of the important parameters to determine the vibration characteristics and the state evaluation of a structure. When studying the structural integrity of a large system, it is much easier to measure accelerations than displacements. However, many engineering standards are based on displacements that are proportional to the stresses in an elastic structure. The displacement can be directly used to determine the accumulated damage. The displacement response is measured by the use of LVDTs and similar direct displacement measuring devices, which require a fixed reference to work properly. The use of optical transducers to measure displacements may be expensive. After measuring a signal of acceleration that is relatively convenient to measure and does not require a set of fixed references on the necessary point of structure, a digital method to double integrate accelerometer data to measure displacements is attempted, which is based on a dynamic loading test of structure.

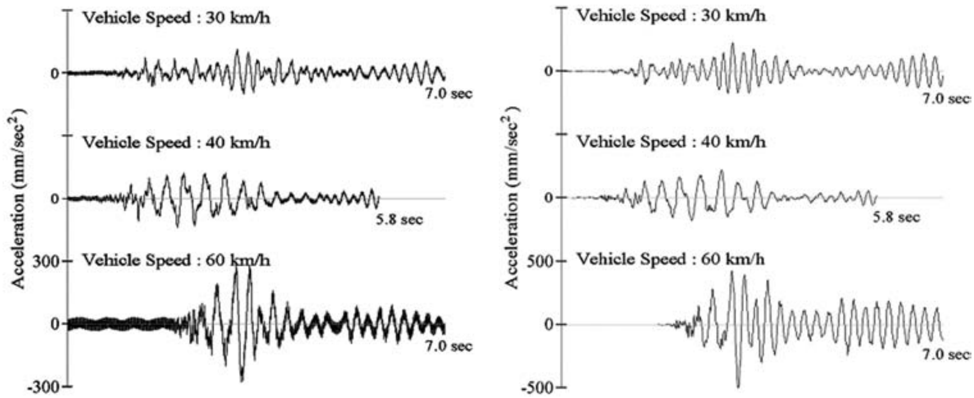
In this paper, the theoretical aspects of the dynamic response conversion algorithm, which is able to remove information about the initial condition of velocity and displacement responses, are described. The theory is based on reformulation of the structural system matrix that is induced by the process of a time domain modal vibration test technique. Digital integration to measure displacements is accomplished, indirectly, by defining the change amounts and the transformed responses in the time and frequency domain respectively. The method presents a solution for the initial value problem under the moving load. In this algorithm, the displacement response can be obtained from the measured acceleration records only. The feasibility for physical application of the proposed technique is tested by the example problems, using the superstructure of the real bridge, under several cases of moving load. The results are compared to the actually measured displacements.

2 RESPONSE CONVERSION

The dynamic response conversion technique is based on reformulation of the ordinary differential equation of motion into state variable form. The transformed velocity response and the transformed displacement response can be obtained from the experimental acceleration records by integration, without requiring the knowledge of the initial velocity $\dot{y}(t_1)$ and displacement $y(t_1)$ information. The transformed response curves are only affected by the initial conditions of the acceleration responses $\ddot{y}(t_1)$ and $\ddot{y}(t_2)$.

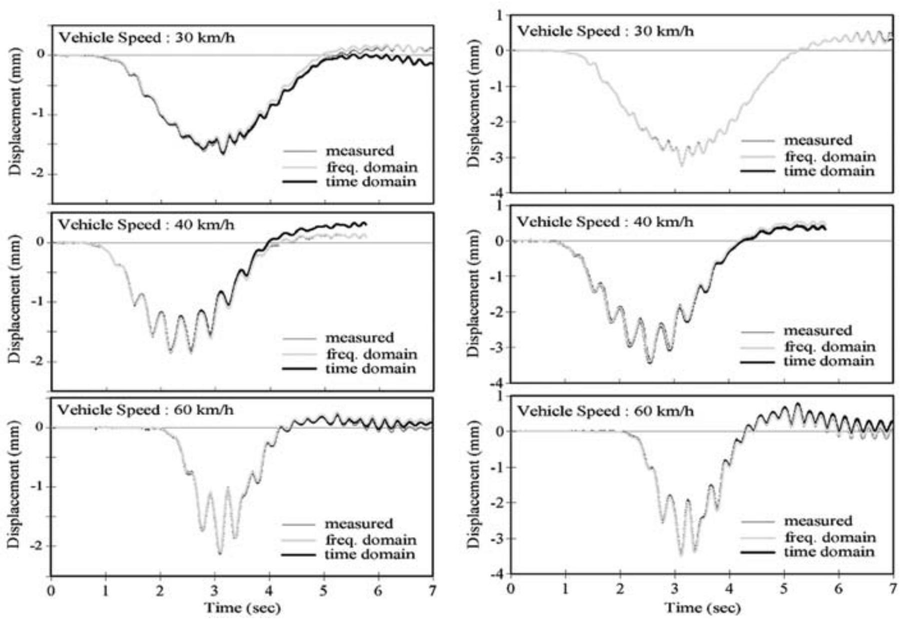
3 EXPERIMENTAL RESULTS

See Figs. 1 & 2.



(a) Girder 1 (b) Girder 3

Figure 1. Acceleration responses.



(a) Girder 1 (b) Girder 3

Figure 2. Identified displacement responses.

4 CONCLUSIONS

An improved formulation using the transformed response is proposed for the dynamic response conversion algorithm. This algorithm is based on the iterative method. In the formulation of the indirect integration scheme, the change amounts and the transformed responses, in relation to three kinds of physical responses, are defined. The transformed response can be obtained from the measured acceleration records, without requiring the knowledge of the initial velocity and displacement information. The relationship between the transformed response and the actual response is derived using the structural system matrix, which is induced by the process of a time domain modal vibration test technique.

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Evaluation of load carrying capacity of bridge based on ambient acceleration measurements

Chung-Bang Yun & Soojin Cho
Dept. of Civil and Environmental Engineering, Korea Advanced Institute of Science and Technology Daejeon, Korea

Jin-Hak Yi
Korea Ocean Research and Development Institute, Ansan, Kyunggi, Korea

Chang-Geun Lee & Won-Tae Lee
Structural Division, Korea Highway Cooperation, Hwaseong, Kyunggi, Korea

ABSTRACT: The load carrying capacities of exciting bridges need to be properly assessed for the safe operation and efficient maintenance. The conventional methods require load tests with controlled vehicles to measure the static and dynamic responses, such as deflections or strains under known loading conditions. However, full or partial block of traffic during the tests may cause not only inconvenience to the traffic but also increase of logistic cost and time. To overcome these difficulties, an alternative method is proposed using ambient acceleration data measured without traffic control in this study.

The load carrying capacity of a bridge (P) is commonly evaluated by combining the design live load (P_r), the rating factor (RF), the deflection (or stress) correction factor (K_δ (or K_ε)), the impact correction factor (K_i), and the correction factors for traffic volume and pavement roughness (K_t, K_r) as

$$\mathrm{P} = P_r \times RF \times K_\delta \, (or \, \mathrm{K}_\varepsilon) \times \mathrm{K}_i \times \mathrm{K}_t \times \mathrm{K}_r$$

where P_r is a given design value; RF is determined by structural analysis using the initial FE model of a bridge; and K_t and K_r are empirically estimated by the structural engineer. On the other hand, two correction factors, K_δ (or K_ε) and K_i, are generally evaluated by load tests on the bridge. Static load tests are traditionally carried out to obtain the deflection correction factor (K_δ) using loaded trucks, and vehicle running tests are carried out to estimate the impact correction factor (K_i). The RF, which is the ratio of the live load resistance and the design live load, is evaluated by the allowable stress design (ASD) method for steel members and the load and resistance factor design (LRFD) method for concrete members.

The proposed method for the deflection correction factor consists of the following procedures: (1) ambient acceleration measurements on the deck due to wind or the traffics on the adjacent bridge, (2) estimation of the modal properties using the stochastic subspace identification method, (3) updating of the initial FE model based on the modal properties, (4) calculating of the static deflection using the updated FE model, and (5) evaluation of the deflection correction factor by comparing the deflection using the initial and the updated FE models. The present method replaces the static deflection measurements of the bridge deck under known loading conditions, which requires expensive deflection measurement devices such as advanced laser devices and traffic control during the tests.

The present method for the impact correction factor utilized the pseudo-deflections obtained by double integration of the measured acceleration records, so that dynamic loading tests using a moving loaded truck may be avoided. For stable integration, only the part of the acceleration record

corresponding to the duration with the vehicle on the bridge was considered. The shift components were eliminated using a baseline correction procedure during the double integration.

For validation of the proposed method, a series of ambient vibration tests and load tests with various truck weights were carried out on a steel plate-girder bridge on an expressway in Korea. The tests were performed in 3 different seasons: August 2004, December 2004, and July 2005. Conventional load tests were also performed, which were composed of quasi-static load tests with a speed of 3 km/h and vehicle running tests with a speed of 50 km/h.

At first, the deflection correction factors were estimated by the proposed method. The results by the proposed method showed big discrepancy compared with those obtained by the conventional method with quasi-static loads and contact type displacement transducers with connecting wires (OU displacement transducers). However, the results by the proposed method are found to be very similar to those by the conventional method using the data from a laser vibrometer, which are proven to be more accurate than those by the OU gages during the field validation tests. The estimated deflection correction factor by the proposed method shows good consistency regardless of the test season. The impact factors were also estimated using the pseudo-deflections obtained from the acceleration data by double integration. The results by the proposed method are very close to those by the conventional method using the measured dynamic deflections, so as the impact correction factors estimated using the impact factors. Using the above correction factors, the load carrying capacities of the example bridge were evaluated.

The results of a series of field tests on a bridge may be summarized as

(1) The proposed method gives very consistent results for the load carrying capacity regardless of the test season, and the results are reasonably close to those by the conventional method.
(2) The accuracy of the deflection sensor is very critical to the conventional method. The conventional OU gage did not provide accurate deflections of the bridge girder, while the laser vibrometer gave good results.
(3) The deflection correction factors by the proposed method using the updated FE model are very close to those obtained by the conventional method and the deflection using the laser vibrometer.
(4) The impact correction factors by the proposed method using the pseudo-deflection are close to those by the conventional method.
(5) Further tests are needed for validation under traffic conditions.

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System identification scheme using genetic algorithm for damage classification in beam-type structures

J.T. Kim, J.H. Park & D.S. Hong
Pukyong National University, Busan, Korea

W.J. Kim
Kyungpook National University, Daegu, Korea

During the past several decades, a significant amount of research has been conducted in the area of damage detection via changes in the modal response of a structure. Also, research for damage detection in structures using statistical analysis is very active. Two classes of statistical models that have been presented are unsupervised learning and supervised learning. Unsupervised learning approach is limited to level 1 damage detection (i.e. alarming the occurrence of damage) or level 2 damage detection (i.e. identifying the location of damage), but supervised learning approach can be taken to move forward to higher level damage identification for classifying, locating, and quantifying the severity of damage.

The genetic algorithm, which can identify damage-driven effects from others, is one of supervised learning approaches. With modal features as the input, the genetic algorithm can be used to discriminate the modal features for damage detection. Mares & Surace (1996) introduced the concept of residual force vector, and applied a genetic algorithm including rank-based selection to identify damages in a truss structure. Routolo & Surace (1997) formulated objective functions with natural frequencies, mode shapes, and modal curvatures, and applied a genetic algorithm including rank-based selection to identify damages in a cantilever beam structure. Friswell et al. (1998) used an objective function that combines weighted natural frequencies with mode shapes, and applied a genetic algorithm to identify damages in a cantilever beam structure. Hao & Xia (2002) applied real-coded genetic algorithm based on a scaled objective function that combines weighted natural frequencies with mode shapes in order to identify damages in a cantilever beam and portal frame structure. Rao et al. (2004) formulated an objective function used normalized residual force vectors and applied a genetic algorithm with tournament selection to identify damages in truss, cantilever beam and portal frame structure. Au et al. (2003) applied a micro genetic algorithm for damage severity. Raich & Liszkai (2004) formulated an objective function using frequency response function (FRF) and applied implicit redundant representation genetic algorithm for damage detection.

When a genetic algorithm is used for damage detection, some essential particulars must be considered. Firstly, effectiveness of the genetic algorithm must be considered. The conventional genetic algorithms need a mount of calculations to search global optimum solution. In the first few generations, if a few highly fit individuals are generated, the genetic process can lead to premature convergence at wrong point. Secondly, the selection of modal features is very important to discriminate damage states. Generally natural frequency changes can not differentiate the damages in symmetric locations, and natural frequencies are not much sensitive to structural damage but affected by temperature, particularly for large structures. Also, mode shapes are affected by environmental or operational noises, and the choice of sensor locations has critical impact on the accuracy of damage detection results.

In this paper, a modal-strain-energy-based genetic algorithm for vibration-based damage detection in beam-type structures is proposed. The following approaches are used to achieve the goal. First, two objective functions that use the differences between the numerical model and measurable modal features are formulated as follow:

$$F_1(\boldsymbol{\alpha}) = f_\omega(\boldsymbol{\alpha}) + f_\phi(\boldsymbol{\alpha}) \tag{1}$$

$$F_2(\boldsymbol{\alpha}) = f_\omega(\boldsymbol{\alpha}) + f_\Theta(\boldsymbol{\alpha}) \tag{2}$$

where $f_\omega(\boldsymbol{\alpha})$ = a function for natural frequencies; $f_\phi(\boldsymbol{\alpha})$ = a function for mode shapes, and $f_\Theta(\boldsymbol{\alpha})$ = a function for modal strain energies. $F_1(\boldsymbol{\alpha})$ is a typical function combining natural frequencies with mode shapes; while $F_2(\boldsymbol{\alpha})$ is a newly proposed function combining natural frequencies with modal strain energies by Kim et al. (2003). Next, the feasibility of the proposed algorithm is experimentally verified for several damage scenarios of free-free beams for which changes in natural frequencies and mode shapes were measured for the first four bending modes before and after damage. A finite element model was also generated to identify the modal parameters of the test structure. A micro genetic algorithm which treats very small population efficiently was used to discriminate the measured modal features that represent either the damage states or the pristine state of the test structures.

Two major results are observed from the study. First, the optimal weights of the objective functions were decided as 1.0, 0.1, and 0.1 for natural frequency, mode shape, and modal strain energy, respectively. Next, the accuracy of damage detection using the proposed method was higher than that of the typical method. Using the typical method, damage near the mid-span of the test beam was detected but damage near the edge was not detected. Using the proposed method, all damages were detected with good accuracy.

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Damage assessment of bridge superstructure using moving load tests

Hyeong-Jin Lee
Dept. of Civil Engr, Changwon National University, Changwon, Korea

Soo-Bong Shin
Dept. of Civil Engr, Inha University

Tae-Won Kang
Korea Maintenance & Contraol Co., Ltd, Seoul, Korea

ABSTRACT: Safety evaluation and repair/rehabilitation of existing bridge structures due to environmental and traffic loads become increasingly important according to the rapid increase of the number of the damaged or deteriorated structures. System identification method can be one of the important tools for the safety evaluation. System identification method has been developed to assess the physical damages directly in the viewpoints of mechanical behavior instead of microscopic damages such as cracks, chemical deteriorations like in NDT (non-destructive test). This method has many potential merits. First, it can localize damages from wide range measurements of structures. Works for more detail investigation can be reduced. Second, the identified quantities can be usually directly used in structural analysis. This means that many applications are possible as in structural analysis.

However, system identification method is not practical enough yet. There are many reasons. Typically, the measurement from the infrastructure such as bridges doesn't have enough the number of measurement locations. The measured quantities such as displacements, acceleration and strain don't have suitable sensitivity to damages. Usually, displacements and acceleration are very insensitive to damages. The strain is too sensitive. This means that the measured signal is meaningless or too noisy. In addition, many signal processing algorithms to de-noise measured quantities are not suitable to process the measured signals from infrastructures because those signal are usually nonstationary and contains very low frequency noises. In addition, System Identification algorithms also have unstable characteristics because those are inverse problems in mathematical viewpoint.

In this paper, a method to extend the number of measurement DOFs was attempted to develop more simple and practical system identification methods by introducing reciprocal theorem and moving load test. The extension of measurements information might be very essential in system identification. Until now, most researches have been focused on more complex methods such as the development of new measurement devices, new algorithm and adoption of new signal processing techniques etc. However, more simplified method can be also a good approach to develop more practical system identification methods. The proposed method is a damage assessment method of bridge superstructure using moving load tests to overcome the lack of measurement information, particularly the number of measurement locations. The loads on bridge are quite unique because those are moving load. This means that we can easily extend measurements DOFs by applying the reciprocal theorem. One point measurement due to moving load can be converted to measurements at many points due to one load. In this way, the number of measurements can be drastically extended even in the limited number of actual measurement. Furthermore, This feature makes many simple system identification algorithms applicable. So, unstable problems in system identification problem can be also solved. This kind of idea is very simple and well known for many researchers. However, the effectiveness of this idea is quite underestimated, until now. So, the purpose of this paper is to remind many researchers about the effectiveness of this idea particularly in identification of bridge structure.

Long-term signal analysis for the existing bridge health monitoring systems

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Statistical time series analysis of long-term monitoring results of a cable-stayed bridge

J. Lee
Research Institute of Industrial Science and Technology (RIST), Hwasung, Korea

S.-P. Chang, H. Kim
Seoul National University, Seoul, Korea

J.-G. Yoon
Daelim Industrial Co. LTD., Seoul, Korea

1 INTRODUCTION

Recent advances in sensing technologies make it feasible and practical to instrument sensors to high expenditure civil infrastructure such as bridges for safety monitoring during construction as well as for long-term condition assessment. This paper presents an investigation of the structural response acquired through the structural health monitoring system installed in the Seohae Bridge (Cable-stayed Bridge) in South Korea. Preliminary analysis of the measured data revealed that the responses of the bridge are mainly affected by temperature variation, thus influence of the temperature is closely investigated using ARX model, statistical time series analysis method. Fourteen channels of temperature data are utilized as input and selected static responses such as vertical displacement of main span, movement of expansion joint, or tilting angle of a pylon are utilized as output. Number of input channels and previous data length included in the model is varied to determine optimum structure of ARX model. Feasibility of ARX model for analysis of bridge monitoring data has been proved, and the underlying relationship between the temperature and the structural response could be explicitly modeled with this approach.

2 ANALYSIS OF MONITORING DATA

Monitoring data accumulated for 2 years from Aug. 2002 to July 2004 are analyzed. Since test operation of the monitoring system was performed about one year from the opening of the bridge in Nov. 2000, the structure and monitoring system can be regarded as stabilized during the analyzed period.

During the preliminary analysis procedure, overall shapes of measured signals are examined, and linear regression is performed to ascertain influence of thermal effect. Then, ARX models are employed to identify the underlying relationships between responses and temperatures.

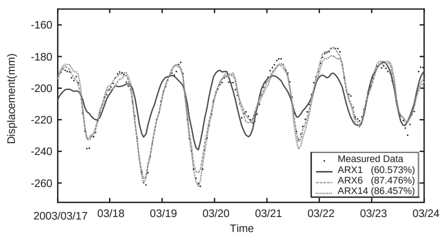
The two years of analyzed period is divided into two 1-year's sub-periods and the data of the first period is utilized as basis data to be compared with the data acquired after this period. Thus Seohae bridge is assumed to be in sound condition during the first period, and the data in this period are utilized as evaluation group for the construction of ARX model.

In order to determine appropriate number of temperature channels, three kinds of ARX model are constructed with different number of input channels. The temperature set of each model is listed in Table 1.

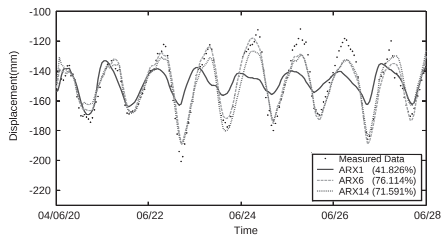
According to the output (n_a) and input (n_b) order variation from 4 (hrs.) to 24 (hrs.) with step size 4 (hrs.), three ARX models (ARX1, ARX6, ARX14) are constructed and the simulated responses are compared. As a quantitative index that can be utilized for performance comparison, 'Fit' factor

Table 1. Number of Thermometer Channels According to Installed Location.

Model ID	Air	Pylon	Girder	Cable	Total
ARX1	1	–	–	–	1
ARX6	1	1	3	1	6
ARX14	1	5	6	2	14



(a) Evaluation Group No.10 – Vertical Displace ment of Main Span



(b) Validation Group No.31 – Vertical Displace ment of Main Span

Figure 1. Comparison between Measured and Calculated Signal.

in Eq. (3) is employed

$$Fit(\%) = 100 \times \left[1 - norm(\hat{y} - y) / norm(y - \bar{y})\right] \tag{3}$$

in which the $norm(y)$ is the second order norm of vector y, $\hat{y}$ is the estimated value of true y, and $\bar{y}$ is the mean value of y. '$Fit(\%)$' equals 100 only if the estimated results are exactly the same as the measured data, and may vary to negative infinity when the error becomes larger.

The results show that higher order gives higher Fit values, but once the order reached up to 16 hours', most of the model's performances become stable. Thus, it can be said that 16 hours' I/O order is optimal for Seohae bridge, and that the response of this bridge is affected by previous 16 hours' temperature variation.

For vertical displacement of main span, the measured data and the simulated data of ARX models are plotted in Figure 1. As shown in this figure, simulation result of ARX1 shows lower performance. But simulation results of ARX6 and ARX14 both trace measured data very closely and show similar performance. This result means that there exist an appropriate number and location of temperature sensors to express structural behavior, and adding number of sensors does not always show performance increment.

3 CONCLUSIONS AND FURTHER RESEARCH

As one of the environmental effects on structural responses, this study investigated the influence of temperatures, and following conclusions are drawn.

– The behavior of the bridge essentially governed by yearly and daily fluctuations of the temperature, so thermal effects should be considered in analyzing field measured data.
– 16 hours' I/O order is optimal for Seohae bridge, and thus the response of this structure is affected by previous 16 hours' temperature variation.
– ARX models with six and fourteen input channels show similarly good performance than the ARX model with single input channel. Thus, determination of appropriate number and location of temperature sensors is more important than increasing the number of including sensors.
– No structural change has been detected during the analyzed period.

In further research, Multiple Input Multiple Output (MIMO) ARX model will be employed for modeling of the correlations between different types of structural responses and the results of the two different methods will be compared.

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Signal analysis from a long-term bridge monitoring system in a three dimensional self-anchored suspension bridge

S. Kim
Seoul National University of Technology, Seoul, Korea

J. Lee
Research Institute of Industrial Science and Technology (RIST), Kyonggi-do, Korea

I.-H. Bae
New Airport Hiway, Co., Inchon, Korea

1 INTRODUCTION

Automatic measurement of instrumented civil engineering structures is now widely applied for behavior monitoring during construction in field as well as long-term monitoring for lifetime assessment of bridge structures. This paper presents schematically the monitoring system installed in Yongjong Bridge, a self-anchored suspension bridge located in the expressway linking Seoul and Incheon International Airport. Since (1) appropriate instrumentation, (2) reliable signal processing and (3) intelligent information processing constitute the major features to be considered for deploying proper monitoring system, corresponding general guidelines and suggestions are also proposed.

A representative example of results that can be acquired through structural health monitoring system is presented by means of data measured during 2 years after the opening of the bridge. Newly equipped railway system on the existing bridge results in change of dead load, consequently dynamic characteristics have also been changed. This result can be detected by the monitoring system during and after railway construction.

2 ANALYSIS OF MONITORING DATA

The monitoring system has been completed in 2001 and a huge volume of signals has been acquired up to date. These signals were carefully analyzed for verifying the system performance and

Figure 1. Overview of Yongjong Bridge.

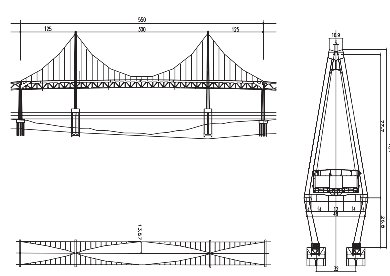

Figure 2. Typical dimension (M).

implementing further use for bridge health assessment. During the system stabilization period, signals showed regular pattern of fluctuation along with the daily and seasonal temperature changes. Some typical signal patterns during before and after railway construction are described.

Hanger tension forces are obtained using the ideal vibrating chord theory. Accelerometers were mounted on 12 representative hangers to evaluate tension forces. Frequencies computed from responses measured under ambient vibration by these accelerometers were used to obtain tension forces.

No particular trend was observed from the analysis of the tensile force in each of the 12 hangers separately. However, the average of tension force for the whole set of hangers presented similar shape to the fluctuation of ambient temperature, as shown in Figure 4. Before railway construction, this average ranged between 789 kN and 800 kN, which corresponds to a variation of $\pm 1.4\%$ compared to the overall mean value. After completion of railway construction at May of 2004, hanger tension has been increased 2.9%.

Joint displacements of both ends were seen to be essentially affected by temperature changes. Result shows the quasi-linear relationship observed between joint displacement and temperature. Displacement of the expansion joints averages 46 mm for a thermal variation of 10°C in ambient temperature.

Dynamic properties have been analyzed using acceleration data under ambient vibration. Measured frequencies of the 1st and 2nd modes are 0.494 and 0.831 Hz, which show almost no difference compared to the field vibration test results, 0.487 Hz and 0.810 Hz. The effect on natural frequencies by railway mass addition shows 1.84% decreasing for the first bending. Since temperature changes have direct influence on the dynamic properties of bridge structure, temperature effects on the dynamic properties have been investigated and results are addressed in subsequent report.

Functional relations between temperature and structural response are defined before analyzing currently measured signals. Structural change or damage is found to occur when measured signals do not agree with the so-predefined temperature-response relationships. Various kinds of system identification methods including neural networks, statistical method, and optimization method can be employed to construct a mathematical function. Among them, the ARX model, a statistical time series analysis method, is used for Yongjong Bridge's data analysis.

3 CONCLUSIONS

The monitoring system installed in Yongjong Bridge is an integrated structural health monitoring system which is composed of sensors, data acquisition systems, signal transmission devices, signal control systems, and computer networks. This system has been successfully installed based on a proper development strategy. After its complete installation, test operation was performed on the system during one year, making it possible to stabilize the system and bring several rearrangements and minor changes in its configuration. The stabilized system produced valuable data to be used for health assessment of the bridge. The analysis of measured data verified that the behavior of the bridge is essentially governed by yearly and daily fluctuations of the temperature. Detailed analysis results of current signal data reveal that the bridge behaves as expected. After completion of railway on the existing bridge, the monitoring system detects changes of measurements which are identical as expected in the analysis. Other research directions have also been addressed to improve future performance of the monitoring system.

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Behavior monitoring of the Korea Highway Corporation test road

S.-M. Kwon & J.-H. Lee
Highway and Transportation Technology Institute, Kyonggi-do, Korea

J.-H. Jang
TM E&C Co., Ltd, Kyonggi-do, Korea

H.-G. Park
NewConsTech Co., Ltd, Seoul, Korea

1 INTRODUCTION

A test road has been built in 2002 by the Korea Highway Corporation in the central part of the peninsula to establish and develop pavement techniques and performances adapted to the local conditions. The test road is a 2-lane road of 7.7 km composed by 25 concrete pavement sections and 33 asphalt pavement sections. A total of 1,900 sensors of 11 types including strain gauges, soil pressure gauges and thermocouples are embedded in the pavement sections to monitor their long-term behavior under environmental and traffic loadings. This paper intends to give an overview of the monitoring system installed in the test road and addresses recent behavior monitoring results obtained for the anti-frost layer of the test road. These results constitute a first step in the development of reliable anti-frost layers through the investigation on the variation of freezing depth with the thickness and material types of the pavement layers.

2 EVALUATION OF THE EFFECTIVENESS OF ANTI-FROST LAYER

When the road subgrade remains under 0°C for a long period, the surface of the subgrade begins to freeze reducing gradually the water content of the subgrade. Due to capillarity, this remaining moisture migrates toward the frozen part of the surface to create ice lenses. Subsequently, frost heaving occurs as the ice lenses enlarge, and provokes thawing around the frozen part due to the increase of moisture content, which finally degrades the structural capacity of the road.

A preliminary study has been implemented using the frost data of the test road in order to evaluate the effectiveness of the installed anti-frost layers. The freezing index of the test road site has been calculated and the freezing depths of the concrete and asphalt pavements were examined using ambient and pavement temperatures as well as water content. The temperature data collected by means of the 250 thermisters buried in the pavements during 1 year from July, 2003 to July, 2004 were analyzed. Figures 1 and 2 plot the thermal variation patterns measured in the upper layers above the sub-base course and in the subgrade (Jeong et al., 2005).

In order to estimate the temperature at the top of the subgrade for concrete pavement, the correlation between temperature and depth has been analyzed as shown in Figure 3(a). The analysis revealed that upper layers presenting relatively thinner thickness presented the largest variation of temperature. It can also be observed that, even if difference of temperature occurs according to the depth of the subgrade, all of the curves present similar slope which means that the temperature by depth of the upper layers in the concrete section is influenced by the ambient temperature.

Figure 3(b) plots the correlation between temperature and depth of the subgrade for the asphalt section. It can be seen that the largest thermal distribution occurs in the filling section, while the smallest one occurs in the cutting section. This can be explained by the fact that the insolation of the cutting section is smaller than the filling section, which decreases the temperature of the pavement and increases gradually the difference of cumulated temperature in the cross-section.

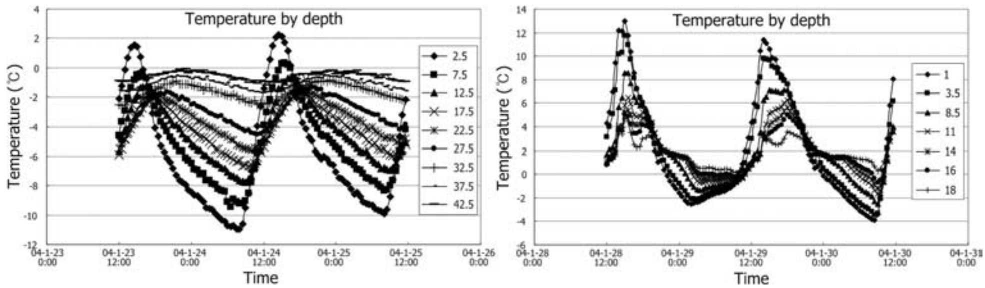

Figure 1. Thermal variation patterns in layers above the sub-base course (left: concrete pavement; right: asphalt pavement).

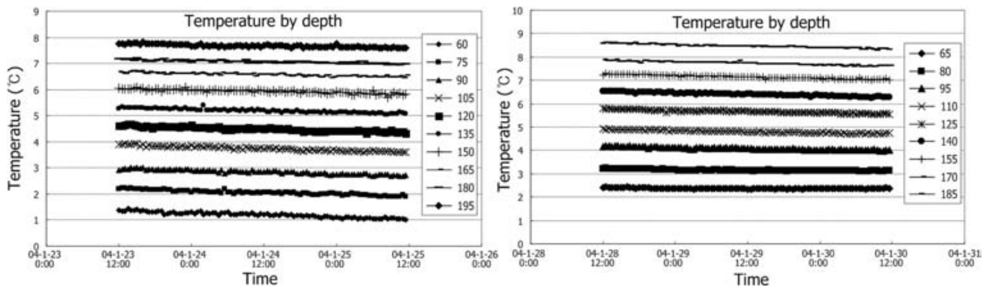

Figure 2. Thermal variation patterns in the subgrade (left: concrete pavement; right: asphalt pavement).

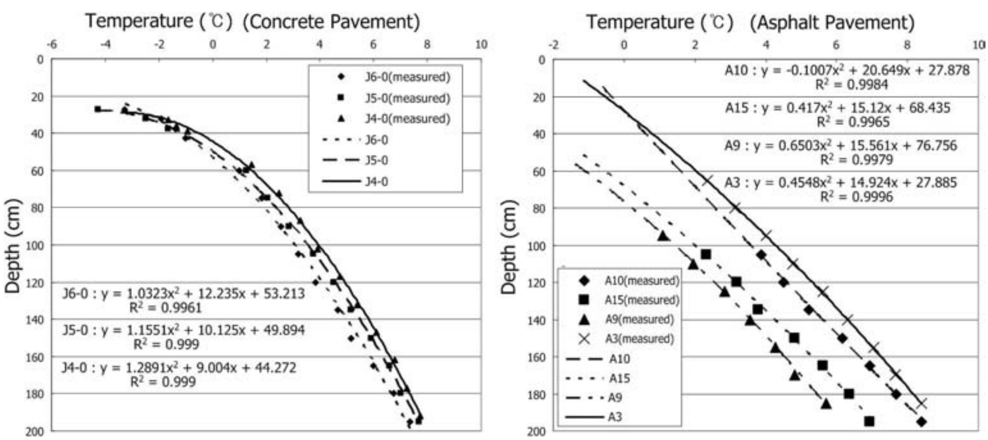

Figure 3. Temperature–depth correlation: (a) for the whole concrete pavement, (b) for the subgrade of the asphalt pavement.

3 CONCLUSIONS

Pavements in Korea are currently designed according to foreign road pavement design specifications, which produce pavement design that cannot reflect the local characteristics. To remedy the absence of appropriate pavement design, the Korea Highway Corporation undertook researches on test road for long-term development of pavement design specifications and improvement of pavement-related technologies, which resulted in the completion of a test road in December, 2002. Preliminary research results on the evaluation of the effectiveness of anti-frost layers have been also addressed. The final objective is to extend the lifetime of pavement constructed in Korea and reduce the budget invested for its construction and maintenance.

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Bridge weigh-in-motion without axle-detector in a cable-stayed bridge

M.-S. Park
Highway and Transportation Technology Institute, Kyonggi-do, Korea

S. Kim
Seoul National University of Technology, Seoul, Korea

B.-W. Jo
Hanyang University, Seoul, Korea

J. Lee
Research Institute of Industrial Science and Technology (RIST), Kyonggi-do, Korea

1 INTRODUCTION

In this study, investigation of the H/W of BWIM system that is designed and installed according to the conventional theories and now in operation stage in Seohae Bridge is performed, and then development of improved BWIM method is attempted with the signals acquired from the installed H/W system. For this purpose, one of the AI (artificial intelligence) techniques, Neural Network (NN) method is employed. A series of truck running experiment are executed to acquire NN training data.

2 FIELD LOADING TEST

Measurement hardware consists of sensors and data acquisition system (DAQ). Location of installed sensors is presented in Figure 1, and the list of the sensors is shown in Table 1.

Table 1. Sensor Location and quantity.

Location				
Dir.	Sec.	Lane	Quantity	Sensor ID*
North-bound	L16	1st	–	–
		2nd	2	K5, K7
		3rd	2	K1, K3
	L17	1st	1	A3
		2nd	4	K6, K8, A2, P2
		3rd	4	K2, K4, A1, P1
South-bound	L16	1st	–	–
		2nd	2	S1, S3
		3rd	2	S5, S7
	L17	1st	1	A4
		2nd	3	S2, S4, A5
		3rd	3	S6, S8, A6

* K: concrete-embedment type, A: steel-weld type, S: concrete-foil type, P: piezo type.

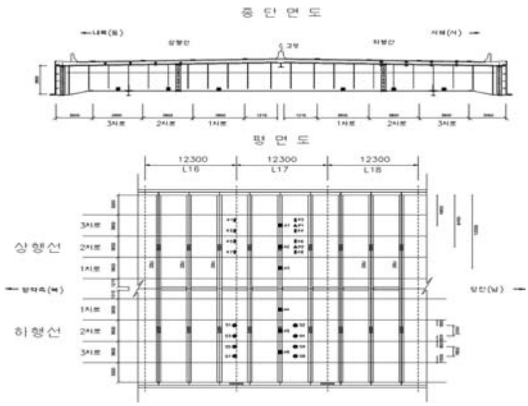

Figure 1. Sensor disposition map.

Experimental vehicle running test is performed on Seohae Cable-stayed Bridge, using pre-weighed 3, 4 and 5-axle dump trucks. Test trucks are shuttled between West-Pyungtaek I.C. and Songak I.C. on Seohaeahn Expressway. Visual image of traveling vehicles are also recorded using video camera for simultaneous analysis with acquired dynamic strain signals. Traveling speed of test trucks is controlled as 60 km/hr, 70 km/hr and 80 km/hr. First vehicle running test is performed for three days from September 13th, 2005, and complementary test is performed on December 11th and 12th.

3 DEVELOPMENT OF TRAFFIC LOAD ANALYSIS SOFTWARE

Traffic load analysis software which is under development in this study consists of three phases such as 1) data preprocessing, 2) training of neural networks, and 3) identification of vehicle information.

Input training vector for passing lane identification network consists of 16 maximum strain values of corresponding channels of each strain gauge attached on the slab (A1∼A8 and K1∼K8 of Table 1), thus the length of the input vector equals to 16. Target output values are designed as integers from 1 to 6, and each output value corresponds to N-1, N-2, N-3, and S-1, S-2, S-3 lanes respectively. Therefore, number of nodes in input layer equals to 16, and output layer has 6 nodes. Output values for each output node can be a real number between 0 and 1, and only target output node should show unity and all other nodes must output zero for exact answer. Single hidden layer is used between input and output layer, and 10 nodes are placed in the hidden layer considering the number of nodes in input layer and output layer.

Input training vector for axle-number identification network is 2 seconds' recorded signal of one strain channel that shows the largest reading in the 16 slab strain channels. Target output is an integer between 1 and 6, which directly matches to the number of axles of traveling vehicle except unity. Number of nodes in input layer equals to 1001, since the sampling frequency was set to 500 Hz, and Output layer has 6 nodes. Similarly to the former case, each output node can show a real number between 0 and 1, and only target output node should be unity for exact result. Also, single hidden layer is placed between input and output layer, and the number of nodes in hidden layer was set to 100 for axle-number identification network.

Figure 2 and 3 show the results of speed calculation, lane identification, and axle-number identification. K3 and K4 channel are utilized for speed calculation as shown in Figure 2, and the speed of the vehicle was calculated as 78.9 km/hr. From Figure 3, identification results of passing lane and number of axles are found as N-3 and 4 respectively, thus correct answers are achieved for this test case.

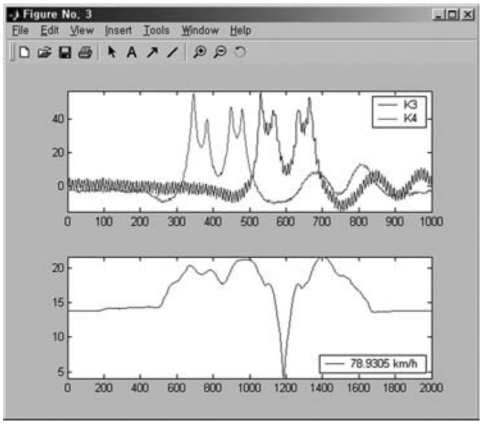

Figure 2. Speed identification result.

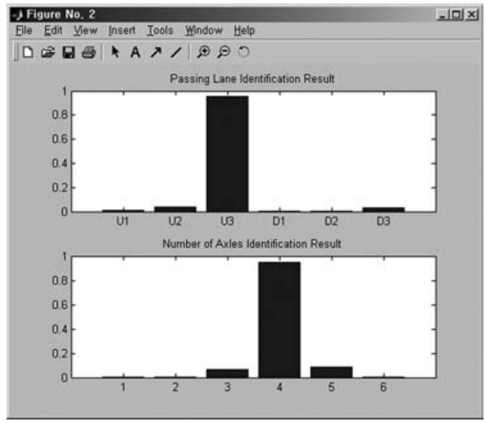

Figure 3. Lane and axle identification result.

4 CONCLUSIONS

In this study, algorithms for identification of traveling vehicle's information is developed using BWIM hardware which consists of 22 dynamic strain gauges and artificial neural network technique. A total of 124 data sets are utilized for training of neural networks and the trained networks properly distinguishes different cases of passing lanes and number of axles. Also, speed calculation module which computes the speed of a vehicle from the two channels' time lag shows accurate results.

A module for weight identification is under development. Complementary experiments are scheduled to acquire enough number of data sets, and the results will be presented in near future.

*Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Development of maintenance and monitoring system for Young-Heung Bridge using the latest technologies

D.W. Kim
Technology Department, GS E & C., Seoul, Korea

K.J. Joo
Construction Technology Institute, Yooshin Eng. Corp., Seoul, Korea

S.-B. Shin
Dept. of Civil Eng., Inha University, Incheon, Korea

ABSTRACT: For a decade many countries and institutes in the world have developed many kinds of management system of bridges. Generally the management system is composed of maintenance system to control manually and monitoring system based on the automatic remote sensors, where the systems are not compatible with each other because of some reasons. When Young-Heung Bridge opened recently, the maintenance and monitoring systems are integrated with the latest technologies and equipments. Thus, Visual Interface Module (VIM) is developed for easy use of structural database in main controlling computer to manage all information and field tablet-computer to collect visual inspection data. Besides, the computer can save the detail map of structure damages as well as visual rating at once. The gathered information of the every element is sorted in accordance with bridge inventory and evaluation systems of bridge condition along time. In addition, the rational algorithm of bridge condition (RABC) makes a quantified visual condition of element and bridge rating. On the other hands, many sensors, such as anemometers, accelerometers, inclinometers, strain gauges, and thermometers, monitor the behavior of the cable bridge. The measured results are automatically sent to main system. These two systems are used for mutual reference of structural safety and integrated condition assessment of cable-stayed bridge. This development serves to overcome many problems and weakness of existing bridge management systems and may be helpful to other maintenance agencies.

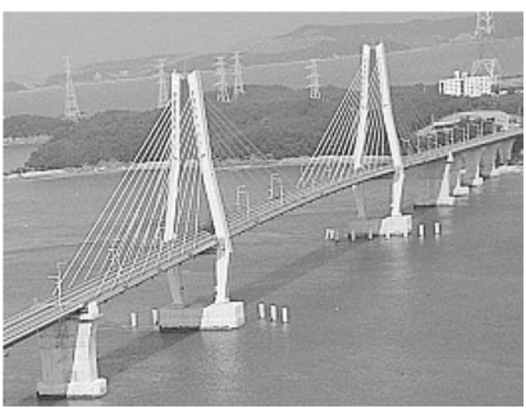

Figure 1. Young Heung Bridge.

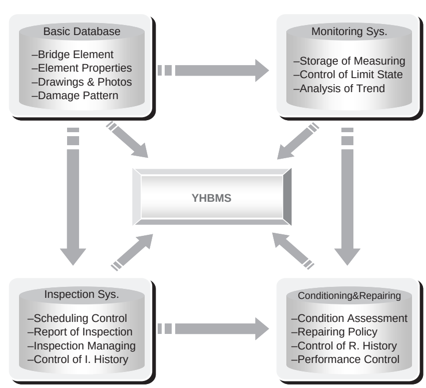

Figure 2. Organization of the YHBMS.

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*Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Development of measuring data system of bridges by wireless transmission using fiber Bragg Grating sensor

K.H. Kwak, H.S. Hwang & B.K. Sung
Department of Civil & Environmental Engineering, Wonkwang University, Iksan, Republic of Korea

ABSTRACT: A bridge, one of primary infrastructures in our society, can be damaged structurally from unexpected environmental changes such as earthquake, storm, or flood. Therefore, bridges have risks of collapse or decrease of life span. Recent researches on safety assessment for structures using fiber optic sensors and real-time measuring system are actively being done in order to obtain basic data of structural damages, degree of damages, and structural problems. At this point of time, this study is to recommend transmission of data measured with FBG (Fiber Bragg Grating) sensor by means of Bluetooth, the wireless communication system.

Indoor and field experiments were performed to verify the reliance of data by wireless transmission. Indoor experiment was carried out with specimens of reinforced concrete beam, which contains FBG sensor, electric resistance sensor, and LVDT (Linear Variable Differential Transformer). With these specimens, the reliance of data by wireless transmission was verified by static and dynamic stress test. Moreover, field experiment was performed with prestressed concrete bridges for reliability of applying indoor experiment to the field. To make it same condition as indoor experiment, static and dynamic stress tests were done to specimens of reinforced concrete beam, which contained FBG sensor, electric resistance sensor, and LVDT, and this could verify the reliability of data by wireless transmission.

Indoor test for measuring stress is done with static and fatigue load at RC beam specimen to verify data transmitted wireless. For the purpose of transmitting data measured through FBG sensor, measurement of wireless data transmission is done after setting up wireless transmitter-receiver interface between FBG sensor system and Bluetooth. Communication sections of measured data through FBG sensor are set up to be 5 m, 20 m, and 50 m and data transmitting abilities of each section are apprehended.

Static test of measured values of FBG sensor and electric resistance sensor showed low error rate of less than 0.3%. Estimation of deflection with FBG sensor was almost accurate within error rate 1%. Moreover, wired and wireless measurement showed almost same data transmission by both sensors. Fatigue test of measured values of FBG sensor and electric resistance sensor showed low error rate of less than 0.28%. Estimation of deflection with FBG sensor was almost accurate within error rate 0.29%. Moreover, wired and wireless measurement showed almost same data transmission by both sensors.

Results of depression of FBG sensor by wireless transmission at static loaded bridge showed error within 1.1% rate on the basis of deflection of FBG sensor by wired transmission. Hence, measured data of PSC bridge with FBG sensor showed error from 0.23 minimally to 1.02% maximally on the basis of measured data with LVDT. Results of depression of FBG sensor by wireless transmission at dynamic loaded bridge showed error within 1.25% rate on the basis of deflection of FBG sensor by wired transmission. Hence, measured data of PSC bridge with FBG sensor showed error from 0.76 minimally to 1.25% maximally on the basis of measured data with LVDT.

This study is to develop a measuring system of bridges which allows wireless transmission, by Bluetooth, of the data measured with FBG sensor. For this purpose, measured data of FBG sensor are transmitted with a variation in the wireless communication section. Experiments on

the specimen are performed by laboratory test and construction field test. The followings are the conclusions acquired from the study:

1. A system transmitting data measured by FBG sensor by means of Bluetooth is developed.
2. Data measured with FBG sensor, transmitted both wired and wireless by Bluetooth, have error within rate 0.3% at the specimen under static and fatigue load. Experiment applying to the bridge showed error within 1.1% of static and 1.3% of dynamic test. Therefore, measured data by FBG sensor in the lab or the field almost accord with the data transmitted wireless.

In conclusion, it is highly recommended to use Bluetooth, the wireless transmitting system, for transmission of measured data by FBG sensor, which is the safe and accurate measuring means. With all these means, maintenance and management of bridges will be possible without any limitation of time and place.

Damage assessment – strength and durability

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Application of a new metal spraying system for steel bridge Part 3. A report on 9 or 13 years experience with the spraying system

T. Kondo
Institute of Technologists, Saitama, Japan

S. Okuno
Nitto Engineering Co., Ltd., Osaka, Japan

A. Yamazaki,
Nippon Paint Co., Ltd., Osaka, Japan

H. Matsuno
Dai Nippon Toryo Co., Ltd., Osaka, Japan

ABSTRACT: Japan often experiences high humidity and temperatures, especially during the summer. Therefore, metallic corrosion prevention is an important requirement for the construction of steel structures construction. Against this background, a new metal spraying system was developed. An outline of the newly developed metal spraying system and a performance of the zinc-aluminum complex sprayed steel bridges was been reported in IABMAS'04. In that paper, the efficacy of the system for corrosion prevention of steel bridges was discussed.

In this paper, we discuss the corrosion prevention mechanism based on analysis results from actual metal sprayed steel structures. These actual metal sprayed steel structures are a lighthouse on a land and a crane girder which juts out far over the sea. The durability of the sprayed film based on the corrosion prevention mechanism is also discussed.

1 OUTLINE OF THE SYSTEM

Conventional galvanized steel has been used for corrosion prevention for many years. However, it mars the beauty of the scenery and it does not have sufficient durability in an environment subject to serious corrosion. If a paint coating is applied to a galvanized surface, it does not have sufficiently hardened film adhesion. And conventional metal spraying systems require stringent measures to prepare a clean surface, and must operate at a high temperature – over the melting point of the sprayed metal. Therefore, the system requires a high grade blasting process for substrate surface preparation, and it cannot be applied to substrate surfaces such as concrete or mortar, which have poor heat resistance. Thus a new high performance and cost-effective metal spraying system was developed. The new system has a special metal spraying apparatus that allows metal to be sprayed onto substrate surfaces at room temperature and uses a coating material for surface preparation, instead of blasting. The coating material provides substrate surfaces with suitable roughness and good adhesion between the sprayed metal and the substrate surface. This metal spraying system can be applied not only to certain metals, but also to concrete or mortar surfaces that have poor heat resistance. The newly developed metal spraying system, a useful metallic corrosion prevention method for steel bridges, was experimentally evaluated. The sprayed complex zinc and aluminum film was pronounced to have corrosion resistance in excess of steel galvanized with 550 g/m^2 of zinc. Adhesion of the sprayed metal is most stable with a surface preparation method that coats the primer, which is a substrate surface preparation material.

The zinc-aluminum sprayed film has retained superior corrosion protection potential and corrosion inhibition effects over an extended period. The film provides good corrosion prevention for extended periods even in coastal environments.

2 ANALYSIS OF THE SPECIMEN FROM 9 YEARS' EXPERIENCE WITH THE SYSTEM

The target structure was a steel lighthouse, located on a piece land in Shonan harbor near Tokyo and constructed in 1951. The existing coating film had been seriously degraded by sea salt. Therefore, zinc-aluminum spraying was used on the entire lighthouse surface in November 1994. However, as blasting could be partially used for surface preparation because of the site construction, preparation during construction involved the use of a coating material and power tools. The investigation was conducted in May 2004.The appearance of the metal sprayed surface has been differed widely on each side. The south-west side facing the sea was extremely deteriorated. However, the exterior of the other sides was good by comparison. As estimated there was no reduction in the film thickness. A good level of film adhesion was maintained and this level remained the same as that applied initially. The electrochemical potential of the film surface remained less noble than the iron natural potential. The potential was similar to that of the salt spray test results which were reported in IABMAS'04. As a result of X-ray microanalysis of the cross section of the specimen taken from the lighthouse surface, we conducted that Zn in the sprayed metal is first oxidized and then Al, and an oxide layer is formed close to the substrate surface. Additionally, there is little intrusion of the chloride ion into the sprayed metal film is and the film has an interception effect.

3 ANALYSIS OF THE SPECIMEN FROM 13 YEARS' EXPERIENCE WITH THE SYSTEM

The target structure was a steel crane girder which juts out far over the Sea of Seto. Zinc-aluminum spraying and several types of anticorrosion coating were applied to the structure in September 1991. The investigation was conduced in September 2004. The metal sprayed surface has almost no rust. As a result of X-ray microanalysis of the cross section of the specimen taken from the structure surface, we conducted that Fe is partially detected on the substrate surface. The residual Fe is due to simple surface preparation with a power tool. There is little O in the sprayed metal and/or on the substrate although it is detected in the surface layer of the film. Unlike the lighthouse case, a priority oxidation of Zn in the film cannot be ascertained here. Difference in the structure location may account for the difference between both the lighthouse and the crane girder. The lighthouse is located in a position facing the open sea whereas the steel crane girder faces an inland sea. The latter is placed in a mildly corrosive environment.

4 CONCLUSION

The zinc-aluminum sprayed film retained superior corrosion protection potential and corrosion inhibition effects over an extended period.

In this paper, the discussion on corrosion protection mechanism of the metal spraying system was discussed. Results of the investigation revealed that with this Zn in the sprayed metal is first oxidized and the Al and an oxide layer is formed close to the substrate surface. There is little intrusion of the chloride ion into the sprayed metal film and the film has an interception effect. However, this cannot be confirmed in the case of the crane girder.

Therefore, it is necessary to conduct a follow-up survey to discuss the excellent corrosion prevention effect of the metal spraying system and examine the LCC view.

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Relationship between bearings type and their most common anomalies

Luís M.R. Freire
Estradas de Portugal, E.P.E.; I.S.T., Lisbon, Portugal

Jorge de Brito
I.S.T., Lisbon, Portugal

ABSTRACT: Bearings have an important role in bridges functioning. An anomaly in a bearing affects the bridge's global behavior, becoming a structural problem. According to statistics, conservation costs of bearings represent around 5 to 7% of the global conservation costs in road bridges. With such significant values, anomalies in bearings must be considered a structural safety problem as well as bridges management problem. Therefore, it is fundamental to establish an efficient inspection and diagnosis system where the different types of bearings have their anomalies Identified and related with their causes.

According to the European and American institutes that regulate and license bridges, in those of medium to great size the use of bearings is indispensable for their execution. Monolithic bridges seldom go beyond overall lengths of 100 m.

The movements in bridge bearings are accommodated and/or supported by basic mechanisms that can be identified as internal deformation for those using elastomers, sliding for those with teflon (PTFE - polytetrafluoroethylene), rolling for metallic roller bearings and rotation or pendulum movement for rocker bearings. The great variety of bearings uses different combinations of these basic mechanisms.

Next a proposal of classification of bearings is proposed in accordance with what is prescribed in the new European Standard EN 1337, complemented with some older bearings still in service and including a brief description of each type. This classification takes into account mostly the constituting materials and the basic movement mechanisms:

- Metallic bearings: they may be divided in the following way:
  - Roller bearings regulated by Part 4 of EN 1337;
  - Rocker bearings regulated by Part 6 of EN 1337;
  - Spherical and cylindrical PTFE bearings regulated by Part 7 of EN 1337;
  - guide or block bearings regulated by Part 8 of EN 1337;
  - elastomeric bearings regulated by Part 3 of EN 1337;
  - Pot bearings regulated by Part 5 of EN 1337;
  - uplift bearings;
  - lead bearings;
  - Concrete hinges;
  - Polymeric bearings.

The most common anomalies occurring in bearings may be grouped and classified based on some criteria, namely according to their composing materials and movements allowed.

A set of anomalies associated with specific bearings is presented next:

- for metallic bearings, there are anomalies of a general nature, common to all types, related with steel degradation, such as corrosion with or without displacement impairment and anchors, bolts or screws detachment;
- for roller bearings, the most common anomalies are associated with materials deterioration, faulty instalment and partial or total impairment of the roller capacity; the main anomalies are

cracking or fracture of the roller, ovalty and/or deformation of the roller, transposition of the guide or bolt of the roller and localized loss of the outer alloy of the roller uncovering the steel; in the case of multiple roller bearings, loss or wear of their cogs may occur;
- for steel rocker bearings, the most common anomalies are related with the basic movement they allow and the material, such as excessive or deficient tilting of the rocker, cracking or fracture of the elements or impairment of the rockers' rotation;
- for spherical and cylindrical bearings, the most common anomalies are related with the basic movement they allow and the characteristics of the material, such as excessive or deficient sloping of the spherical calotte, cracking or fracture of the elements, impairment of the displacement of the calotte, partial detachment or slipping of the teflon layer and uneven thickness of the teflon layer;
- in guide or block bearings, the most common anomalies are related with the bearings and coating materials and the guides and locking devices, such as loss of a guide or a locking device, deterioration of guides or locking devices, impairment of displacement or rotation and coatings defects, such as loss of adhesion to the support, cracking, creep or crushing of the neoprene and crushing of the teflon layers;
- for elastomeric bearings, the most common anomalies are related with defects in the elastomer, the metallic reinforcement or their connection; examples of anomalies in the neoprene are overly high distortion – displacement over half the height, slipping over the support and neoprene defects, such as cracking, creep, crushing or partial settlement; for the steel plates, perforation or pitting, corrosion and partial or total loss of adhesion to the support can be referred; for bearings with sliding plates, deterioration of guides or restraints, cracking, impact signs and broken corners, oxidation of the metallic parts and loss of adhesion or slippage of the teflon layers can be mentioned;
- for pot bearings, the most common anomalies are related with defects of the elastomer, the metallic parts and the sliding plates (for those bearings that allow horizontal displacement); examples of anomalies in the neoprene are crushing, creep and high lateral deformation that may lead in extreme cases to its falling outside the pot; for the metallic parts of the pot, corrosion, cracking, impact signs and displacement impairment by obstruction of the clearance between the lid and the pot may occur; in sliding bearings, the possible problems with the sliding plates are identical to those referred for elastomeric bearings;
- lead bearings present a set of anomalies related with the material's characteristics, such as crushing, cracking, creep, corrosion and loss of adhesion to the support;
- in concrete hinges, the anomalies are normally related with the peculiar functioning and shape of this type of bearing, strangled heavily reinforced concrete sections where very high stresses accumulate; therefore, crushing or corners' cracking, concrete cover delamination in the middle region, where plasticization of the reinforced concrete section occurs, and reinforcement corrosion may be found;

There are anomalies that may be considered common to most bearings, whose probable causes derive from installment faults and lack of maintenance. In this group of anomalies, the following may be considered the most common:

- Geometry defects of the bearing support, such as loss of horizontality or parallelism, unevenness and dimensions lower than those of the bearing;
- defects of conservation of the bearing support, such as delamination, partial fracture, cracking and settlement of the bearing;
- Discontinuity in the contact surface;
- Dampness or water pounding in the bearing support;
- Debris or vegetation that impair totally or partially the bearing displacement;
- placing the fixed bearings with the shortest side non-parallel to the column's axis;
- Lack of lubrication thus increasing resistance to movement;
- Inverted instalment of the bearing;
- Mobile bearing not aligned with the fixed point or the fixed support.

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Residual structural performance of rolled H Members submerged in seawater for a long time and their anti-corrosion strategy

E. Watanabe, K. Sugiura & T. Utsunomiya
Kyoto University, Japan

M. Yamamoto
Japan Atomic Energy Research Institute, Tokyo, Japan

ABSTRACT: A study is conducted to find an anti-corrosion strategy for the effective maintenance of steel structures in the coastal areas on the basis of corrosion exposure tests on a rolled H member. The steel cross section is hypothetically assumed to be reduced at a constant corrosion speed. Basic comparisons are made between a titanium-clad steel specimen and specimens with different paint coatings in consideration of the cost and lives of expectancy assuming a constant corrosion speed for the steel. It was found from the comparison that the least life cycle cost at 100 years from the construction was found when the fluorine resin coating is chosen and the repainting is made at every 30 years and the member is replaced once with the new member. Furthermore, it is found that the titanium-clad steel could be the most economical choice if the unit price could be reduced by 50%.

1 INTRODUCTION

Steel structures can be built anywhere such as near rivers and at seaside. The speed of corrosion varies from fresh water zones to salt water zones. The dissolved oxygen in water either fresh or salt controls the corrosion of steel. In the seawater, due to higher electric conductivity than the fresh water, the difference of the speed of oxygen supply between the zone above water level and in the water immediately below causes corrosion as the result of formation of the macro cell. Furthermore, rust once generated tends to accelerate the corrosion of steel members by the repetitive process of oxidization and deoxidization. Lastly, the waves and littoral transport in the sea cause the formed rust to peel off and thus accelerates the corrosion faster than in fresh water.

Although in recent years, the anti-corrosion technology has been improved significantly, the problem of corrosion is a long-term process and thus it might take a long time before the effect of the anti-corrosion methods is assessed. Consequently, the validity of the short-term prediction methods is among the top research targets. In the mean time, the asset management is becoming more and more important and the optimum methods to determine the least Life Cycle Cost (LCC) are discussed in many fields.

Basic comparisons are made among 8 different cases with variations of interval for repainting:

(1) Without any application of paint (Case 1), i.e., naked steel
(2) Polyurethane resin coating with 20 years interval of painting: Cases 2 and 3 where in the former, the repainting starts after 20 years and in the latter after 10 years
(3) Fluorine resin coating: Cases 4, 5, 6 and 7 where the repainting starts after 30, 20, 10 and 10 years, respectively. In Cases 4 and 7, the interval of painting is 30 years; while in Cases 5 and 6, the interval is only 20 years
(4) Protection with titanium lining: Case 8

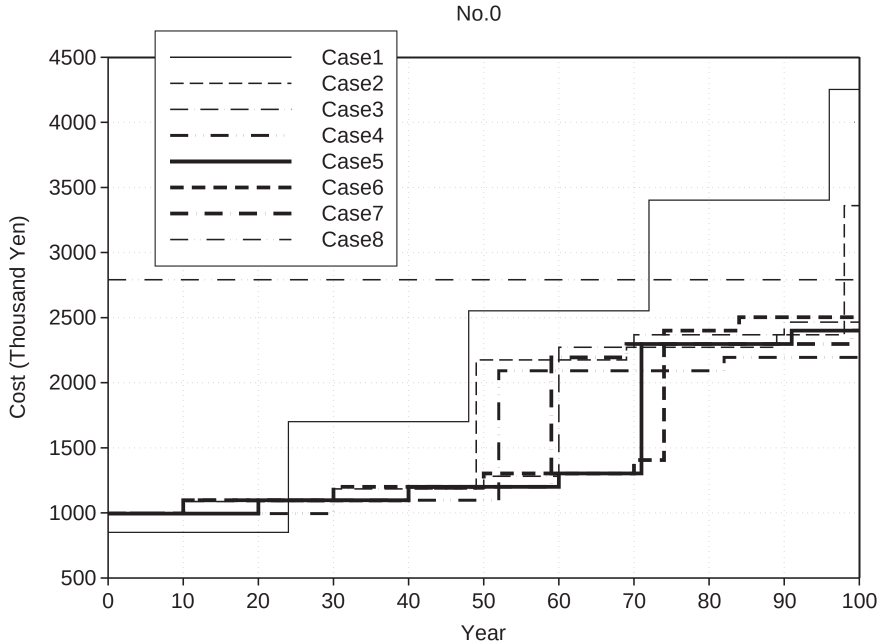

Figure 1. Comparison of Life Cycle Cost among various Anti-corrosion Measures based on Current Costs.

In addition to the corrosion speed of the steel assumed to be 0.3 mm/year, the life of painted coating is assumed to be 20 years for the polyurethane resin, 30 years for the fluorine resin and the titanium lining is assumed to be maintained for more than 100 years, respectively.

The steel is simply assumed to cost JY*700 thousand/ton (*JY refers to Japanese Yen) and the painting cost is assumed to be JY7.2 thousand/m^2 initially and JY5 thousand/m^2 for repainting in the case of polyurethane resin; JY7.4 thousand/m^2 initially and JY5.3 thousand/m^2 for repainting in the case of fluorine resin; while titanium costs JY100 thousand/m^2. It was found after comparison that the least life cycle cost at 100 years from the construction was found in the case when the fluorine resin is chosen and the repainting is made at every 30 years and the member is replaced once with the new member as shown in Figure 1. Based on the results of these basic cases, further comparisons are made by considering possible reduction of painting cost by 10, 20 and 30% for the Polyurethane resin coating; increase of painting cost by 10, 20 and 30% for the Fluorine resin coating and reduction of cost for the titanium-clad steel by 20 and 50%.

2 CONCLUDING REMARKS

This paper is concerned with the anti-corrosion methods by paint coatings and titanium-clad steels for steel structures that are assumed to be exposed in the coastal area. It has been found that the most severe corrosions will occur in the splash zone or in the shallow zone just below the M.L.W.L. Based on 23 years corrosion exposure test that has been conducted by Nippon Steel Corporation on a rolled H carbon steel member, a study was conducted to predict the life cycle cost considering the corrosion rate found from the exposure test and long-term durability tests on the deterioration properties of paint coatings including Polyurethane resin and Fluorine resin. Furthermore, a comparative study is made with the case when titanium-clad steel is in use.

From the results of the simulation, the following observations may be made:

(1) Better L.C.C. may be found by using the fluorine resin coating rather than using polyurethane resin coating because of firstly only relatively small price difference in reality and secondly the longer durability of the former resin, namely, 30 years of painting life as compared with only 20 years of the latter resin.
(2) Furthermore, it is interesting to note that keeping the maximum painting interval and repainting at the last minute may lead to better L.C.C. keeping in mind the assumption that the H-section must be replaced by a new member whenever the load-carrying capacity becomes less than the allowable stress.
(3) It might also be interesting to note that the use of titanium-clad steel may become competitive if its commercial price could be significantly reduced for example, by 50%.

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Strength of corroded tapered plate girders under pure shear

P.J.S. Cruz & L. Lourenco
Civil Engineering Department, University of Minho, Guimarães, Portugal

M. Santos
Information Systems Department, University of Minho, Guimarães, Portugal

H. Quintela
School of Technology and Management of the Polytechnic Institute of Viana do Castelo, Viana do Castelo, Portugal

P. Cortez
Information Systems Department, University of Minho, Guimarães, Portugal

1 INTRODUCTION

Steel plate girder structures, composed of nominally flat plates connected together usually by welding, are used in large structures of vital importance for society. Plate girders can also be designed as tapered plate girders, usually done by means of a web panel whose depth varies linearly. Designed in accordance with the distribution of bending moments, along the longitudinal direction of the structural systems, tapered web panels with variable inertia will provide the required resistance, inducing to a rational and efficient solution.

When steel plate girder structures are affected by corrosion, the estimation of the load carrying capacity may contain a relatively high level of uncertainty. The uncertainty in the evaluation of corroded tapered plate girders strength can be minimized by using models based on sound theoretical principles, validated by extensive and reliable data. In this way, an in-depth study in front of serviceability and ultimate limit states of corroded tapered plate girders is necessary.

The application of Data Mining (DM) techniques to analyze civil engineering data has gained an increasing interest in recent years, due to intrinsic characteristics such as ability to deal with non-linear relationships. Cruz et al. (2004) presented an alternative approach for the prediction of critical shear resistance of non-prismatic tapered plate girders, which used DM techniques and achieved encouraging results. In this study, the same approach is refined and applied using a wide range of experimental cases.

The aim of this study is, therefore, to investigate the effects of a local thickness reduction, due to corrosion, in tapered web plates, on the elastic critical shear buckling load and ultimate shear capacity and create a simplified tool to be used in the evaluation of tapered plate girders.

2 NUMERICAL MODEL

The considerable slenderness of a web panel is one of the main distinctive features of tapered plate girders. The web buckling can appear for relatively small values of the shear load with regard to the shear load that yields the web material.

The usual models to evaluate the shear capacity of plate girders are based on the diagonal tension field theory, interpreted by three resistant mechanisms: the first mechanism, the elastic shear buckling strength of the web plate; the second one, the post-buckling strength of the web plate, interpreted by the development of the tension field; the last one, the sway failure mechanism, which implies that the web panel reaches failure when plastic hinges are developed in the flanges.

Nonlinear analyses have been conducted in a three-dimensional finite element model of transversely stiffened corroded tapered plate girders subjected to pure shear. Corrosion is simulated by thickness reduction. The boundary conditions proposed would match closely the behavior of a simply supported beam under loading in its middle length. The beam represents, by symmetry, half of the simply supported beam.

A perfect web panel does not buckle. Therefore, the introduction of a geometric imperfection pattern is necessary for a post-buckling load-displacement analysis, turning it into a continuous response problem instead of bifurcation. The results were obtained for web panels with relatively small initial deformations ($h_1/200000$).

The presented model was validated with experimental and analytical results: experimental results by Lee & Yoo, design methods by Basler, Cardiff and Lee & Yoo.

3 SENSITIVITY ANALYSIS

For a random steel plate girder, sensitivity analyses have been conducted to evaluate the influence of the geometric properties in the shear behavior of a web panel. The geometric parameters h_1, t_w and α are the most influent on the shear behavior of steel web panels.

4 DATA MINING

Data Mining (DM) is often defined as the automated extraction of novel and interesting information from large datasets. DM has it roots in statistics, probability theory and machine learning. One of the underlying principles of knowledge discovery in data is to promote the process of building data-driven expert systems as an extension of the more traditional Artificial Intelligence expert systems approach. The idea is now that experts can learn from new findings in the data as well. The aim of this study is to produce accurate models to predict the Ultimate Load, using two kinds of DM techniques: Artificial Neural Networks and Decision Trees.

The framework used for the experiments involves two steps: (i) a clustering work to search and explore some kind of homogeneity in the data set, and (ii) the generation of prediction models adjusted for each homogeny cluster.

The dataset used for the experiments was developed using 5884 of the Lourenço (2005) study cases. Each row denotes the geometric design parameters of tapered plate girders, the results of critical load and ultimate load and the values of damage in each of the 9 considered sections. All experiments were conducted using the Clementine DM package (SPSS).

The clustering work was carried out using Kohonen's self organizing maps. In the Kohonen's network several parameters were experimented, being the final topology set to 16 input nodes and 6 output nodes corresponding to a map with a 2×3 grid. Then, the C5.0 algorithm was applied to each of the 3 clusters, in order to obtain a set of rules. The training set used a random sample with 2/3 of the available data, while the test set contained the remaining 1/3. The set of classification rules managed to correctly predict the cluster membership with an accuracy of 99%. After the clustering step, a Multilayer Perceptron was used to predict the ultimate load for each cluster, by using a random sample with 2/3 of the data, while the test set contained the remaining 1/3.

5 CONCLUSIONS

This experimental research is a "first step" for the creation of a simplified tool to be used in the evaluation of existent tapered plate girders. Using a validated finite element method, it is possible to create a large and extensive database, with various scenarios of degradation. This data, analyzed with Data Mining techniques, is a contribution to the development of new knowledge about the shear behavior of corroded web panels in plate girders. This will be certainty an important tool in the inspection and assessment of steel girder bridges.

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Accelerated exposure test of uncoated and metal-coated steels and its application

I-T. Kim & Y. Itoh
Nagoya University, Nagoya, Japan

1 INTRODUCTION

The coatings in atmospheric environments are deteriorated with exposure time, and periodic recoating is necessary for steel bridges to prevent corrosion attacks, to maintain the functional ability and to preserve cosmetic appearance. The performance prediction of the coatings becomes one of the most important issues in maintenance of existing steel bridges.

In evaluation or prediction of long-term durability of coatings for steel bridges, the field exposure tests and bridge inspections have been often used. However, it takes a long time to have any degradation to predict the lifespan of in-service bridges, even though they allow field examinations. For this reason, accelerated exposure tests are often employed to complement the field exposure test results. The accelerated exposure test has been also applied to a performance testing of newly developed coating systems in a short time.

To open up applications of the accelerated tests in various regional environments, the study examined the correlations between accelerated exposure test and field exposure test. Among available accelerated exposure tests, the S6-cycle corrosion test specified in Japanese Industrial Standards (JIS) K5621 was adopted and carried out on uncoated structural steels and metal-coated steels. Based on the present test data and published data of field exposure tests, the correlation of the S6-cycle test to the field exposure tests was evaluated, and its application to field environments with the amount of flying salt was presented.

2 EXPERIMENTAL PROCEDURE

In corrosion testing for uncoated steel plates, the steel substrates were made of two types of 9-mm thick mild steels with different manufacturing procedures, the blast furnace steels and electric steels standardized by JIS SM490. The steel plates were cut into the required size (70 mm × 150 mm), and then they were surface-treated. For metal coating testing, the blast furnace steel plates were metal-coated with four types of metallic coating systems, i.e., zinc hop-dip galvanizing, zinc-aluminum alloy thermal spraying by JIS method, low-temperature zinc-aluminum alloy spraying, and aluminum thermal spraying.

All prepared test specimens were placed at an angle of 15° from the vertical in an environment testing chamber, and aged during 30, 60, 90, 120 and 150 days for the uncoated specimens and 100, 200 and 300 days for the metal-coated specimens. The environmental condition of the chamber was controlled conforming to the S6-cycle corrosion condition, which consists of 30 minutes of salt water atomizing, 90 minutes of wetting, 120 minutes of drying by hot wind, and 120 minutes of drying by warm wind.

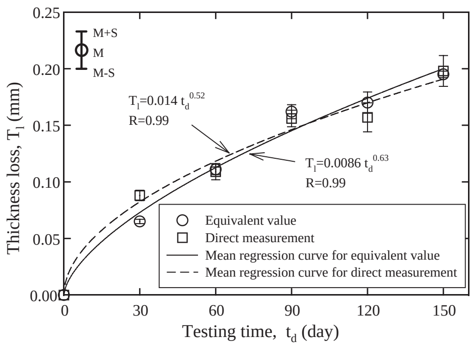

Figure 1. Thickness loss of uncoated steels.

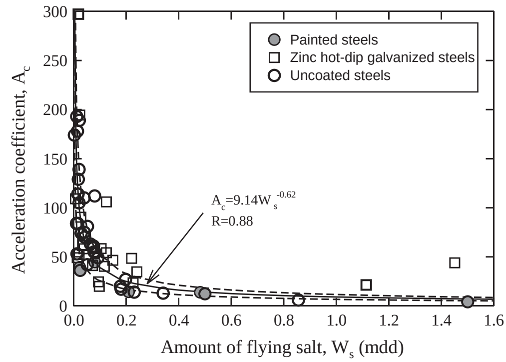

Figure 2. Acceleration coefficients.

3 RESULTS

3.1 *Thickness loss*

The mean thickness loss (M) and the standard deviation (S) of the uncoated specimens are plotted in Figure 1. The mean regression curves are also shown. The thickness loss increases with the testing time, but the increasing speed tends to decrease gradually.

The test results metal-coated specimens show that the thickness loss of metallic coating films can be approximately expressed by a linear equation, and become large in order of aluminum thermal spraying, zinc hop dip galvanizing, zinc-aluminum alloy thermal spraying and low-temperature zinc-aluminum alloy spraying.

3.2 *Acceleration coefficient*

The weight losses of the uncoated specimens for 30, 60, 90, 120, and 150 days were compared with 1, 3, 5 and 9 year field exposure tests at 31 sites in Japan. Acceleration coefficients A_c of the S6-cycle tests to the field exposure tests were calculated by the amount of flying salt, and they are shown in Figure 2. Mean regression curve of the data with the involution function of Equation 1 is also plotted by a solid line.

$$A_c = 9.14 W_s^{-0.62} \tag{1}$$

where, W_s is the amount of flying salt (mg/dm^2/day, mdd). The upper and lower confidence curves, located a standard deviation away from the mean regression curve, are also plotted by dashed lines.

Acceleration coefficients for zinc hot-dip galvanized steels and painted steels were determined by using the present test data and published data, and they are plotted in Figure 2. This indicates that the correlation between the S6-cycle tests and field exposure tests is approximately expressed by Equation 1. Therefore, using this equation the corrosion degradation degree and remaining lifetime of uncoated steels, painted and zinc hot-dip galvanized steels at the filed exposure sites can be approximately predicted quickly based on the amount of flying salt at the site.

4 CONCLUSIONS

The objective of this study is to determine the acceleration coefficient of the S6-cycle test condition and its application to prediction of anti-corrosion performance of steel bridge members under various corrosive environments. The accelerated exposure test of S6-cycle was performed on uncoated steels and four types of metal coatings. Comparing with published data, the acceleration coefficients of S6-cycle tests to field exposure tests were determined, and their application to metal coatings and painting was examined.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Condition assessment of concrete bridges during demolition

R. Bargähr & T. Vogel
Institute of Structural Engineering, ETH Zurich, Zurich, Switzerland

ABSTRACT: The research project ZEBRA lasting from 1997 to 2004 is a framework to gain knowledge from bridges in Switzerland that were demolished. These bridges enable a condition survey including destructive testing and non-destructive tests can be validated. Especially tendons can be accessed to inspect their actual state, which is almost impossible, if the bridge has to sustain its proper function. In total 89 bridges were recorded and 82 out of them were demolished. Detailed examinations were performed on 36 bridges.

1 INTRODUCTION

During the demolition of a bridge significant information about the condition of individual structural elements can be gained. These information would be difficult to get if the bridge had to sustain in service. So far only few investigations of this type have been carried out in Switzerland. One possible reason is that owners usually underestimate the benefit of such investigations. With increasing maintenance and repair costs the prediction of possible weak points in a structure gains more and more importance. Knowing the vulnerable weak spots, investigations can be planned and realised more effectively. In addition, new scientific methods concerning bridge assessment can be applied or verified before, while or after demolition. ZEBRA is an acronym for "ZustandsErfassung von BRücken bei deren Abbruch" and means condition assessment of bridges during demolition. It was aiming at the following goals:

- Condition of badly accessible, crucial or endangered structural elements
- Efficiency and accuracy of non-destructive testing methods
- Success of earlier rehabilitation measures
- Coincidence of planned and effective dimensions

The main focus was the condition state of post-tensioning tendons, because the examination by applying non-destructive test methods is rather difficult.

2 BRIDGE STATISTICS AND REASONS FOR DEMOLITION

In Switzerland most road bridges are owned either by one of the 26 cantons or by one of the more than 3000 municipalities. Other owners are railway companies and governmental or private companies. Due to this dispersed ownership it is rather difficult to establish a complete file of all bridges that are planned to be demolished. Only for state-owned bridges or bridges where the Swiss Federation pays a substantial share of construction and maintenance costs, the Swiss Federal Roads Authority has an overview of ongoing projects.

The first task was to gather information about planned demolitions well ahead of time. A data sheet containing all important specification of a bridge was sent to the responsible authorities. By this opportunity not only information was gathered but the relevant persons could be informed about the research project. Spontaneous reports of planned demolitions from authorities during the ongoing research project without request from ZEBRA showed that the concept was well accepted.

Another important aspect of a data sheet is that the statements can be systematically collected and analysed in a database. Some interesting points are illustrated in the extended paper.

3 CORROSION DAMAGE TO PRESTRESSED BRIDGES

Of the 82 covered bridges 36 where investigated in more detail resulting in an own report. All those 36 bridges were built with prestressed concrete except one. As a matter of routine the condition of the tendons was surveyed distinguishing the three components *anchorage* (anchor and end region of strands or wires), *duct* and *prestressing steel* (strands or wires). For demolitions on site this was achieved by observation during the process. In cases where bridges were cut into pieces the cross sections were examined in detail, but the anchorages were normally not exposed. The observed corrosion attack could be explained satisfyingly in each case. It was not always obvious, however, whether corrosion had occurred prior to the mounting of the tendons or during the service life. In most cases chlorides infiltered due to leaky sealing reaching the tendons and migrating under capillarity into and along the inside of the tendon duct. There were two cases, however, where chlorides were not involved in the corrosion of tendons. These are illustrated in the extended paper.

An increased thread to the prestressing steel by both leaking ducts and improper grouting exists in all places where prestressing tendons are used to tie prefabricated elements and therefore intentionally cross element joints. Respective faults were found at beam joints on piers as well as in midfield.

Corrosion at anchorages occurred when besides lacking or leaking sealing the concrete of the recesses was of bad quality. When anchorage zones were inadequately grouted a similar case occurred. In those cases not only anchorage plates and sleeves corroded, but also the end regions of the strands or wires.

4 CONCLUSIONS

The post-tensioning systems used up to the present stand the test of time. Corrosion may occur, however, in cases where all barriers (sealing, concrete cover, duct and grout) to chlorides fail. In detail these are:

- Improper or improperly drained sealing of the bridge surface, especially in the regions of anchorages, couplers, construction joints and vents
- Bad or leaky covering concrete, especially in recesses for anchoring
- Ducts with inadequate thickness or with dints originating from placing and casting
- Corroded ducts
- Bad conditions for proper grouting
- Steep tendons or tendon pieces that have not been grouted in two steps, as they occur in prestressed ties of V-shaped columns of frame bridges

Corrosion at mild steel is an indication for damages of tendons placed more distant from the surface. At anchorages corrosion is more frequent than along the tendons between them. However, with proper grouting, it is restricted to the anchorage plates and the end regions of strands and wires.

The investigations confirmed to a large extend the development of bridge design with prestressed concrete, as it has continued since the now demolished structures had been built. An optimal protection of the concrete and especially the prestressing steel from chloride charged sewage from the traffic space requires several barriers, namely:

- a high-quality sealing with a sufficient gradient of the pavement on top
- an adequate covering of the reinforcement with a dense concrete
- dense ducts
- a careful grouting of the tendons.

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A new method for two-stage structural damage identification

Y. Jiarong
Department of Bridge Engineering, Tongji University, Shanghai, China

ABSTRACT: Structural damage identification techniques are methodologies that detect and locate damage in structures and quantitatively assess its severity via measured responses. During the service periods, structures are confronted with different unexpected situations which may likely not be considered in the design process. And, more often than not, these loads will introduce into the structures various damages. By comparing the dynamic response of the structure under investigation before and after possible damage, it is possible for people to detect these damages. The Two-Stage Structural Damage Identification is a promising scheme in the field. By this way, structural damages can be located and quantified in the two stages independently.

In this paper, two new damage identification indexes namely Modal Curvature Index (MCI) and Modal Moment Index (MMI) are introduced into the scheme. In the first stage, the damaged segments of structures can be located and to some extent quantified by MCI. In the second stage, the damage severity of those located segments can be precisely estimated by calculating the MMI of each element. Later the numerical analysis of a simple beam is carried out to illustrate the method. It proves that the new method can not only correctly locates damaged segments but also offers practical estimates as to the damage severity. And it can be applied to many types of structures such as beam, truss, and plate etc.

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Cruz, Frangopol & Neves (eds)

Hungerford River Bridge No. 7 – a case study of assessment from first principles

A.K. Pope
Mouchel Parkman, West Byfleet, England

1 INTRODUCTION

1.1 *Aim of the paper*

The paper is intended to demonstrate what can be done with some simple clear thinking. In this case, the overall capacity of the bridge, for dead plus live load, was doubled. It will also argue that good assessment is an investment and not just a cost, with the potential for large returns on the original outlay.

1.2 *History of the Bridge*

Hungerford River Bridge No. 7, also known as Charing Cross Bridge, was originally constructed between 1860 and 1863. Soon after this, an additional bridge, No. 7A, was built next to the original. Over the years, various changes were made to the bridges, finally ending up in the 1970's with four trusses and two decks per span, but with an additional structure, the Middle Road, bridging between and carrying an additional track. These bridges form the only access into Charing Cross Station, which is one of London's major stations.

In the 1970's, the original decking of both Bridge 7 and the Middle Road were replaced using modern steel construction. Unfortunately, in the process, the U-frame action that had stabilised the top chords of Bridge 7 was removed and the moment connections between the trusses and the deck were replaced by physical pin jointed connections.

2 ORIGINAL ASSESSMENT

The bridge was originally assessed, by others, in 1997 with an assessed live load capacity of only 1 BSU (1 unit of live load). The "Real Train" loading from passenger trains was 8 BSU. This vital link was kept open after a risk assessment, until a full investigation could be completed.

Mouchel Parkman was commissioned to carry out a Stage Two assessment of the bridge in 2004.

BS 5400 expects that compression members, whether in trusses or in beams, will be restrained either by fully effective bracing or by partially effective bracing. Unfortunately, Bridge 7 had neither of these forms of restraint, so the top chord axial stress was limited to 42% of yield.

The truss connections were highly eccentric, leading to high local moments on the truss chords. The combination of limited axial capacity with high local bending was significant, resulting in a structural capacity only slightly greater than dead load.

3 STAGE TWO ASSESSMENT

The loads on the bridge were challenged, resulting in more capacity being made available for train live load. This including ignoring passenger loading on unused platform extensions and using lower partial factors for the highly controlled live loading.

3.1 *Increasing the structural capacity – Slenderness in BS 5400*

It is a common misconception that axial stresses on compression members are limited to prevent buckling. Obviously, if a compression member is loaded enough it is likely to eventually buckle. However, in the real world we are more concerned with the stresses that occur at the bottom end of the "Buckling" curve.

The original BS 5400 calculation limited axial stresses to 42% of the normal factored yield stress of 165 MPa, i.e. 69 MPa. What this did NOT mean was that the top chord would buckle when the axial stress reached 69 MPa. What it actually meant was that when the axial stress reached 69 MPa, the total stress on the inside of the bow would reach the limit of 165 MPa.

3.2 *Analysis using real imperfections*

The top chords were assessed using measured imperfections. The measured bow for the top chords was taken as 50 mm, to represent three standard deviations off the norm, which compared well with the default BS 5400 assumption of 47 mm.

The analysis of the effect of imperfections is non-linear, because the plan bending moments induced (also known as P-delta effects) cause deflections that change the shape of the structure. The structure was therefore analysed using a number of steps until a steady state was reached.

When the truss loads were applied to the ends of the chords (the BS 5400 assumption) very close correlation was achieved with BS 5400. However the more realistic introduction of the loads in stages along the chord resulted in significantly lower imperfection bending stresses, doubling the overall capacity of the bridge. The result was further improved by careful examination of global and local stress interaction.

The combination of the reduction in live load and the increase in structural strength raised the assessed capacity from 1 to 12 BSU. This demonstrated the wisdom of Network Rail's two-stage strategy for assessment. The first stage assessment passed a high proportion of structures relatively cheaply: this allowed a concentration of resources on structures that really need it.

3.3 *Return on investment*

An investment of approximately £40,000 removed the need for, *at best*, many hundreds of thousands of pounds of strengthening works or, *at worst*, the cost of replacing the bridge.

4 CONCLUSION

The assessment in this paper used the same basic principles as BS 5400. The principles, however, were applied in a method specifically targeted at the unusual form of structure.

The combination of first principle non-linear analysis, practical engineering, site measurement, and close liaison with a knowledgeable Client has led to a greatly improved assessed capacity at a tiny fraction of the cost of the possible strengthening bill.

The paper has demonstrated the benefits of employing good engineers who can make the most of the existing structures (colloquially, "sweating the assets"). It is too easy to look at bridge management as a simple business process where a penny saved now is a penny gained, which may work when buying ball bearings, but can be counter productive when buying services that are hard to define accurately. The intellectual input of engineers at assessment, feasibility or design stages is cheap compared to the cost of doing actual physical works, especially in the railway environment. However, this extra investment has the potential to save part, if not all, of the much more expensive physical works. This will usually far outweigh any additional intellectual engineering cost. It is therefore cost-effective to build relationships of trust with good engineers, and to let them act in a professional manner.

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Modelling the response of the New Svinesund arch bridge: FE model verification and updating based on field measurements

M. Plos
Chalmers University of Technology, Göteborg, Sweden

ABSTRACT: It is of great importance that the huge investments that are made in the infrastructure can be utilized during the entire lifetime of the structures. To reach this, finite element (FE) analysis has become an important tool for improved bridge assessment and maintenance based on realistic modelling of the structural response. Non-linear FE analysis has successfully been used to prove higher load carrying capacities for a number of Swedish bridges. However, in a FE model of a bridge, there are a lot of uncertainties, e.g. regarding boundary conditions and constraints between different parts of the structure. By performing field tests and measurements, many of these uncertainties can be limited using structural verification to update the structural model. In a research project at Chalmers University of Technology, methods are developed for improved assessment and maintenance of bridges by means of FE analysis with model updating from field tests and measurements.

On the highway between Göteborg and Oslo, the New Svinesund Bridge was constructed and opened to traffic in 2005. A single arch carries the main span over the fiord at the border between Sweden and Norway, see figure 1. When constructing the bridge an extensive measurement and field test programme was performed to check and follow up the construction process, and to get a better understanding of the as-built structure. In connection with this, a FE model of the bridge was developed and analysis of the bridge construction and test loading were performed. In the Chalmers project, the bridge will be used as a case study for the methods developed.

1 THE NEW SVINESUND BRIDGE

The single concrete arch that carries the main span of 247 m is the longest bridge in the world of its kind. The highway passes on top of two steel box-girder carriageways, connected through

Figure 1. An artists impression of the New Svinesund Bridge, seen from east. The main span consists of a single arch carrying one bridge deck on each side (from Vägverket (2005)).

integrated transversal beams. The arch is very slender and its lateral stability is obtained through the rigid connections to the carriageways. The arch was built using the cantilever construction method with temporary concrete piers over the abutments and cable-stayed supporting from the arch over the pylons to the rock behind. The carriageways were partly launched from one end of the bridge, and partly lifted in place, with the central part under the arch in one piece from barges on the fiord.

2 MEASUREMENT AND TEST LOADING PROGRAMME

Due to the unique design and importance of the bridge, a measurement and test loading programme is being carried out, including both the construction and the first 3–5 years of operating. Through the programme the design and construction is checked and verified and a better understanding of the as-built structure is obtained. The programme includes measuring of strains and temperatures in the arch, accelerations of the arch and carriageways and weather data. In connection with a test loading of the completed bridge, also deflections of the arch and carriageways and forces in the hangers were measured under static and dynamic loading by eight 25-ton trucks.

3 FE MODELLING AND ANALYSIS

To be able to evaluate the measurement results, a FE model of the bridge was developed. It comprises the complete bridge, including the temporary supporting structures. It is built up mainly by beam elements, complemented by truss and spring elements. It was checked through comparison with design and measurement results.

In a master’s thesis project the arch launching was analysed and compared to the measurements. The FE model was further developed to include all construction stages, the stiffness development and creep of the concrete and the temperature variations. It was found that the construction process had a large influence on the cross-sectional moments in the arch, with 50% higher moments in the abutments for the self weight when correctly modelled. The creep, stiffness evaluation and temperature variation during construction had only a minor influence here. The deformations during construction was hard to follow in the analysis, just like the strains in the arch. Different construction events can clearly be seen, but the continuously varying temperature made the strain variations over time complicated to evaluate.

For the test loading, FE analysis were made to determine the load positions, and to predict the response for each load case. After the test loading, a preliminary comparison with the measurements results was made. The loads in the hangers, as well as the arch deflections were found to correspond well. However, the strains in the arch did not. There are several possible reasons for this, ranging from measurement significance and accuracy, over temperature influences not reflected to errors and uncertainties in the model. This is currently being further evaluated.

4 CONCLUSIONS AND FURTHER RESEARCH

In the Chalmers project presently being carried out, methods will be developed for improved assessment and maintenance of bridges by means of FE analysis with model updating from field tests and measurements. Here, the New Svinesund Bridge is used as a case study. Data from the measurements on the bridge will be used to verify and update the FE model so that it can be used for maintenance and structural assessments during the bridge service life. The central objectives of this project are to perform a state-of-the-art investigation, to evaluate existing methods for FE model updating, to apply the methods to the Svinesund bridge, to investigate the possibilities to use FE analysis for enhanced life cycle analysis, and finally to draw general conclusions and recommendations.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Development of evaluation system for service life of concrete bridge deck structures

B.H. Oh, Y.C. Choi & J.B. Park
Dept. of Civil Engineering, Seoul National University, San, Shinrim-dong, Gwanak-ku, Korea

ABSTRACT: The purpose of the present study is to develop a realistic expert system which can estimate reasonably the safety and the service life of concrete bridge decks, based on the deterioration models that are derived from traffic loads and environmental effects. The deterioration models due to chloride ingress and carbonation are established. The damage models due to repetitive traffic loads considering dry and wet condition of deck slabs are proposed. These models are used to calculate the remaining life of bridge deck slabs. The prediction models for service life of deck slabs due to loading and environmental effects are developed. The models include flexure, shear, fatigue, corrosion, carbonation of deck slabs. The expert system for prediction of remaining service life is finally developed based on the theories established in this study.

1 INTRODUCTION

Many bridge decks suffer from serious deterioration due to repeated vehicle leads and environmental factors. The repeated vehicle loads cause damage accumulation due to fatigues. This damage accumulation becomes more serious as the number of overloaded vehicles increases recently. In addition to the vehicle loads, the bridge decks are also subject to serious environmental attacks such as chloride penetration, freezing and thawing, and carbonation etc. The purpose of this study is to develop a realistic method for accurate condition assessment of reinforced concrete (RC) bridge decks. The assessment includes not only the safety, but also the service life of bridge decks.

2 DEGRADATION OF REINFORCED CONCRETE BRIDGE DECKS

The corrosion initiation time, t_0, can be obtained from Eq. (1). by introducing $x = c$ (where $c =$ cover depth), $t = t_0$ and $C = C_{thr}$ (where $C_{thr} =$ threshold value of chloride for corrosion initiation).

$$t_0 = \frac{c^2}{4D_{Cl}}\left[\frac{1}{erfc^{-1}\{(C_0 - C_i)/(C_{thr} - C_i)\}}\right]^2 \tag{1}$$

where $C_0 =$ surface chloride concentration.

The cross-sectional area of reinforcing steel bar is reduced by the corrosion. The current area of steel bar, $A(t)$, may be written as follows.

$$A(t) = \begin{cases} nD_i^2 \dfrac{\pi}{4} & for\ t \le t_0 \\ n(D(t))^2 \dfrac{\pi}{4} & for\ t_0 < t < t_0 + D_i/(0.023 \cdot i_{corr}) \\ 0 & \text{for } t \ge t_0 + D_i/(0.023 \cdot i_{corr}) \end{cases} \tag{2}$$

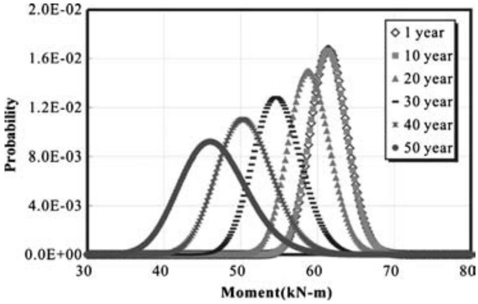

Figure 1. Distribution of time-variant moment strength of deck slab.

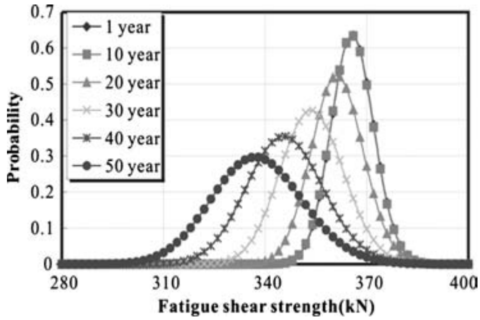

Figure 2. Variation of fatigue shear strength for deck slab with time.

where $n =$ number of rebar; $D_i =$ initial diameter of rebar; $t =$ current time; $t_0 =$ corrosion initial time; and $I_{corr} =$ corrosion rate parameter.

3 VARIATION OF RC DECK STRENGTH DUE TO CORROSION

The corrosion of steel bar reduces the area of rebar. The reduced area of steel bar can be calculated from Eq.(2) which needs to determine in advance the initiation time t_0. In order to calculate the initiation time, t_0, one needs to determine the concrete cover depth c, diffusion coefficient D_{cl}, initial surface chloride concentration C_0, and critical threshold concentration C_{thr}. In this study, probabilistic approach has been introduced to calculate the current area of steel bar. Monte Carlo simulation was introduced to identify the reasonable values for various design variables. The variation of moment strength based on the reduction of rebar area has been calculated as shown in Fig. 1. Fig. 2 summarizes the calculated probability distributions for fatigue shear strengths of a slab according to service period. It can be seen that the mean fatigue strength decreases with time due to degradation and the variability of strength increases.

The deterioration of deck slab due to corrosion of rebar has been considered to calculate the fatigue life. The fatigue life decreases with deterioration and is found to be lowest at the edge of girder flange.

4 CONCLUSION

The purpose of the present study is to propose a rational and realistic condition assessment system for deteriorating bridge deck structures. For this purpose, realistic load models acting on bridge decks as well as deterioration models have been developed. The load model considers the randomness of wheel loads, wheel locations on the deck, traffic patterns, and frequencies of occurrence of specific traffic patterns. The deterioration models include the chloride penetration, corrosion initiation, and area loss of rebar due to corrosion. The fatigue strength of deck slab under moving wheel load has been also established with due consideration of dry and wet conditions of slab. All the variables of the present load and deterioration models were treated probabilistically. The present study provides all the solutions automatically which give more realistic condition assessments and service life for reinforced concrete bridge deck structures.

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Applications of acoustical techniques for detection and assessment of damage in aging structures

K.Z. Zahariev, Y.B. Kin & B.W. Parsons
Purdue University Calumet, Hammond, IN, USA

ABSTRACT: The effect of natural frequency shift due to crack propagation through a steel plate was investigated. For the experimental measurements of the vibration signature, the impulse resonant spectroscopy (IRAS) method was used by utilizing software able to extract the frequency domain from the recorded vibration response by fast Fourier transform (FFT). An FEA mode-frequency analysis was conducted in order to gain more insight of the phenomenon considering the first three flexural modes of vibration. In addition, using the crack propagation history as input for fatigue analysis the remaining life of the part at each stage of damage was predicted based on the well-known Paris formulation. Both, the vibration and the fatigue analytical and experimental results showed very good agreement.

1 INTRODUCTION

In recent years, a large number of studies have been carried out on conventional (visual examination, dye penetrant, magnetic particle induction, ultrasonic, radiographic, et cetera,) and modern damage detection techniques for inspecting structures exposed to fluctuating loads, such as aircraft structures, automotive parts, structures used in ocean environment, buildings, bridges, pipelines and other industrial equipment. Cracks can be introduced in the structure during the manufacturing process or due to the limited fatigue strength of the material in the course of exploitation. During the work cycle the cracks open and close, and as the load reversals continue the cracks may develop to such extend that there is a significant threat for the integrity of the structure. Therefore, structures like these must be monitored for the existence and the development of cracks, by using non-destructive crack detection techniques, and also by implementing fatigue analysis to acquire the remaining life expectancy before a costly repair is needed to be made.

The current conventional damage detection methods are either visual or localized experimental procedures that require that the vicinity of the damage is known and that the site of inspection is easily accessible. Due to these limitations, it is believed that monitoring the global vibration characteristics of the structure is a promising alternative for damage detection and quantification.

2 RESULTS

The basic idea of vibration-based damage detection is that damage will modify the stiffness and may also affect its mass distribution and its damping properties, which will result in change of the dynamic response of the system. To justify this idea the experimental measurements of the vibration signature, the impulse resonant spectroscopy (IRAS) method was used by utilizing software able to extract the frequency domain from the recorded vibration response by fast Fourier transform (FFT). In addition a FEA mode-frequency analysis was conducted in order to gain more insight of the phenomenon considering the first three flexural modes of vibration.

The results from the IRAS test and the mode-frequency analysis are good evidence for the applicability of the method for indirectly measuring the crack length in the plate. Also, it was

Table 1. Life prediction for the center cracked specimen.

Current crack length, mm	Critical crack length, mm	C	m	Predicted life, cycles	Real life, cycles	Difference, %
4.75	7.70	8×10^{-104}	13.386	13,060	25,100	−48
5.31	7.70	1×10^{-42}	5.1397	8,770	17,500	−50
6.07	7.70	2×10^{-28}	3.1996	7,487	11,000	−32
6.75	7.70	2×10^{-17}	1.7132	5,794	5,500	+5

Table 2. Life Prediction for the offset crack specimen.

Current crack length, mm	Critical crack length, mm	C	m	Predicted life, cycles	Real life, cycles	Difference, %
3.00	7.22	2.00E−21	2.2296	77,982	62,800	24
4.07	7.22	2.00E−18	1.8289	42,147	36,000	17
6.16	7.22	8.00E−20	2.0184	9,626	9,400	2

confirmed that the change of the dynamic behavior of a cracked plate can be predicted by conducting vibration modal analysis. The modal shapes results from the Finite Element Analysis can give us visual perspective of the distribution of the "sensitive" zones in the specimen for each mode. An attempt to acquire these results experimentally would be much more time and equipment costly. This knowledge can help us determine, for a known location of the damage or the crack, the most sensitive vibration mode, for which the frequency should be monitored. Using information from acoustical test and simulation of vibration analysis with the help of FEA permitted to estimate remaining life of the parts loaded under cycling regime. The results of predicted life are given in Tables 1 and 2.

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Numerical modelling of damaged masonry arch bridges

J. Bień & T. Kaminski
Wroclaw University of Technology, Wroclaw, Poland

1 INTRODUCTION

A subject of this study is modelling and analysis of damaged masonry arch bridges by means of FEM. This approach provides accurate representation of damages of various localisation and size as well as reliable and measurable results. The paper describes a methodology for evaluation of damages' influence on the structural condition of various types of masonry bridges which is based on assessment of the load carrying capacity value.

2 FEM MODEL OF MASONRY BRIDGE STRUCTURE & DAMAGES' DESCRIPTION

The study concerns the most typical masonry bridges: single-span and rectangular in plane structures with segmental and single-ring arch barrel. Proposed FE models of such structures comprise a 2D plain strain problem representing 1 m width strip of a bridge (Fig. 1a). A specific approach to modelling of masonry material of the arch barrel is based on so called 'micromodelling' strategy. It assumes a complex model of the arch barrel composed of 40 masonry blocks and 1 cm thick mortar joints represented by 2D quadrilateral elements with individual properties (Fig. 1a). The backfill is modelled with 2D triangle elements. Blocks' material is linear with infinite strengths. Joints' material is nonlinear with substantially limited tensile strength. Such a method provides representation a hinge mechanism failure mode (Fig. 3). They appear within single joints by yielding of its material what simulates cracking. It is realized by means of so called 'concrete damaged plasticity' material model accessible in the applied ABAQUS system. All of the material properties and most of the geometric parameters have typical predefined values. Constant geometric characteristics are: $L_0 = 5$ m, $d = 0.45$ m, $h = 0.35$ m and the variable one: $r/L = \{1/6, 1/4, 1/2\}$ (Fig. 1a). Between the arch barrel and the backfill a contact surface is modelled with coefficient of friction: $\mu = 0.4$. Procedure of loading is composed of two successive phases: dead load (self-weight of the arch barrel and the backfill) and live load P (a single axle load in the form of UDL applied by rigid plate at quarter point of the span). The second phase is realized on a displacement-controlled loading path.

The load carrying capacity is defined as the highest value P_{ult} of P presented in Figure 1b.

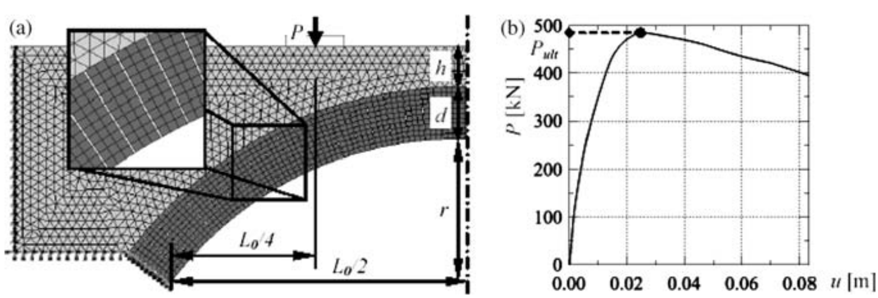

Figure 1. FE model of a masonry bridge (a) and exemplary load-displacement relation *P-u* (b).

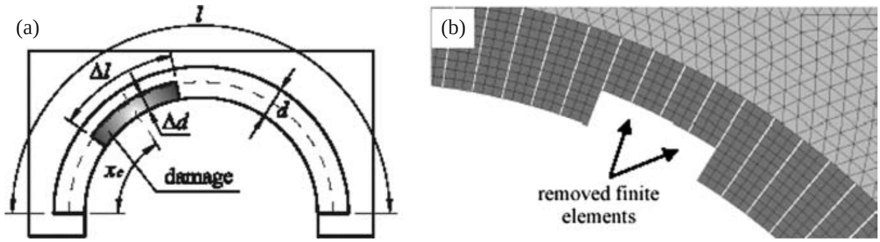

Figure 2. Parametric description of damages (a) and numerical representation of material loss (b).

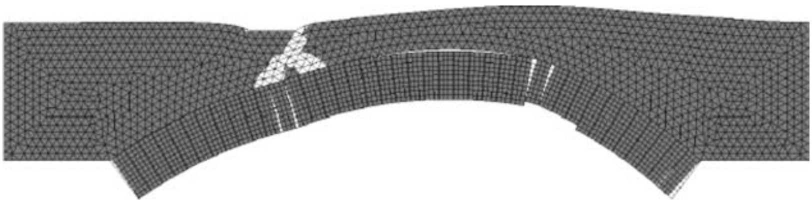

Figure 3. Final result of damaged bridge analysis with visible 4-hinge mechanism of collapse.

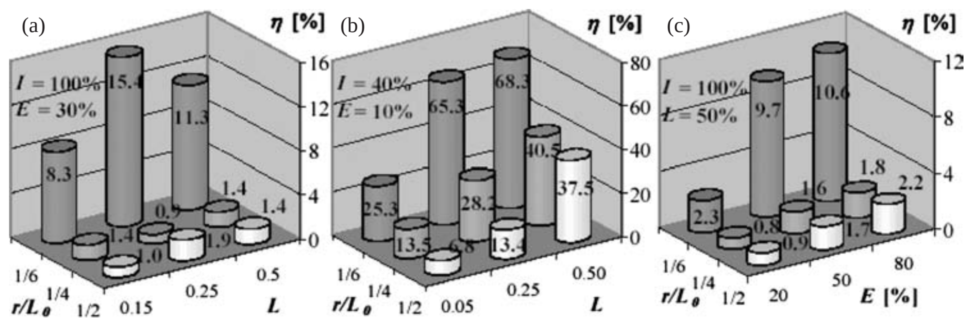

Figure 4. Influences of strength reduction (a), losses of material (b) and longitudinal fracture (c) on the load carrying capacity for various parameters: intensity I, extent E, location L and rise to span ratio r/L_0.

Considered types of damages – strength reduction, longitudinal fractures and losses of material – concern the arch barrel only. Damage parameters: location L (eq. 1), extent E (eq. 2), intensity I (eq. 3) and destruction level D (eq. 4, for strength reduction only) are given as follows:

$$L = \frac{x_c}{l}, \quad E = \frac{\Delta l}{l} \cdot 100\%, \quad I = \frac{\Delta d}{d} \cdot 100\%, \quad D = \frac{\Delta R}{R} \cdot 100\% \qquad (1),(2),(3),(4)$$

where x_c, Δl, l, Δd and d according to Figure 2a; ΔR = strength decrease; R = designed strength.

Strength reduction is modelled by decrease of strengths and modules of elasticity (by 50%). Longitudinal fracture is modelled by element thickness decrease representing reduction of the arch barrel width (equal to 0.8 m). Losses are modelled by removing of selected finite elements.

3 ANALYSIS OF MASONRY BRIDGES WITH DAMAGES

Selected results of analyses are presented in Figures 3, 4. Influence of considered damages is given on a percentage basis as a ratio ç of ΔP_{ult} (being the difference of the capacities for undamaged and damaged structure) to the load carrying capacity of undamaged structure P_{ult}.

Conclusion is that flat arches are the most sensitive to all analysed damages. The damages located near the arch barrel crown influence the load carrying capacity the most significantly. Generally relationships between capacity and damage parameters is not transparent nor linear.

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Bond-slip behavior of corroded reinforcing steel in concrete bridges

C. Fang
Department of Civil Engineering, Shanghai Jiaotong University, Shanghai, China

ABSTRACT: Virtually all highway bridges contain concrete elements. For concrete bridges, reinforcing steel provides strength and ductility through its bond and anchorage to the concrete, and the effectiveness of that can be reduced by corrosion of the embedded steel bar. Local bond aspect between corroded steel bars and concrete were experimentally investigated based on pullout and cyclic loading tests. Evaluation of the bond mechanism is especially useful when the main reinforcing bars are corroded. However, the aspects regarding different levels of corrosion on bond need further study. The purpose of this study is therefore to evaluate the effect of steel corrosion at different levels on bond behavior under both pullout and cyclic loading.

The reinforcement steel bars used were hot-rolled grade-II deformed bars. The geometry of the specimen is shown in Fig. 1. The 28-day cured concrete had an average measured strength of 52 MPa. Electric current was applied to specimens to accelerate the corrosion of reinforcement steel. The amount of corrosion products was theoretically calculated according to Faraday's Law. The actual amount of corrosion product was measured as the loss in weight of the reinforcement steel bar.

The tests were carried out on a testing machine as shown in Fig. 2. A specially designed loading frame was used for the loading. A crack opening displacement (COD) gauge was used to measure the slip more accurately when the slip was small, i.e. less than or equal to 2 mm while a linear variable differential transducer (LVDT) was used to measure the displacement during the total testing procedure.

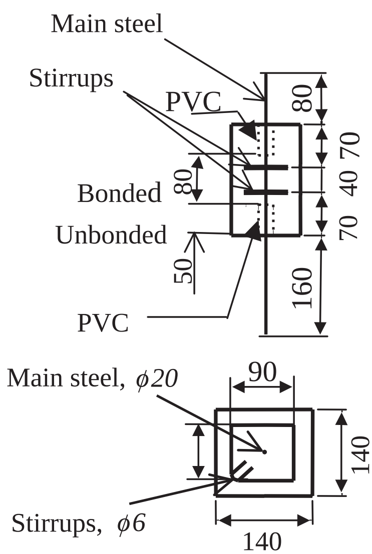

Figure 1. Geometry of the specimens. All dimensions in mm.

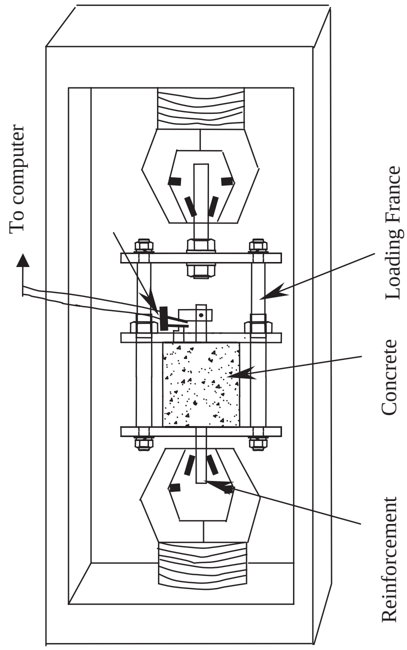

Figure 2. Pullout and cyclic loading setup.

The results revealed that when corrosion level was low the bond reduction was low while the bond reduction was significantly higher when corrosion level was high. Under cyclic loading, the bond capacity in unloading phase was only three-fourth of that in loading phase. Based on the results the following conclusions have been drawn.

(1) The bond strength generally decreased as the corrosion level increased, for both confined and unconfined bars. The exception was low corrosion levels in confined bars, where the bond strength did not show a significant reduction.
(2) Under both pullout and cyclic loads, the bond reduction was less substantial for confined bars than for unconfined bars. This showed that the confinement contributed substantial part to reduce reduction in bond strength. For confined bars, no substantial bond loss was observed when the corrosion level was low (in the range from 0% to 3%). For unconfined bars, bond strength decreased drastically as corrosion level increased. The change in bond strength for unconfined deformed bars was very drastic, but for confined deformed bars rather gradual.
(3) Under cyclic loading, the relatively high level corrosion contributed to reduction in bond primarily in the initial five loading cycles. This effect decreased with loading.
(4) The shape of unloading and reloading branches of bond stress-slip curves as well as the reduced envelopes depends on the loading history.
(5) The above findings and conclusions are preliminary, and the author plans to repeat some of these experiments in future research.

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West Mill Bridge – comparison of initial and long-term structural behaviour

Lee Canning & Sam Luke
Mouchel Parkman, United Kingdom

ABSTRACT: West Mill Bridge was the first all FRP composite on the public highway network in Western Europe, officially opened in October 2002. The bridge is 6.8 m wide and carries a single carriageway over a 10 m span. The bridge comprises a pultruded GFRP deck spanning transversely across, and adhesively bonded to, four main GFRP-CFRP hybrid composite beams. The bridge was instrumented with electrical resistance strain gauges and optical fibre sensors to enable monitoring of the structural behaviour.

Prior to the opening of the bridge, a load test was undertaken with a 28-tonne lorry to provide a datum for the structural behaviour, to be compared with later load tests. This paper discusses the results from the initial load test and a similar load test undertaken in July 2005. In particular, conclusions are drawn on

- the integrity of the adhesive bond and degree of composite action between the main beams and deck,
- transverse spanning behaviour of the pultruded GFRP composite deck,
- longitudinal spanning behaviour of the GFRP-CFRP composite hybrid main beams,
- the effective width of the GFRP composite deck.

In addition, the field data from the structural load tests are compared with the original theoretical predictions based on the design data.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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 ISBN 0 415 40315 4

Road bridge expansion joints: Existing systems and most common defects

A.J. Marques Lima
Brisa – Engenharia e Gestão; I.S.T., Lisbon, Portugal

J. de Brito
I.S.T., Lisbon, Portugal

ABSTRACT: Expansion joints are a current piece of equipment in today's road bridges. There are various types and dozens of makers spread all over the world. Notwithstanding their evolution particularly since the 1970's, there is a widespread feeling that their service life is low, even less than the one of pavements in which they are included.

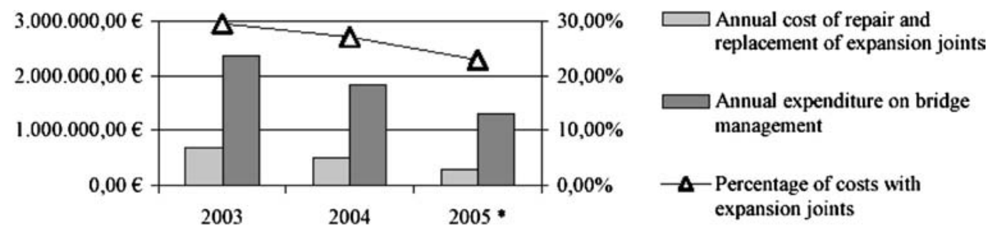
On the other hand, conservation costs can be considered as extremely significant. In the case study of the Portuguese highways' concessionary Brisa, presently with over a 1000 km of highways and 1500 bridges in service, in the last three years the cost of repair/replacement of expansion joints has reached average values of around 25% of the total expenditure on bridge maintenance (Fig. 1).

Even though difficult to quantify but surely significant, there are other costs associated with existing anomalies in expansion joints. From this point of view, indirect costs with rehabilitation interventions stand out since they almost always imply cutting off sections of the road and causing traffic bottlenecks.

In this context, the need for a progressively better management of this type of equipment becomes obvious leading to the creation of systems autonomous or integrated within the bridge management system.

The classification proposed in this paper considers twelve distinct types of expansion joints, according to their increasingly bigger allowed movement range. Criteria used to differentiate the various types of joints have to do essentially with their constituting materials and their morphology. A very succinct description of each type of expansion joint is presented in the paper.

Type 1 – Open joints
Type 2 – Buried joints under continuous surfacing
Type 3 – Asphaltic plug joints
Type 4 – Nosing joints with poured sealant
Type 5 – Preformed compression seal joints
Type 6 – Elastomeric flexible strips



* Costs estimated based on the values obtained between January and August.

Figure 1. Costs involving repair/replacement of expansion joints (data from Brisa).

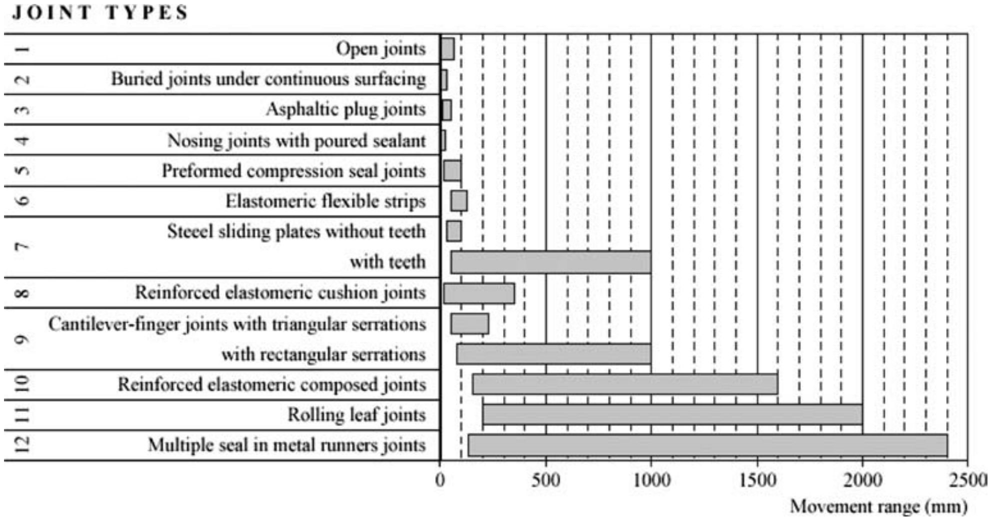


Figure 2. Range of longitudinal movements allowed by the various types of joints.

Type 7 – Steel sliding plates
Type 8 – Reinforced elastomeric cushion joints
Type 9 – Cantilever-finger joints
Type 10 – Reinforced elastomeric composed joints
Type 11 – Rolling leaf joints
Type 12 – Multiple seal in metal runners' joints

In Figure 2 a table is presented with the range of longitudinal movements allowed by each type of joint according to data gathered from the main makers that market them in Portugal.

Anomalies at the joints can be divided into seven distinct categories:

– transition to the pavement/pavement;
– geometry;
– movement;
– anchorage to the structure;
– joint/joint material;
– drainage;
– users' comfort.

The causes of these anomalies and their possible evolution with time are presented in the paper. The present paper has two distinct objectives: to establish a classification of the expansion joints installed in the universe of Portuguese road bridges, for which a succinct description and the possible longitudinal movement range of each type of joints proposed are presented; to identify the most common anomalies found in joints during the service stage and their relationship with the various types of joints.

A contribute to a better understanding of the expansion joints thematic has been given and some of the bases for the development of a management system of these equipments are presented within the scope of a Master thesis under preparation by the first author.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Experimental and analytical model analysis of Babolsar's steel arch bridge

M.H.A. Beygi
Civil Engineering Faculty, Mazandaran University, Mazandaran, Iran

M.T. Kazemi
Civil Engineering Faculty, Sharif University of Technology, Tehran, Iran

B. Lark
Civil Engineering Department, Cardiff University, Cardiff, Wales, U.K

Z. Tabrizian
Civil Engineering Faculty, Mazandaran University, Babol, Mazandaran, Iran

ABSTRACT: The paper presents the experimental and analytical model analysis of a steel-girder arch bridge. The field test is carried out by ambient vibration testing under traffic excitations. Both the peak picking method in the frequency domain and the stochastic subspace identification method in the time domain are used for the output-only model identification. A good agreement in identified frequencies has been found between the two methods. It is further demonstrated that the stochastic subspace method provides better mode shapes. The three-dimensional finite element models are constructed and an analytical model analysis is then performed to generate natural frequencies and mode shapes in the three-orthogonal directions. The finite element models are validated to match the field natural frequencies and mode shapes.

This paper concentrates on both experimental and analytical model analysis of a steel arch bridge over the Babolrood River located in Babolsar in Iran. Field model testing was carried out using ambient vibration testing under "natural" excitations induced by traffic & crossing truck (weight = 32 ton).

The main parts of Babolrood River Bridge are steel arches. The bridge is a steel arch with a length of 90 m. This bridge was designed about 70 years ago. The superstructure of the bridge consists of the vertical and lateral load carrying systems, and the deck system & the horizontal bracing system. The arch span consists of 15 diagonally braced members. The main attached on both sides of the arch and the floor system is suspended through these. The floor system consists of about 200 mm thickness concrete slab supported by five longitudinal stringers. The stringers are placed on the transverse built-up floor beams and the bearing consists of pin and roller combinations to allow rotation and translation. The top shoe of this bearing is connected to the bottom flange of the steel girder, which is then connected to the pin. The slots in the bottom flange of the steel girder allow translation are forced vibration tests and ambient vibration tests. Forced vibration tests are directly related to the application of standard techniques of experimental model analysis in which the structure is excited by artificial means such as shakers or drop weights.

Relatively long records of response measurements are required and the amplitude of measurements is relatively small in the ambient vibration test. The field model testing of the arch steel Babolrood River Bridge was carried out using the method of ambient vibration. The equipment used to measure the response consisted of triaxial seismometers linked to their own data acquisition system. The system contained digital recording strong motion velocity graph. A lock was positioned at each station with the seismometers oriented in the vertical, transverse and longitudinal directions. Seismometers were connected to the data acquisition system by shielded cables & connected to GPS. All measurements were taken by placing the instruments on the pavement due to the limited access to the actual floor beams and the testing time constraints involved. Measurement stations were chosen at each joint (panel point) of suspenders connected to the deck. As a result, a total of 16 locations (8 points per side) were measured.

Three test setups were conceived to cover the planned testing area of the arch of the bridge. A reference location, here in after referred to as the base station, was selected based on the mode shapes from the preliminary finite element model. Each setup consisted of four base triaxial seismometers stations and four moveable triaxial seismometers stations.

Data from three test setups for each of the right-hand lane and left-hand lane were measured. Once the data were collected in one setup, the moveable stations were moved to the next locations while the base stations remained stationary. Each setup yields a total of twelve sets of data from moveable stations and twelve sets of base station data. This sequence was repeated three times to get measurements on all stations. The sampling frequency on site was chosen to be as high as 1,00 Hz to capture the short-time (higher-frequency) transient signals of the ambient vibration in detail. The ambient vibration measurement was simultaneously recorded for 360 s at all seismometers, which resulted in total 18000 data points per data set (channel). During one of the tests, normal traffic was allowed to flow over the bridge at normal speeds. The sampling rate of 1,00 Hz is too high for the frequency range of interest. However, with the development of data acquisition systems of high computer speed and capacity, there is no difficulty in working with a high sampling rate. Re-sampling and application of a low-pass filter can be easily done by the digital computer and software in the office, but the field test is not easily repeatable. A higher sampling rate in the field provides the possibility to study the effect of sampling rate on the extraction of the model properties.

Ambient excitation does not lend itself to FRFs or IRFs calculations because the input force is not measured in an ambient vibration test. Therefore, a model identification procedure will need to base itself on output-only data. Two complementary model analysis methods are implemented here. They are the rather simple peak picking (PP) method in the frequency domain and the more advanced stochastic subspace identification (SSI) method in the time domain. Although the input forces are not measured when performing ambient vibration measurements, this problem can be circumvented by adopting an adapted model identification technique. In this technique, the base station (reference) signal is used as "input" and the corresponding FRFs and coherence functions are computed for each response measurement point with respect to this station.

The measured data are first de-trended which enables the removal of the dc components that can badly influence the identification results. A 1,00 Hz sampling frequency on site results in a frequency range from 0 to 50 Hz. For most bridges, however, the frequency range of interest lies between 0.5 and 15 Hz and contains at least the first ten natural frequencies. For the peak picking method in the frequency domain, the ANPSDs for all measurement data without decimating are shown in Fig. 8. The peak points are clearly shown and then the frequencies can be picked up. Though the PP method is fast and provides a reliable identified natural frequency in most cases, it sometimes cannot yield enough good mode shapes.

Three-dimensional linear elastic finite element models of the arch span of the Babolrood Bridge have been constructed using *LUSAS 13.5* finite element analysis software. The model is developed for both the analytical model analysis and earthquake response analysis, and represents the structure in its current as-built configuration. The arch members, girders, stringers, floor beams, and bracing members are modeled by two-node beam elements that have three translational degrees of freedom (DOFs) and three rotational DOFs at each node.

It can be seen that the displacements in experimental method higher than analytical model. The FE analytical model analysis was validated by experimental model analysis in terms of natural frequencies and mode shapes. Theoretically, a perfect model would match all experimentally determined mode shapes and frequencies exactly. In practice, it is not expected to be a perfect match between all analytical and measured model properties. Therefore, only the most structurally significant modes and frequencies are used in the comparison process. In addition, the higher modes identified through ambient vibration measurements are not reliable since the higher modes are not excited sufficiently.

To facilitate the seismic evaluation/retrofit of the Babolsar River Bridge on, the dynamic properties have been studied by analytical model analysis with the 3D finite element method and by experimental model analysis with ambient vibration testing. A relatively simple model testing procedure on a real and large bridge under actual working conditions is presented. The potential usefulness is being able to verify analytical models and to monitor the health performance for large bridges.

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Study on safety alerting system of beam bridge

Y.Q. Xiang, J.F. Wang & W.L. Yang
Dep. of Civil Engineering, Zhejiang University, Hangzhou, Zhejiang province, P.R.C.

ABSTRACT: In China, most of the bridges in service are beam bridges. Studying and developing safety alerting system for beam bridges is very practical meaningful for ensuring their security and improving the service level. On the basis of the description on the alerting and health monitoring technology of bridges at present, the overall design and some key technologies were discussed, such as monitoring scheme, data gathering and transmitting, signal disposal, database design, security index system construction, comprehensive assessment strategy etc.

1 INTRODUCTION

The bridge is inevitably corroded by the environment and harmful substances in the construction and the service stage. Besides, the outside actions, such as vehicle, wind, earthquake, fatigue, man-made actions, the degradation of the material's performance etc, will also lead to various degrees of damage and retrogression before the members reach their designing service life. If the damage can not be detected and repaired in time, the traffic safety and the bridge life will be influenced, and even worse. It can lead to abrupt destroy or collapse of the bridge. According to the bridge investigation data, by the end of 2003, there are 310773 bridges in service in china, the total length of bridge is 12466143 m. And 10443 bridges in unsafe were found, the length of which is 378439 m, among which the beam bridge occupies a considerable proportion. The security problem of beam bridges is outstanding. In this paper, the conceptual design and some key technologies of the safety alerting system for beam bridges are discussed.

2 OVERALL FRAME DESIGN AND SOME KEY TECHNOLOGIES

The main purpose of the alerting systems for beam bridges is to predict its working behavior and detect its damaging state according to the responses and monitoring results of the structure's performance under loading. Some warning and corresponding measures should be brought forward for the potential damages. According to the analysis of function request and the goal, the safety alerting system for beam bridge generally consists of the sensor subsystem, data gathering and transmitting subsystem, data processing and control subsystem, security alerting, evaluation and decision subsystem etc.

The design of sensor subsystem is the establishment of the monitoring scheme. The main work includes the choice of monitoring items, monitoring locations and sensors etc. The monitoring items are usually classified into the site environment, the material characteristics and the static and dynamic responses. Therefore, it is advisable to adopt nondestructive inspection techniques to avoid accessional structure damages in the process of data collection. Different beam bridges have different mechanical behavior. Hence, their monitoring locations and monitoring items may be different according to types of beam bridges and the structural features in the design and construction stage. The elected instruments and detection methods should be mature, reliable and durable.

In data gathering and transmitting subsystem, signals of electricity, light, sound, magnetism and so on can be transformed to digital signal and then be transmitted to centre database to be treated by corresponding techniques. If the bridges are far away from the city or there are plenty of separated bridges to be monitored, the signal should be transmitted through wireless networks such as GSM and GPRS etc.

Data disposal and control subsystem constitutes data disposal, control server and the corresponding software systems.

Safety alerting, evaluation and making decision subsystem are involved in the establishment and analysis of the structural finite element model, the establishment of the security index system, damage diagnosis, safety alerting and comprehensive assessment strategy etc.

An accurate 3D FEM adapting to damaging detection must be constructed in order to know the influence of damage on the bridge performance quantitatively. Nowadays, the analysis efficiency and the analysis precision are incompatible in the analysis of large structures. The basic FEM mode of complicated bridge structure can be constructed by the degenerated solid elements method (Ling & Xiang et al, 1998). The internal force and state of the bridge will change when it has serviced for years or undergone an accident, especially the terrible natural disaster. The change of the stiffness, boundary conditions and material parameters will take place to some extent. Through comparing the test data of monitored bridge with the analytical ones, the change of some parameters will be identified by the methods based on frequency, mode and their derivatives such as modal flexibility and modal curvature; the method based on frequency response function; the method based on strain mode and the non parameter methods based on measured dynamic data such as neural network method and genetic algorithm etc. The working state and safety state of the bridge can be assessed or some warning is sent by the variable weights assessment theory based on AHP according to above monitoring information and analysis data.

Structure alerting can be divided into two steps of the primary alerting and structural alerting. Primary alerting is set up in the data collecting element. When the pre-process result is beyond the pre-setup limitation, the primary alerting program will automatically detect the parameters related to the sensors and the output of other sensors to judge if the overrun has happened. At the same time, the gathering unit will send a request to the control center to prior transmit and deal with the data.

The structural alerting system is setup in the bridge assessment system in the control center and threshold alerting strategy is adopted. A 3-level alerting system can be actualized aiming at stresses, crack and alignment of bridge deck. The safety alerting system will start up corresponding function in different level of alerting condition to make sure that the system is effective.

3 CONCLUSIONS

The construction of safety alerting system for beam bridges is very practical meaningful in guaranteeing the traffic unblocked and structure safety as well as improving the service level of bridges. The technology is involved in the crossing and fusion of knowledge on multi disciplines. The key techniques are the design of the beam bridge alerting system, the monitoring scheme, data collection and transmission strategy, signal resolving method, database design, the basic finite element model, the security index system, safety alerting and comprehensive assessment strategy etc. It is recommended that the special design for a safety alerting system of bridge should be done by the corresponding experts under the guideline of overall design.

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Comparison between damage detection methods applied to beam structures

R. Salgado, P.J.S. Cruz, L.F. Ramos & P.B. Lourenço
Civil Engineering Department, University of Minho. Guimarães, Portugal

1 INTRODUCTION

The structural evaluation of bridges has become an essential topic since many of these structures have achieved their service life and some damage may occur. The current methods for detection of damage are visual inspection and local non destructive evaluation (NDE) methods. In order to be efficient, these techniques need a priori global location of the damage and easy access to the damage zone. For global damage detection, methods based on dynamic monitoring have been proposed. They can be categorized into four different levels: 1) detecting if the structure is damaged; 2) finding the location of damage; 3) estimating the severity of damage and 4) evaluating the remaining service life of the structure. A review of those methods was provided by Sohn et al (2003).

The damage detection methods evaluated in this study were: COMAC, curvature, damage index, discrete wavelet analysis (details), continuous wavelet transform, wavelet packet transform and Holder exponent methods. All of them are level 2 methods. Also, level 1 methods were used as comparison between principal frequencies and MAC method. The comparison was done on the first five mode shapes of the structures.

2 BRIDGE STRUCTURE

The bridge considered for the evaluation of the damage detection methods was designed following the AASHTO bridge code 1994, Salgado (2000). It is a composite simply supported bridge with two steel I beams (HE800B) and concrete slab with 300 MPa of compressive strength and 215 mm of depth. Its total length is 20 m.

3 DAMAGE SCENARIOS

In this study, damage was simulated with an open cracks located in the mid-length region of the steel I beams. Firstly, the severity of damage was considered with four crack depths. In the second general damage case, the extension of damage was evaluated using three crack patterns. The analyzed bridge was modeled as one dimensional simply supported composite beam according to the Euler Bernoulli hypothesis. Only heavy traffic was considered in the simulations. The dynamic response for cracked beams is obtained with the procedure proposed by Salgado et al (2005) which considers that cracks cause a local change of stiffness near to the location of damage. In some cases, artificial noise was added to the dynamic response in order to simulate errors during the acquisition of the data. The noise level used was 1.0%, referred as N1.

4 DAMAGE DETECTION METHODS

4.1 *Methods of level 1*

These results show us that change in frequencies method was able to identify damage only for the most severe damage scenario I4. Besides, damage could be also detected for the damage scenario

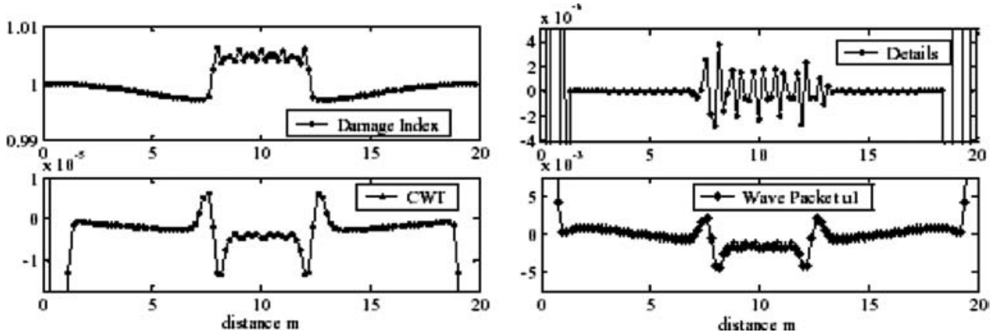

Figure 1. Typical comparison of level 2 damage detection methods (case I2D3 without noise is shown).

I3 and damage pattern D3. MAC method cannot distinguish if damaged mode shape vectors have enough variation for being able to identify the damage in most of the cases. Only in the damage scenario D3I4, this method showed values that point out possible damage in the structure.

4.2 *Methods of level 2*

4.2.1 *Comparison of damage detection methods of level 2*

Curvature, DI, details, CWT and WPSu1 method could successfully identify damage location for all the cases. COMAC, Holder exponent and WPSac1 method were less precise in the damage location identification. If a noise level of 1.0% is added to the dynamic response, damage identification is evidently affected in all the methods. Details and CWT could not identify damage in any case. It is because damage identification is done in the finest scales composed by high frequencies, the same as the added noise which hides the singularities peaks. Curvature, DI, WPSu1 and WPSac1 could identify the damage for the most severe damage cases. COMAC method showed in many cases local perturbances but they were not in the damage region, indicating false detections. In the Figure 1 an example of the damage detection methods is shown.

5 CONCLUSIONS

With respect to level 1 methods, changes in resonant frequencies and MAC method were found to be not reliable damage detection methods. Changes in resonant frequencies method was successful only for the most severity damage cases, whereas, MAC did not provide a clear change in the evaluated mode shape vectors. Level 2 methods successfully identified the damage location when noise was not evaluated. Exception were COMAC, Holder exponent and WPSac1 methods. When noise was added to the dynamic response, an abrupt decrement in the damage location for all the methods was noticed. Curvature, DI and WPSu1 showed the best behaviour when noisy data information was given.

*Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)*

Predictions of crack width for prestressed concrete deck slabs in box girder bridges

B.H. Oh & Y.C. Choi
Dept. of Civil Engineering, Seoul National University, Gwanak-ku, Korea

ABSTRACT: The purpose of the present study is to investigate the crack width of transversely post-tensioned concrete decks in PSC box girder bridges. Full-scale box girder members were fabricated and tested. Major test variables include the magnitude of precompressive stresses and prestressing steel ratios. The crack widths and deflections of deck slabs were measured automatically according to the increase of applied loads. The strains of prestressing steels and nonprestressed rebars were also measured during the loading process. The bond characteristics of post-tensioned prestressing steels were identified from the relation between prestressing steel strains and nonprestressed ordinary rebar strains under the same loading conditions. The bond effectiveness of prestressing steel was then obtained from these test data. The proposed formula is compared with the present test data and design code equations.

1 INTRODUCTION

The crack widths must be limited to certain allowable values especially in PSC structures under service loads. For a crack width equation of prestressed members to be rational, the bond characteristics of both prestressed and nonprestressed reinforcements must be considered. It is generally difficult to evaluate the local bond behavior of prestressing tendon because the tendons are partly in contact with the duct and partly with the surrounding concrete. It is therefore necessary to identify the effective circumference of multi-strands in which the strands are bonded to concrete. The purpose of the present study is first to investigate the bond behavior of prestressing steel and then to propose a realistic formula for predicting maximum crack width of PSC deck slabs.

2 TESTS OF FULL SCALE BOX GIRDERS

Four full-scale concrete box girder members have been designed and fabricated to investigate the crack width and crack spacing of deck slab in PSC box girder bridges. The transverse width, longitudinal length and total height of test members of concrete box girders are 6,500 mm, 2,300 mm and 1,600 mm, respectively. The main design variables include the magnitude of precompression, diameter of steel bars, nonprestressed steel ratio and prestressing steel ratio in the top slab of box girder bridges. The magnitudes of precompression due to lateral prestressing are 0, 2.35, 3.52, and 4.70 MPa, respectively. This is to see the effect of prestressing on the control of crack width under service loads. The yield strength of mild reinforcement was 398 MPa from material tests in the laboratory. The average compressive strength of concrete was 40 MPa. The yield and ultimate strength of the strands were 1,600 MPa and 1,892 MPa, respectively. The test members were loaded in four-point loading conditions as shown. The load was applied to the test members using the automatically controlled actuators. The linear variable differential transducers (LVDTs) were installed to measure the deflection profiles of the top slab of the box girder. In order to monitor the flexural crack widths under applied loads, the crack width gages were attached over the cracks after reading the initial values of crack widths with microscopes.

3 TEST RESULTS AND ANALYSIS

When prestressed steels and nonprestressed rebars are simultaneously used, different amounts of steel stresses are expected to develop in prestressed steel and ordinary rebar because the bond behavior of prestressing tendons is different from that of reinforcing bars. It can be seen that fairly good linear relations are obtained between the increments of tendon stress and ordinary rebar stresses. The increment of tendon stress is about 46.5 percent (0.465) on the average of that of ordinary nonprestressed rebar stress. This indicates that the effectiveness of bond property of prestressing steels is much lower than that of ordinary ribbed bars.

4 DEVELOPMENT OF CRACK WIDTH FORMULA AND COMPARISON WITH TESTS

The crack width may depend on many factors such as steel stress, effective concrete area in tension, nonprestressed reinforcement type and ratio, prestressing steel type and ratio, and strength of concrete. If time-dependent effects are considered in the PSC members, the stress of nonprestressed rebar will increase due to creep deformation of concrete. In this study, the increase of ordinary steel stress due to time-dependent effects after prestressing has been taken care of and only the increment of steel stress after decompression has been considered to compare with crack width increase according to load increase. The realistic prediction equation of crack width is derived in this study as follows.

$$w_{max} = 3\times10^{-6}\left(f_s - 40\right)\phi_s\left(\frac{A_{t,eff}}{A_{st}+\xi A_{pt}}\right)^{0.75}\frac{h-x}{d-x} \tag{1}$$

$$\xi = \sqrt{\frac{\tau_{ap}}{\tau_{as}}\frac{\pi+(n-1)}{n\pi}\frac{\phi_s}{\phi_p}} \tag{2}$$

where w_{max} = predicted maximum crack width in mm, A_{st}, A_{pt} = total area of prestressing steels and reinforcing bars, x = neutral axis depth of cracked section in mm, h = height of cross section in mm, d = effective depth in mm, ϕ_s = diameter of rebar in mm, ϕ_p = diameter of PS strand in mm, f_s = increment of rebar stress after decompression in MPa, f_0 = steel stress at the initial occurrence of crack in MPa, and τ_{ap}/τ_{as} is 0.465 for post-tensioned tendons which was obtained from this study. Here, (h-x/d-x) was introduced to represent the crack width at the bottom face of flexural members.

The proposed crack width formula has been compared with the present test results. The proposed equation agrees very well with test data for all stress range of prestressed and nonprestressed members while current code equations exhibit large deviations from test data.

5 CONCLUSION

The longitudinal cracking at the bottom face of upper decks of PSC box girder bridges have been frequently observed and this may cause some serviceability and durability problems in those bridges. The lateral prestressing in the top slab of such PSC box girders may reduce or eliminate the cracking at service load range. Full-scale box girders have been fabricated and tested. The bond effectiveness of prestressing steels was obtained from the present test results and it was found to be about 0.468 compared with nonprestressed ordinary rebars. The effective circumferential perimeter of multi-strands in a duct which is bonded to concrete is derived and used to determine the crack width under applied loads. A realistic prediction equation for crack widths was derived in terms of effective combined reinforcement ratio, diameter of rebar, and the rebar stress after decompression. The proposed equation correlates very well with test data.

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Structural damage analysis for SHM system design of PC girder bridge with losing of prestress

D. Dan, L. Sun & J. Li
State Key Laboratory for Disaster Reduction in Civil Engineering, Tongji University, Shanghai, China

ABSTRACT: Structural damage can be generally regarded as degeneration of working condition or performance degradation. For the designing a structure or its SHM system, it is significant to predict the structural damage under various mechanical and physical conditions as well as the mechanical behaviors of structure with damage. The process of structural damage analysis of PC girder bridge under prestress losing condition was studied in this paper. Demonstrated by a PC continuous girder bridge segment in Donghai Bridge, the losing evolution rules of longitudinal pretension in girder under operation condition was investigated, accordingly, its influence on the response of bridge in some key sections of the beam and the monitoring points were studied. The evolution laws of bridge static properties (Stiffness and Resistance) were discovered in order to give a deep understanding on structure static behaviors. The evolutions of the dynamic properties (Modal parameters) due to the losing of pretension were investigated too, which aims at detaching this variance from the one caused by local damage. The influence on statistical character of the acquisition data were explored, as a result, some statistical data processing routine can be designed under the guide of these apriority knowledge.

1 GENERAL INSTRUCTIONS

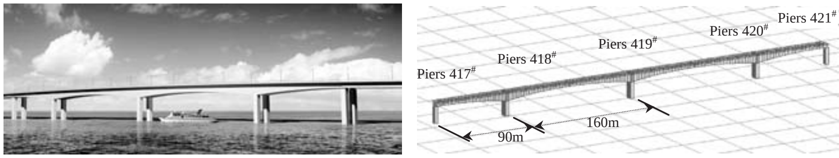
There exists a blindness and unreason trend in the designing and implementing of SHM systems nowadays. Many SHM systems are designed and implemented in a hurry in spite of having such following problems studied sufficiently. In fact, before a SHM system being designed and implemented, we should firstly to give a deep understand on the structural behaviors. In this study, a counterplan for SHM research named structural damage analysis (abbreviated as SDA) is proposed, which is aimed at solving above-mentioned problems.

Continuous segmental prestressed concrete bridges have been used extensively all over the world. The loss of pretension is a main problem which infects the health of this kind of structures. The SDA for pretension losing is also needed to understand the mechanical behaviors and to design a rational SHM system. This paper makes a case study on the pretension loss of a $90 + 160 + 160 + 90$ spanned PC continuous bridge.

2 GENERAL THEORY OF SDA

The proposal SDA should be composed of the following tasks: 1) to forecast the possible type, location, and grade of structural damage under various condition; 2) to analysis the evolvements of existent damage or possible damage in structure; 3) to analysis the changes of mechanical properties and the responses of structure under various damage situations; 4) to relate the damage and mechanical response of structure. Aimed at the conceptual design of bridge SHM system, the objectives of SDA should embody as: First, to find the locations of possible damage for locating the sensors on structure in the process of SHM system initial design; secondly, to grasp the changes of structural properties, so as to offer guidance for structure modal parameters identification and damage identification. Finally, to build the relationship between damage and monitoring quantities, in order to find suitable monitoring information processing method and structure state evaluation method. Two routines can be followed by SDA, one is the previewed evaluation routine. Another analysis routine of SDA is online evaluation routine.

3 PRETENSION LOSS SDA AND ITS APPLICATION ON SHM DESIGN



(a) Vice-sea-route transit bridge of Donghai bridge (b) Global model

Figure 1. The model of 90 + 160 + 160 + 90 spanned PC continuous bridge (Modeled by Midas 6.7.1).

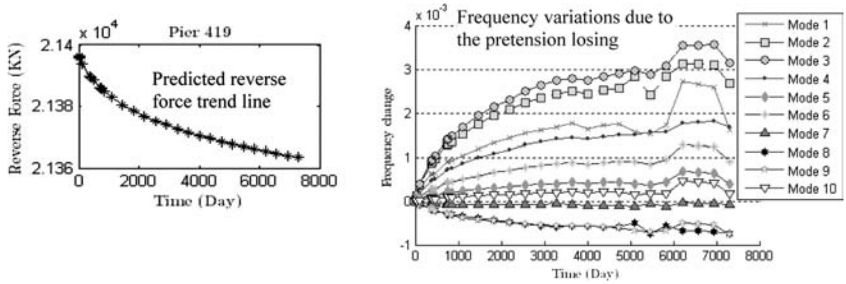
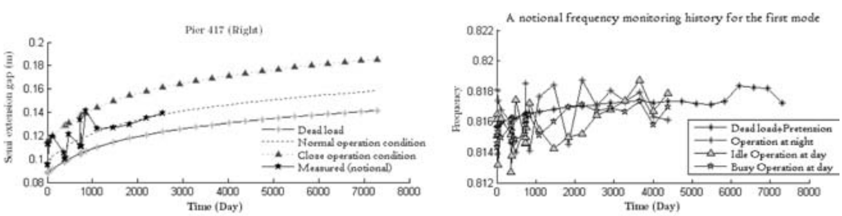


Figure 2. Static responses and dynamic properties variations under the long time pretension loss, creep and shrinkage condition of PC continuous bridge.



(a) Change of semi-extension gap (d) The history of frequency of first mode

Figure 3. Time-dependent dynamical baseline for health evaluation.

4 CONCLUSION AND DISCUSSION

The result showed that the damage due to the loss of pretension can affect both the static response and dynamic properties of the structure gradually. For a long time view, the pretension loss can lead to the low order mode frequency a little increase, whereas, the high order mode frequencies take the opposite tendency. Theoretically speaking, the pretension loss can affect the basic frequency in several percent, which is a relative extent to conceal the variations caused by other reasons. Moreover, the trend that low order frequency goes up as the loss of pretension is remarkable because it is opposite to that of ordinary stiffness damages done.

Structure damage analysis is a useful methodology in structural evaluation and design. There are no novelty mechanic analyses methods are concerted in the process of SDA, almost all the techniques in the SDA routines are traditional, and they are arranged from a new viewpoint which is differed from the traditional design based analysis routines. The results service to the SHM or other evaluations missions. SDA is a combination and of inverse and positive analysis.

*Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)*

Masonry arch railway bridges in Austria: Sustainable historical structures for today's traffic

M. Mautner
Austrian Federal Railways – OBB Infrastruktur Betrieb AG, Vienna, Austria

ABSTRACT: Masonry arches are among the oldest types of load-bearing structures still in service, although of increasing axle loads, train speeds and greater volume of traffic.

Since most of the railway lines in Austria date from the 19th century, the Austrian Federal Railways are constantly faced with the task of maintaining and repairing its large stock of arch bridges.

This paper provides general information on Railway Arches in Austria.

A short overview of the masonry arches, of the Austrian Federal Railways is given. Some statistics on the masonry arch railway bridge population are shown such as number, span length, span number, condition, age and material of masonry arch bridges.

Natural stone was regarded to be the most suitable material because of its positive properties and availability.

Short descriptions of the original historical design rules up to actual considerations, evaluating load capacity, are mentioned.

It was not only the design rules that give understanding in the structural layout also the design of the making of was the most sensitive affair. How cost intensive and detailed designed the construction of centrings had been. The static calculation of the entering was more comprehensive than the graphical evaluation of the arch itself.

The relationship between the main causes that lead to damages and the resulting effects are listed up. Masonry arches are considered to be the most reliable structures of the bridge population. In general the budget for maintenance and repair were used for more urgent repairs on other constructions. Over the years masonry arches have provided their services. Only their age and the great number bring to mind the importance to improve the lost experiences.

The influence and magnitude of load distribution and fill interaction are discussed by experts. However the changes and modification of the boundary conditions during the last hundred years could offer different results.

The development of actions due to traffic are mentioned is this paper and related to the structures of masonry arches. In combination with the effects due to aging and weathering the strategy of arch management has to be completely rethought, if lifetime should be prolonged another fifty years or more.

Important aspects shown in this paper is the condition of the track as load carrying system. The elastic behaviour of track is influencing the arch. Sleeper pads or ballast mats may be used to get more elasticity.

Inter alias the paper presents to examples of masonry arches of the Austrian Federal Railway to be representative for a typical situation in practise.

Viaduct of Mattersburg on OBB – Line Wiener Neustadt – Sopron (Hungary):

Constructed 1847, this viaduct made of bricks is one of the oldest and monumental structures with a total length 280 m, consisting of 20 arches. In this presentation a short summary of the damages and their repairs are described.

The knowledge of structural history and the existence of original documents may be important to understand the causes of damages.

Weidenbach Arch on OBB – Line Vienna to Hohenau (Tech Republic):

Constructed 1851, the arch made of bricks has a span length of 5.8 m and a height of fill of 0.8 m.

Due to damages cause by dynamic effects a mass-spring system has successfully been implemented.

Circumstances are mentioned that lead to this solution. A short description of the elements of the mass-spring system is listed up.

Finally the paper provides some aspects of new gained experiences in the field of NDT techniques and measure systems. Methods of repair have to be adopted due to the increasing dynamic influences.

Monitoring systems and non destructive material tests could help to understand the actual condition of the arch and perhaps the ongoing changes.

In Europe about 200,000 masonry arches are still in service. The infrastructure managers have to be aware that masonry arches do have a limited live time. This might be 150 or 200 years, or more?

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Numerical modeling and assessment of the shear key problems of FC girder bridges

N.A. Khattak
Colt Engineering Ltd., Edmonton, Alberta, Canada

J.J. Roger Cheng
Department of Civil & Environmental Engineering, University of Alberta, Edmonton, Alberta, Canada

ABSTRACT: More than 40% of FC girder bridges in Alberta have undergone rehabilitation. The progressive deterioration of the field grouted longitudinal shear keys eliminates the load sharing between the girders and hence subjects the girders to possible overloads. A number of rehabilitation schemes and their combinations have been designed and applied on these bridges. However, there doesn't seem to be a consensus on a particular favourable strategy for this kind of a bridge system. To get a better understanding of the FC girder bridge behaviour and the performance of different rehabilitation schemes used in the past, a detailed finite element study was undertaken, and is being presented in this paper. The study indicated that all rehabilitation strategies improved load sharing among the girders. However, a combination of transverse prestressing and transverse steel underslung diaphragms was the best rehabilitation technique to avoid future shear key cracking.

1 INTRODUCTION

Shear key based bridge systems using channel section girders were built in large numbers in the province of Alberta in Canada in 1960s to 1980s. These bridges are classified by Alberta Transportation as FC girder bridges, as shown in Figure 1. The FC girders are prestressed, precast U-shaped girders and have typical span length ranges from 20 m to 38 m. More than 40% of these bridges have been rehabilitated for deterioration in the shear keys which result in longitudinal cracking in the bridge deck and hence affect the load sharing among the bridge girders. A number of rehabilitation schemes have been applied to arrest the shear key cracking phenomena. Most rehabilitation techniques have been somewhat successful in improving the load sharing between the girders but the reappearance of longitudinal cracks in the bridge deck after some time indicates that consistent success has not been achieved because of a lack of a broad available knowledge base on the analysis of these rehabilitation schemes and their influence on the performance of the bridges.

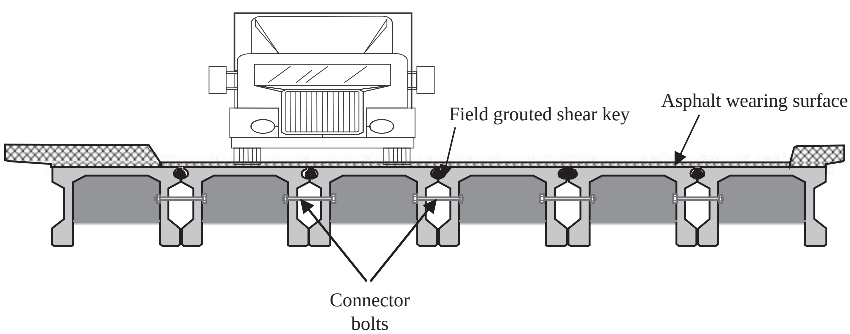

Figure 1. A typical FC girder bridge.

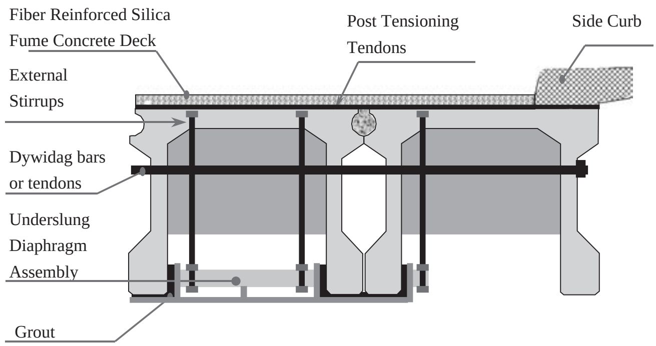

Figure 2. Schematic showing some rehabilitation schemes.

2 NUMERICAL MODEL AND ASSESSMENT OF FC GIRDER BRIDGES

The finite element model for this study was calibrated with the help of a number of field tests, using static and dynamic load tests and ambient vibration tests. The success of a particular rehabilitation scheme was measured on the basis of its response to certain assessment parameters. A 32 m single span, six adjacent girders bridge was selected to prepare the finite element model because similar span and geometry considerations occurred in most of the bridges that were tested in the field.

A total of 180 FC bridges have been constructed in Alberta, Canada, of which 76 bridges have been rehabilitated for shear key problems. Some of rehabilitation schemes used in the rehabilitation are shown in Figure 2. The rehabilitation schemes that were selected for comparison and were applied on the finite element model are:

- Additional connector bolts to tie adjacent girders;
- A non reinforced, fully composite concrete deck over the girders;
- Transverse prestressing of the bridge cross section;
- Transverse stiffening (underslung diaphragm shown in Figure 2) of the bridge cross section;
- Transverse prestressing and transverse stiffening combination.

The comparison of different rehabilitation schemes is based on the results of two parameters; differential deflections at shear key locations and natural frequencies of the transverse bending mode shapes. Three independent test measurements of deflections, strains and natural frequencies were used to compare the model to the test.

The use of transverse post-tensioning was found to be a useful alternative but depended heavily on its positioning on the webs of the girders. The use of continuous steel underslung diaphragms for the transverse stiffening of the system showed that by decreasing the spacing and increasing the sectional properties of these diaphragms, differential deflections at the shear keys could be successfully arrested. The combination scheme consisting of using the steel underslung diaphragms in combination with transverse prestressing extracts the benefits of both systems, provides an element of redundancy in the system and allows the use of less conservative variations of the individual schemes, when they are to be used together.

3 CONCLUSIONS

It was concluded that the combination of prestressing applied under the flange of the girders and continuous steel underslung diaphragms provide the best solution to the shear key cracking and load sharing problems. The use of this combination scheme was shown to give torsional rigidity to the bridge cross section, arrest all differential movement between adjacent girders and hence improve load sharing between the girders. Hence, this study provides sufficient evidence that the transverse prestressing – underslung diaphragm combination is the most appropriate rehabilitation scheme for this type of bridges.

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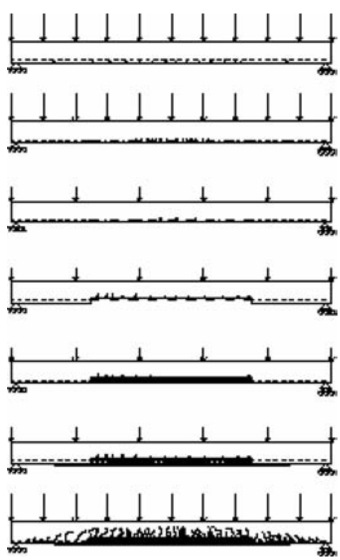
Degradation of structural performance – experiment introduction and expected results

M. Bergström
Luleå University of Technology, Sweden

B. Täljsten
Luleå University of Technology, Sweden, Technical University of Denmark, Denmark

ABSTRACT: Much effort has been put on investigating degradation of concrete structures, repair and upgrading separately, as can be read in numerous publications, i.e., Green et. al. (2003), Morgan (1995) and Täljsten (2004). However, an overall view has not been taken where the whole life cycle of a concrete structure is considered. In particular, no laboratory tests have been presented in the literature to the author's knowledge.

A structure passes several stages during its life. Normally two major stages are discerned, the service limit state (SLS) and the ultimate limit state (ULS). Concrete structures are designed for both these stages. In the SLS normally the deformation and crack widths are controlled. Deformation due to comfort demands and crack widths due to durability demands. In the ULS the structure is designed for its ultimate capacity – which for civil and building structures almost never is reached. From a safety aspect the ULS is most important; however, for the client the SLS with regard to maintenance, repair and upgrading are most costly. If the SLS was better understood, in particular from a rehabilitation point of view, more robust and cost effective repair and upgrading system could be developed.



a. Undamaged beam

b. Corrosion

c. Removal of variable load (dead load remains)

d. Removal of damaged concrete and repair of reinforcement

e. Refilling with new concrete

f. Application of CFRP plates

g. Loading until failure

Figure 1. Seven stages create the studied life cycle.

This paper is also a part of “Sustainable bridges”. “Sustainable bridges” is a European project which focus is to preserve bridges throughout Europe and create unanimous codes for all participating countries.

The project presented in this paper, Degradation of Structural Performance (DOSP), will investigate the behaviour of concrete beams which will endure a simulated life cycle procedure. The test program will direct the beams from full strength of the intact beam through degradation, repair and upgrading with FRP plate bonding to its original strength again or near. The cross-sectional strain distribution will be monitored during the test using Fibre Bragg Grating (FBG) Strain Sensors as well as traditional strain gauges. This gives the possibility of comparing results in between the two monitoring techniques over proportionately long time span. An accelerated corrosion procedure is used to corrode the flexural tensile reinforcement.

The cycle may be divided into seven stages, a to g, presented shortly in Figure 1, Horrigmoe (1998) and Sand 2001. This life cycle is possible in the real case scenario for bridges or other concrete structures which are subjected to chlorides, i.e. de-icing salt or sea water.

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Assessment, monitoring and control of bridge vibrations

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Evaluation of dynamic properties of the Infant Dom Henrique Bridge

F. Magalhães, A. Cunha & E. Caetano
Faculty of Engineering of the University of Porto, R. Dr Roberto Frias, Porto, Portugal

A. Adão da Fonseca, R. Bastos
AFA - Consultores de Engenharia SA, Porto, Portugal

ABSTRACT: The River Douro is crossed by several outstanding bridges linking the cities of Porto and Gaia. The 19th Century metallic Maria Pia and Luiz I Bridges and 20th Century concrete Arrábida and S. João Bridges got, at the beginning of the 21st Century the company of the Infant Dom Henrique Bridge. The evaluation of the most relevant dynamic properties of the bridge, after completion, was performed by the Laboratory of Vibrations and Monitoring of FEUP, using both experimental and numerical tools. This study was also used to validate a finite element model of the bridge which will be the baseline model for future damage detection studies. In this context, the present paper describes the main features of the developed work and presents the obtained results, emphasizing the excellent correlation achieved between measured and calculated dynamic properties.

1 DESCRIPTION OF THE BRIDGE

The Infant Dom Henrique Bridge is composed of two mutually interacting fundamental elements: a very rigid prestressed reinforced concrete box beam supported by a very flexible reinforced concrete arch, as shown in the elevation in Figure 1. The arch spans 280 m between springs and rises 25 m until the crown, thus exhibiting a shallowness ratio greater than 11/1.

2 AMBIENT VIBRATION TEST

The ambient vibration test was developed without disturbing the normal use of the bridge, with accelerations induced by traffic and wind. To measure the very low amplitude accelerations 4 tri-axial 18-bit strong motion recorders were used.

During the ambient vibration test, two recorders served as references located permanently at the reference cross-section of the deck, in both sides of the deck (upstream and downstream), while the other two recorders scanned the bridge deck by measuring the acceleration along the 3 orthogonal directions in both sides of 15 cross-sections.

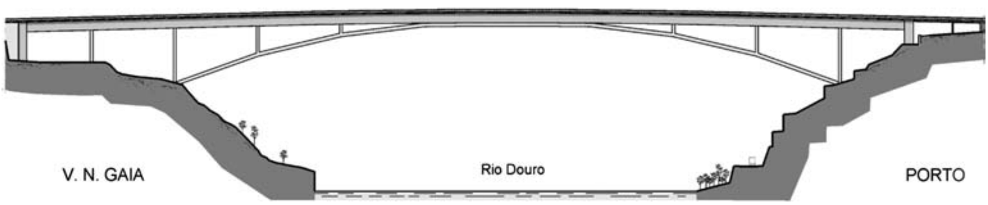

Figure 1. Elevation of the bridge.

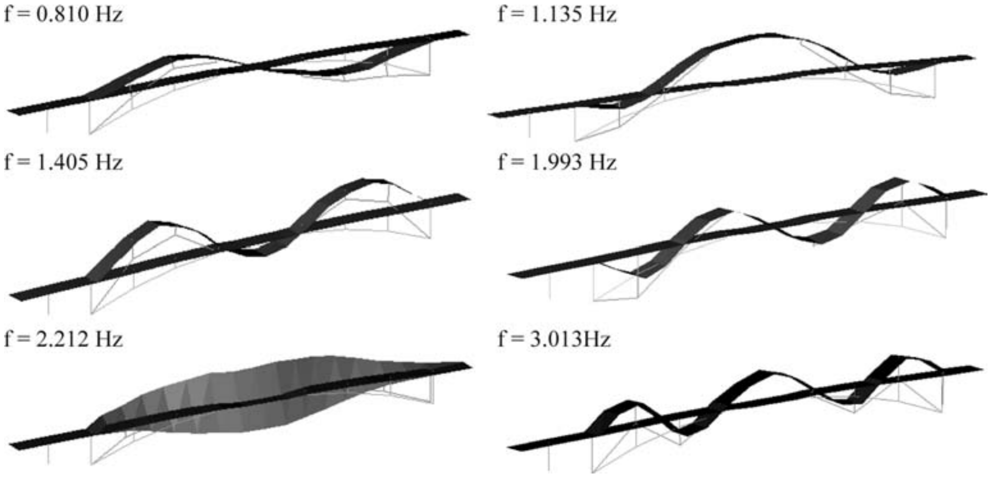

Figure 2. Perspective of some of identified vertical bending and torsional modes.

3 IDENTIFICATION OF THE MODAL PARAMETERS

The identification of the modal parameters from the data collected in the ambient vibration test was achieved with two separate output-only identification methods: the "Enhanced Frequency Domain Decomposition" and the "Data driven Stochastic Subspace Identification".

Some of the mode shapes identified by the first method are represented in Figure 2. The comparison of the modal parameters estimated by the two applied methodologies shows that the identified natural frequencies are almost coincident, the differences observed in the modal damping coefficients are generally small and the correlation of the mode shapes is very good (MAC > 97%).

4 NUMERICAL MODELLING

The structural behaviour of the bridge was modelled in the ANSYS software using 3D bar finite elements. The modelling of connections of the deck with abutments and columns has an important influence on the modal parameters of the bridge and for low levels of displacements the behaviour of these links was not easy to predict. Then, three alternative numerical models were developed, with different characteristics of the connections. It was concluded that, to achieve a realistic representation of the bridge dynamic behaviour, it was necessary to include horizontal springs in the abutments to take into account the existence of friction forces in the sliding bearings.

The correlation between values of the modal parameters in the final numerical model and the measured ones is very good, with relative errors of the natural frequencies lower than 5% and the MAC values always greater than 95%.

5 CONCLUSIONS

This paper shows that the development of an ambient vibration test together with the application of powerful output-only modal identification techniques allows the very accurate estimate of the most relevant dynamic structural properties of bridge structures. These are essential to achieve an updated numerical model that can enable a realistic characterization of the dynamic behaviour of the bridge. The updated model can be used to detect possible changes in the modal parameters due to any structural damage suffered by the bridge during its service life.

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Enhanced exploitation of bridge vibration measurements by Operational Modal Analysis

B. Peeters & H. van der Auweraer
LMS International, Leuven, Belgium

ABSTRACT: This paper will look into recent modal parameter estimation developments that allow maximal and automated vibration data exploitation. The PolyMAX method for Operational Modal Analysis will be introduced and applied to data from an extensive dynamic study of the Guadiana cable-stayed bridge as well as to continuous monitoring data of a football stadium.

1 INTRODUCTION

The experimental identification of the dynamic properties of civil engineering structures is of great interest in many occasions: design verification, updating of analytical (Finite Element) models, identification of damping properties, indirect determination of cable forces of cable-stayed bridges from measured eigenfrequencies, condition monitoring of structures, assessment of structural health after an earthquake, and verification of effectiveness of rehabilitation activities (Cunha et al. 2003).

In civil engineering practice, experimental dynamic analysis is often limited to applying a Fourier transform to acceleration data and identifying the structural resonances as peaks of the Fourier spectra. This paper will look into recent modal parameter estimation developments that allow maximal and automated vibration data exploitation. A big step forward in the analysis of civil engineering vibration data was the use of parametric ("curve-fitting") techniques as opposed to earlier peak-picking concepts. These curve-fitting techniques allow modeling the data by models with increasing order and the use of so-called stabilization diagrams. These diagrams led to a more objective identification of the structural modes and the separation between the true structural modes and numerical modes that are fitting the noise. However, sometimes, and especially in noisy cases, the interpretation of stabilization diagrams is not straightforward. Recently, a new frequency-domain parameter estimation method was proposed: the so-called PolyMAX method. The main advantage of PolyMAX is that it yields extremely clear easily interpretable stabilization diagrams. This does not only facilitate the task of the analyst, but also opens the door to automation. Obtaining the structural dynamic characteristics in a fully automatic way is in fact a fundamental requirement of a permanent monitoring system.

The paper will briefly discuss the PolyMAX method for Operational Modal Analysis and illustrate its benefits by using data from an extensive dynamic study on the Guadiana Bridge and a continuous monitoring system installed in a football stadium.

2 OPERATIONAL MODAL ANALYSIS OF THE GUADIANA BRIDGE

The technique of Operational Modal Analysis is illustrated using ambient vibration measurements from the International Guadiana Bridge (Fig. 1); see also Magelhães et al. (2005). A typical auto-correlation sequence (computed from a bridge deck acceleration signal) is shown in Figure 2. The

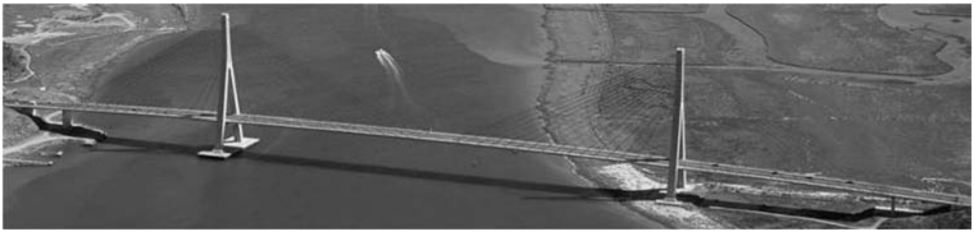

Figure 1. International Guadiana Bridge connecting Portugal and Spain.

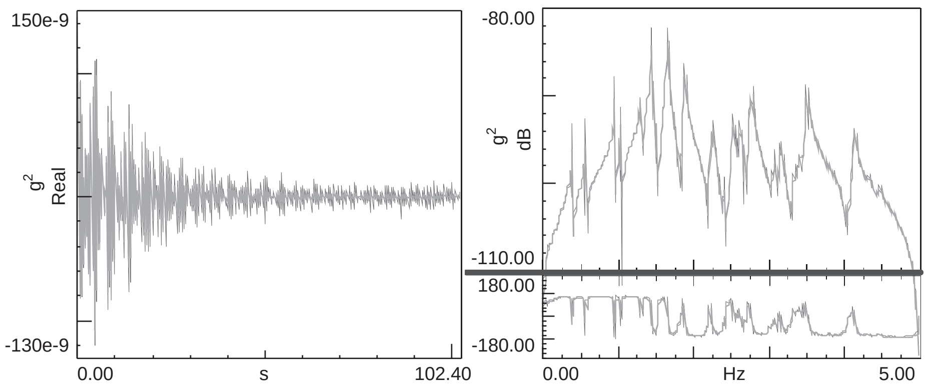

Figure 2. (Left) Typical output correlation; before and after applying an exponential window. (Right) Corresponding half spectra.

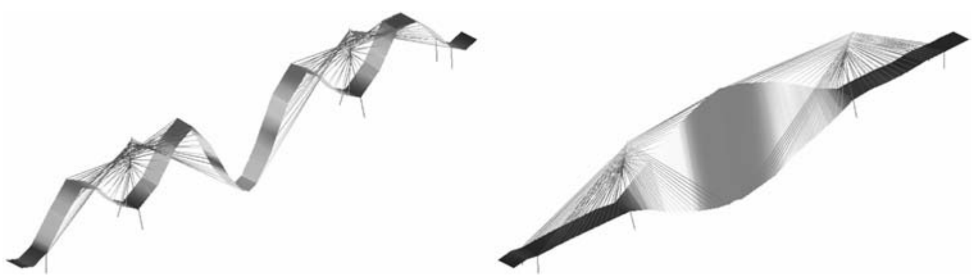

Figure 3. Guadiana Bridge mode shapes extracted from Multi-run ambient vibration data.

figure also shows the Fourier transform of this correlation sequence, the so-called *half-spectrum*. Some typical mode shapes extracted from the vibration measurements are represented in Figure 3.

The full paper also describes a fully automated modal parameter extraction procedure that has been applied to continuous monitoring data from a football stadium.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Comparative study of system identification techniques applied to New Carquinez Bridge

X. He, B. Moaveni, J.P. Conte & A. Elgamal
Department of Structural Engineering, University of California at San Diego, USA

ABSTRACT: The New Carquinez Bridge (NCB), a newly built long-span suspension bridge, is located 32 km northeast of San Francisco on interstate Highway I-80. With a main span of 728 m and side spans of 147 m and 181 m, the NCB is the first major suspension bridge built in the United States since the 1960s. The design and construction of the NCB incorporates several innovative structural features that have never been used previously for a suspension bridge in the USA, namely (1) orthotropic (aerodynamic) steel deck; (2) reinforced concrete towers; and (3) large-diameter drilled shaft foundations. The NCB is also the first suspension bridge in the world with concrete towers in a high seismic zone.

Dynamic field tests were conducted on the NCB, in November 2003, just before the bridge opening to traffic. They included ambient vibration tests (mainly wind-induced) and forced vibration tests based on controlled traffic loads and vehicle-induced impact loads. These tests provided a unique opportunity to determine the dynamic properties of the bridge in its as-built (baseline) condition with no previous traffic loading and seismic excitation. In this study, ambient and free vibration responses of the bridge were used to identify modal parameters of the NCB. The ambient vibration test was conducted after midnight (local time) and thus the bridge ambient vibration response was mainly caused by wind. The impact tests were conducted using one or two heavy trucks (about 400 kN each) driving over triangular shaped steel ramps (60 cm long and 10 cm high) designed and constructed specifically for these tests.

Different state-of-the-art time domain system identification algorithms were applied to the measured response (output-only) of the NCB. The system identification methods used include: (1) ERA based on free vibration data, (2) Multiple-reference Natural Excitation Technique combined with ERA (MNExT-ERA) based on ambient vibration data, (3) Random Decrement Technique combined with ERA (RDT-ERA) based on ambient vibration data, and (4) Data-driven Stochastic Subspace Identification (SSI-DATA) based on ambient vibration data.

The identified natural frequencies and damping ratios of the first 12 significant vibration modes are reported in Table 1. The MAC values between each mode shape identified using MNExT-ERA and the corresponding mode shape identified using another method are reported in Table 2. Figure 1 shows the complex valued mode shapes of the NCB (main span only) identified using MNExT-ERA in the complex plane (also called polar plots). It is observed that the identified natural frequencies and mode shapes using these various system identification methods and different test data are in excellent agreement. Considering that the estimation uncertainty of damping ratios is inherently larger than that of natural frequencies, the damping ratios identified using different methods are also found to be in good agreement, except for those identified using ERA. Thus, this study provides a reliable set of identified modal parameters of the NCB, which can be used to calibrate finite element models of this bridge or used as a baseline in future health monitoring studies of this bridge.

This study also investigates the performance of the various system identification methods considered. The system identification methods used in this study performed very well in identifying modal parameters (especially natural frequencies and mode shapes) of a long-span suspension bridge based on its response measurements (output-only). Of the three system identification methods applied to ambient vibration data, the computational efficiency of RDT-ERA is significantly lower than that of the other two methods. Damping ratios identified using MNExT-ERA, RDT-ERA

Table 1. Modal parameters identified using different system identification methods.

	Natural frequencies [Hz]				Damping ratio [%]			
Mode	ERA	MNExT-ERA	RDT-ERA	SSI-DATA	ERA	MNExT-ERA	RDT-ERA	SSI-DATA
1	0.191	0.193	0.193	0.193	2.20%	0.25%	0.21%	0.19%
2	0.203	0.204	0.202	0.200	−1.15%	1.98%	1.25%	2.18%
3	0.258	0.258	0.258	0.258	0.43%	0.21%	0.16%	0.24%
4	0.350	0.350	0.350	0.350	0.15%	0.15%	0.16%	0.17%
5	0.409	0.415	0.414	0.414	−0.03%	0.23%	0.26%	0.16%
6	0.452	0.470	0.471	0.471	−1.30%	1.17%	1.33%	0.16%
7	0.483	0.484	0.484	0.483	0.07%	0.19%	0.23%	0.23%
8	0.561	0.561	0.561	0.560	0.28%	0.16%	0.22%	0.23%
9	0.643	0.645	0.645	0.645	0.32%	0.09%	0.16%	0.11%
10	0.739	0.738	0.737	0.740	−0.08%	0.18%	0.25%	0.33%
11	0.795	0.799	0.799	0.799	0.10%	0.16%	0.24%	0.24%
12	0.958	0.958	0.957	0.957	0.31%	0.29%	0.29%	0.19%

Table 2. MAC values between mode shapes identified using MNExT-ERA and other methods.

Mode	ERA	RDT-ERA	SSI-DATA	Mode	ERA	RDT-ERA	SSI-DATA
1	0.980	0.998	0.998	7	0.959	0.996	0.990
2	0.621	0.917	0.964	8	0.903	0.997	0.991
3	0.982	1.000	1.000	9	0.992	1.000	1.000
4	0.992	1.000	1.000	10	0.971	0.990	0.984
5	0.985	1.000	1.000	11	0.895	0.998	0.999
6	0.952	0.989	0.976	12	0.831	0.975	0.982

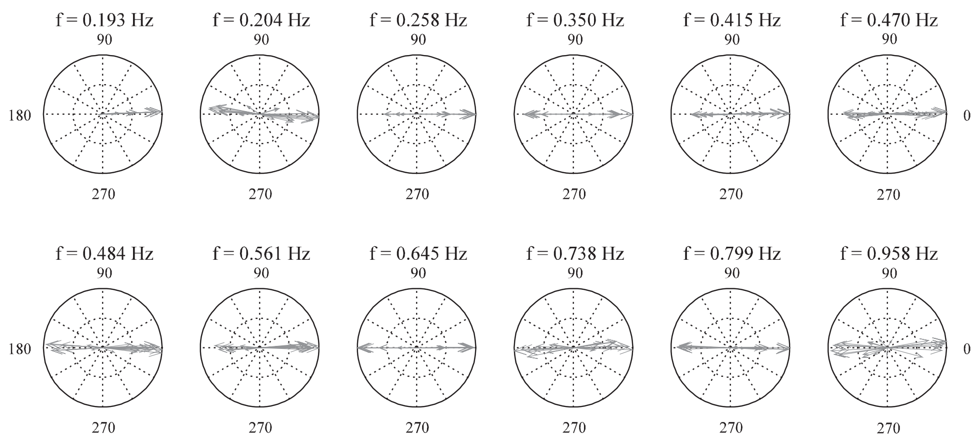

Figure 1. Polar plot of identified vibration mode shapes from MNExT-ERA using ambient vibration data.

and SSI-DATA are more consistent with each other than with those obtained using ERA. Errors in identifying the damping ratios using ERA are probably caused by the fact that the load applied to the bridge during the vehicle-induced impact tests departed from an ideal impulse load due to the continuous motion of the truck on the bridge before and after the impact.

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Analysis and control of vibrations of Guarda footbridge

E. Caetano, A. Cunha & C. Moutinho
Faculty of Engineering of the University of Porto (FEUP), Porto, Portugal

T.P. Mendonça
BETAR Consultores Lda, Lisboa, Portugal

ABSTRACT: This paper describes the methodology followed in the development of a detailed numerical study of the dynamic behavior of the new pedestrian arch bridge of Guarda, in Portugal, in order to predict the levels of vibrations induced by pedestrians and make a preliminary design and efficiency analysis of tuned mass dampers to control the lateral and vertical structural response.

1 INTRODUCTION

A new pedestrian arch bridge (Fig. 1) with a main span of 90 m has been recently proposed for the city of Guarda, in Portugal, designed by Tiago Mendonça (Mendonça 2005), from BETAR Consultores.

A preliminary dynamic study developed by BETAR indicated that the bridge was prone to vibrations induced by pedestrians. Therefore a detailed numerical study was developed by the Laboratory of Vibrations and Monitoring (VIBEST, www.fe.up.pt/vibest) of the Faculty of Engineering of the University of Porto (FEUP), with the purpose of better characterizing the dynamic behavior of the bridge under pedestrian loading and eventually defining appropriate control measures.

The existence of several natural frequencies in the range of pedestrians' excitation (lateral and vertical) and the identification of several sources of incertitude, such as those associated with the characterization of loads, with the specification of damping levels and with the definition of vibration limits considering the expected intensive use of the bridge, motivated the development of a numerical modeling, using different levels of simplification and considering the dynamic effects induced by a single pedestrian, groups of pedestrians or continuous streams of pedestrians.

The current paper describes the methodology followed to develop this study and the main results achieved.

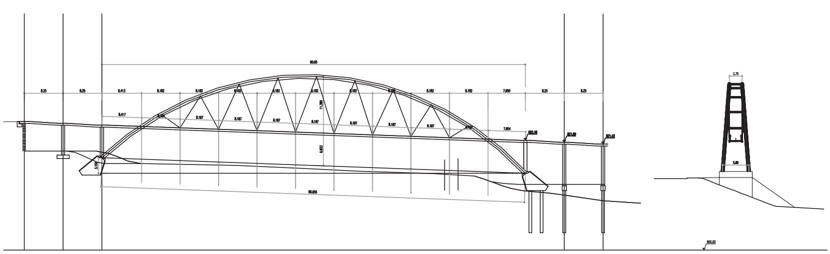

Figure 1. Side views of Guarda footbridge.

2 CONCLUSIONS

It is shown that the bridge displays very flexible behavior along the lateral direction, being also particularly susceptible to vertical vibrations induced by small groups of pedestrians running with a Gaussian distribution of frequencies centered at two particular vibration modes.

Therefore, several TMDs were preliminary designed and the corresponding control efficiency was numerically predicted in order to demonstrate the feasibility and usefulness of the vibration control solution proposed.

However, the results obtained depend on several important uncertainties, especially related with the damping characteristics of the structure, which means that only after construction, will it be possible to take a definitive decision concerning the final design and real implementation of control measures.

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Human-induced vibrations on footbridges

J.M.W. Brownjohn & A. Pavic
Department of Civil and Structural Engineering, University of Sheffield, UK

ABSTRACT: Dynamic assessment has become increasingly important for designers of footbridges, given the publicity of recent vibration serviceability 'failures'. What used to be an apparently simple problem has become very complex, involving not only structural dynamics but also biomechanics and psychology to try to prevent dangerous situations of large vibration response and possible panic. Quite apart from the usual difficulties of providing reliable mathematical (finite element) models, there are still not quantified uncertainties in the levels and nature of vertical and horizontal loadings due to pedestrian walking as well as feedbacks and other interactions due to multiple pedestrians or even crowds.

This paper reviews some of the issues involved and suggests some key areas for future research and development and some winder considerations for design of footbridges to reduce the risk of unserviceable behaviour under human dynamic loads.

1 INTRODUCTION

Vibration serviceability is becoming the dominant design criterion for footbridges as a consequence of a trend towards lighter, more slender, and more aesthetic structures.

Human-induced dynamic loading causing vibrations occurs frequently and is often the dominant load for footbridges. This should not be surprising as this load stems from the very purpose of a footbridge – to convey pedestrians. However, the message about the crucial importance of human-induced vibrations on footbridges seems not to be getting through to the owners, consultants, contractors and regulatory bodies responsible for the design and management of footbridges worldwide. As a consequence, many new landmark designs of footbridges suffer from vibration serviceability problems. Also, some old footbridges and even road bridges may suffer from the same problem if change in their normal usage occurs. The purpose of this paper is to introduce the field of footbridge vibration serviceability under human-induced dynamic loading and outline key emerging issues for future research and codification.

2 RATIONALISATION OF VIBRATION SERVICEABILITY PROBLEM

Every vibration serviceability problem, including the one pertinent to footbridges, can be rationalised into the following three questions:

1. what is the vibration source and how to describe it analytically?
2. what are the characteristics of the vibration path (i.e. footbridge structural mass, stiffness and damping properties)?
3. what norms or criteria are used to assess calculated/measured structural vibration levels?

3 HUMANS AS VIBRATION SOURCE FOR FOOTBRIDGE

During walking, a pedestrian produces a dynamic time varying force which has components in all three directions: vertical, horizontal-lateral (sway) and horizontal-longitudinal. This single pedestrian walking force, which is due to accelerating and decelerating of the mass of the body, has been studied for many years. In particular, the vertical component of the force has been most investigated as it used to be regarded as the most important and strongest of the three forces. Other types of human-induced forces important for footbridges are due to running and some forms of deliberate vandal loading. Some of these types of human-induced forces have been studied not only for a single person, but also for small groups of people. However, loads due to large groups (crowds) of pedestrians have seldom been formally investigated.

4 FOOTBRIDGE STRUCTURE AS VIBRATION PATH

Common practice in footbridge design requires an estimation of structural modal properties (natural frequencies, mode shapes, damping ratios and modal masses) to calculate their vibration response to different types of dynamic excitation. Modal properties are also required when designing remedial measures for existing lively footbridges by means of viscous and/or tuned mass dampers (TMDs). It is necessary that all these properties are estimated as accurately as possible. However, at the design phase there still does not exists the means for reliable estimation of modal properties of relevant modes of vibration. FE analysis alone does not produce the accuracy required for designing TMDs, for example. Therefore, reliable estimation is usually done on already built structures. This approach is crucial when rectifying existing lively structures.

5 FOOTBRIDGE VIBRATION RESPONSE AND ITS PERCEPTION

The main receivers of vibrations on pedestrian bridges, who govern their vibration serviceability, are walking people. Although Walley reported that “a pedestrian at rest on the bridge might ‘feel’ the passage of other pedestrians and be disturbed”, Leonard claimed that it was economically unjustifiable to design footbridges where standing people would feel no vibrations. To avoid disappointment and unwarranted expectations, this fact needs to be clearly explained to the footbridge users and owners.

6 KEY EMERGING ISSUES

There is considerable uncertainty when dealing with any of the three key aspects of footbridge vibration serviceability: vibration source, footbridge structure and perception when performing vibration response assessment. Clearly, there is scope for introducing probability- and riskbased principles into footbridge vibration serviceability design.

Recent problems with two high profile landmark footbridges: Solferino in Paris and Millennium Bridge in London and the authors’ experience in research and consultancy in this area, suggest a number of other important issues which are emerging rapidly e.g.:

1. bouncing on perceptibly vibrating structure in conjunction with synchronisation of small groups and vandal loading;
2. verification and further development of a mathematical model for lateral synchronous excitation including criteria for acceptable horizontal vibrations;
3. human-structure dynamic interaction in the vertical direction;
4. acceptance criterion for vertical vibrations for moving pedestrians in the frequency region around 2 Hz; and
5. management of footbridge design process considering risks associated with vibration performance and design of remedial measures.

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Clarification of the effect of high-speed train induced vibration on a railway steel box girder bridge by monitoring using Laser Doppler Vibrometer

T. Miyashita, H. Ishii & Y. Fujino
Dept of Civil Engineering, University of Tokyo, Japan

T. Shoji & M. Seki
Central Japan Railway Company, Nagoya, Japan

ABSTRACT: The effect of train-induced vibrations and the train speed on local stresses at each part of a steel box girder bridge was studied by field measurement. The field measurement was conducted using conventional sensors for train induced vibration and laser Doppler vibrometers for ambient vibration. The data acquired were analyzed to identify the natural frequencies and the changes based on the retrofit in the modal shapes of the railway bridge. The results show that vibrations at the lower flange of the main girder increased the local stresses causing damage. Next, the study shows that the vibration depends on the cyclic external load induced by train speed. Finally, train induced vibration phenomena was explained using simple analytical model combined with measurement and influence of train speed increase on local vibration was predicted.

1 INTRODUCTION

At a railway steel box girder bridge, damage was observed on the web of a main girder at the bottom end of a welded vertical stiffener. The cause of the damage is thought to be an increase in the local stress due to vibrations induced by high-speed trains. Thus, parts of the bridge of similar detail were also retrofitted and this proved to be effective in preventing further damage.

However, the relation between train-induced vibration and the increase in local stresses remains to be unclear. Similarly, the relation between train speed and the induced vibrations need to be investigated, including the effects of acceleration on a railway bridge since there are plans of increasing the present speed of trains in service.

2 THE OUTLINE OF FIELD MEASUREMENT

2.1 *The bridge for measurement*

The bridge studied for field measurement is a pair of steel mono-box girders with 4 spans. In bridges which have similar detail, damage was observed on the web of a main girder at the bottom end of a welded vertical stiffener. The parts of the bridge of similar detail were retrofitted using a T-shape member installed between the web and the lower flange using high tension bolts. The trains which pass on the bridge have 16 cars which are 25 m long, and having maximum speed of 270 km/h at present.

2.2 *Measurement using conventional sensors*

Accelerometers and strain gauges were installed in the bridge for measurement of train induced vibration. Since it is suspected that the cause of high local stresses was local vibration of the web or lower flange of the main girder, accelerometers were installed on both the web and the lower flange of the main girder. Strain gauges were installed on the main girder and the main girder web 20 mm away from the toe of welding between the web and the vertical stiffener.

3 MONITORING USING LASER DOPPLER VIBROMETER

3.1 *Laser Doppler Vibrometer (LDV)*

Laser Doppler Vibrometer (LDV) is an optical instrument employing laser technology to measure velocity of points on a vibrating object. The characteristics of the LDV are: first, compared with conventional transducers such as accelerometers, non-contact and long distance measurement (up to 100 m) is possible without adding mass or stiffness to an object. Second, resolution of velocity is very high (0.1 μm/s) and bandwidth is very wide (0–35 kHz). Therefore, in situations when the installation of the measurement devices is difficult, it is possible to measure high frequency components of small vibration such as ambient vibration. Third, by attaching a scanning mirror unit in front of the laser sensor head, measurement on multiple points (±20 degrees) is made possible.

3.2 *Measurement system*

The measurement system consists of three scanning type LDVs (named SLDV) and one single point type LDV (named RLDV). The RLDV always measures a reference point and is used to calculate the phase between measurement points for the identification of modal shapes. There are 3 measurement sections which include the center of the girder and each SLDV is positioned below each section. The number of the measurement points by the SLDV is 9 at each section and vibration measurement is conducted at each point by scanning the points.

Using LDV, measurements of ambient vibration was conducted. During ambient vibration measurement, measurements were conducted before and after retrofitting the bottom part of the vertical stiffeners. The objective of the measurement is to identify natural frequencies and modal shapes at the sections before and after retrofitting.

4 ANALYTICAL STUDY

An analytical study for train-induced vibration phenomena is conducted using a simple analytical model. The analytical model consists of a superimposed spring-mass model using identified natural frequencies based on the measurement at the focus section. Input forces for the model are applied by the impulse force based on the interval of train wheels. Output response is equal to the superposition of unit impulse response functions. Using this analytical model, train induced vibration phenomena is explained and influence of train speed increase on local vibration is predicted.

5 SUMMARY AND CONCLUSIONS

- At a main girder, the cyclic external loads of the bogies generate forced vibration and the frequency is proportional to the train velocity.
- Frequencies of local vibration at a web or lower flange of the main girder are integer multiples of the frequency of the external cyclic load. It was shown that the amplitude of the local vibration increases when the frequency of the local vibration is an integer multiple of the external cyclic load.
- Vibration at the lower flange has a localized effect on the stresses at the bottom of the vertical stiffener.
- A measurement system consisting of three scanning type LDVs and one single point type LDV has been developed. It was proven that the system can identify modal shapes of three sections of a steel box girder bridge.

– The change of modal shapes of the section is identified based on the ambient vibration measurement using LDV before and after retrofitting of the vertical stiffeners.
– Simple analytical model can explain the train-induced vibration phenomena qualitatively and predict the influence of train speed increase on local vibration. Responses become large at train speeds where the natural frequencies are integer multiples of the frequency of the main girder induced by cyclic loading.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Integrated monitoring of bridges by response measurements

S. Deix
Arsenal research, Vienna, Austria

R. Geier
Schimetta Consult ZT GmbH., Vienna, Austria

ABSTRACT: The conservation of the reliability (load bearing capacity, serviceability, durability) of structures is from major concern in civil engineering to prevent, or at least minimize, endangering of human life or high economic costs as a result of damage. Considering these aspects and taking into account, that nowadays a shift from construction of new towards rehabilitation of existing structures is recognizable, it becomes obvious that reliable structural health monitoring technologies are required in future. Main target is to increase service life of bridges and to ensure structural safety at the same time. During the last decade in particular the analysis of the vibration response in combination with advanced finite element simulations was improved. The derived modal parameters are representing the vibration response and are used to analyse the structural development over time.

Extensive testing and artificial damage tests on an existing railway structure in Austria during the last years have increased the doubts concerning identification and early detection of minor damage by vibration measurements, if simple identification techniques are employed. In general a dense sensor grid is required to derive a large number of frequencies and higher order modes. In addition, the analysis of damping coefficients as a further assessment parameter has not led to reliable results yet.

Due to these limitations, the key for successful monitoring will be integration of different methods to obtain a maximum of information concerning structural condition. As each specific monitoring technique offers advantages and at the same time has some disadvantages, it is important to combine existing techniques in such way, that the advantages supplement each other. In terms of bridge monitoring the combination of dynamic and static measurements, completed by a visual inspection is the most reliable approach.

The paper should present the concept of a monitoring system, which is implemented and installed in the Viennese "Reichsbrücke". The system is providing continuous data and a real time assessment of the bridge condition.

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Cable-deck dynamic interactions at the International Guadiana Bridge

V. Gattulli & M. Lepidi
University of L'Aquila, Monteluco di Roio

E. Caetano & A. Cunha
Italy University of Porto, Porto Codex, Portugal

ABSTRACT: The International Guadiana Bridge (Fig. 1) is a concrete cable-stayed bridge, formed by a central span of 324 m, two lateral spans of 135 m and two transition spans of 36 m, spanning the Guadiana River close to the border with Spain, at the southern part of Portugal. The bridge was designed by Câncio Martins (1992) and opened to traffic in 1991. Given the relatively severe wind and high seismic risk characteristics of the site, extensive studies were developed prior, during and after construction (Branco, 1987; LNEC, 1992; Branco et al., 1991, 1993). Despite the generally good performance under normal traffic and ambient conditions, the stay cables soon proved to be vulnerable to wind excitations, and that results in the occurrence of frequent oscillations of high amplitude, accompanied by a significant "rattling noise". Furthermore, a study developed at commissioning stage by Pinto da Costa et al. (1994) pointed to some vulnerability of certain cables to the so-called phenomenon of parametric excitation.

After that a series of test campaigns under normal traffic use and with different wind conditions has been conducted (Caetano & Cunha 2003, Caetano et al. 2005).

Recently, different mechanisms which cause the raising of high-amplitude cable vibrations in cable-stayed systems has been investigated by means of simple cable-supported beams (Fujino et al. 1993, Gattulli & Lepidi 2003, Gattulli et al. 2005). Their occurrence can be actually explained either due to direct cable excitation, in primary 1:1 internal resonance, or to different mechanisms of nonlinear cable-deck coupling, in 1:2 and 2:1 frequency ratio between global (deck) and local (cable) modes (Gattulli & Lepidi 2003). The measured data collected during the field observation of cables vibrations in the International Guadiana Bridge reveal a high dense frequency spectrum, with regular recurrence of internal resonance conditions (Caetano & Cunha 2003, Caetano et al. 2005). This data are used to update different finite element models of the bridge, which include a proper description of cable transverse motion (Caetano et al. 2000, Caetano & Cunha 2003, Caetano et al. 2005). Thus, from the results of a modal analysis, couples of interacting global and local modes are accurately selected to verify, and compare each other, the relevance of the different excitation sources.

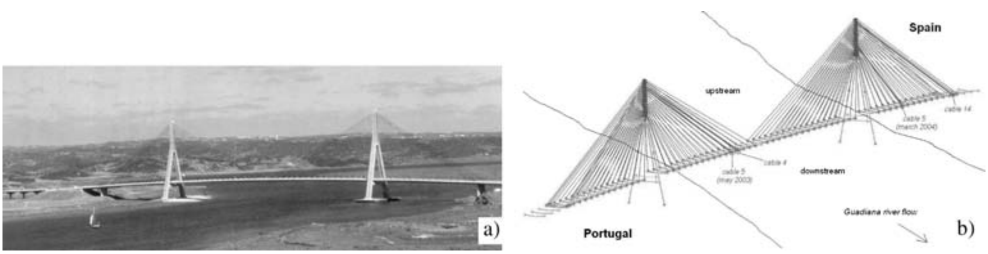

Figure 1. Guadiana Bridge: a) view (Câncio Martins, 1992) b) positions of highly vibrating cables (red).

To this aim, the mechanical properties of a parametric cable-stayed beam model (Gattulli et al. 1999) are selected in order to ensure an effective synthetic description of the complex bridge dynamics. Extended analysis on the forced response of the model allow to recognize the importance of the different structural parameters which are involved in the cable excitation mechanisms.

In conclusion, the study aims to verify the importance of previously identified nonlinear phenomena in cable-deck interaction in the measured response of the Guadiana Bridge. The investigation has been conducted starting from the amplitude levels of oscillations, at specified modal frequencies, measured on the bridge in the absence of direct measures of the complex aerodynamic and traffic excitation. On the light of the equivalent model solution and of the rough simplification of the external action, the measured low deck oscillation amplitudes seem to be still inconsistent with an effective justification of the observed high amplitude cable midspan displacement, neither assuming a simple linear behaviour, nor exploiting the effects of nonlinearities as sources of different known interaction mechanisms related to the occurrence of internal resonance conditions. Since the excitation of an extremely localized mode, with very low deck participation, seems to be the most promising hypothesis to match the experimental data, more studies should be made either in better modeling the complex external excitations or in verifying if any still unexplored mechanism (aerodynamic instability, other ratio beam to cable resonances,…) could really justify the observed dynamic behavior in the Guadiana cable-stayed bridge.

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Cost-effectiveness of bridge seismic retrofit using lead-rubber bearings

E.H. Wang
Ming Hsin University of Science and Technology, Taiwan

S.H. Lai
De Lin Institute of Technology, Taiwan

ABSTRACT: Highway bridges, including those constructed under modern seismic design codes, were severe damaged during the Chi-Chi earthquake in Taiwan. According to the Taiwanese Highway Bureau's preliminary report, at least nine bridges were severely damaged, including three bridges that were under construction. Five bridges collapsed due to fault rupture, and seven bridges were moderately damaged (Yen 2002). Bridge design specifications used in Taiwan have been revised three times since 1960. Prior to 1960, several design guide specifications were used for practical design. Some of them were based on Japanese design codes. The latest design specifications were based on the American Association of State Highway and Transportation Officials (AASHTO) bridge design specifications issued in 1995. The seismic design part of the code was not changed much from the 1992 AASHTO specifications.

Similar experiences have occurred in the U.S. Bridges have been vulnerable to earthquakes, sustaining damage to substructures and foundations and, in some cases, being completely destroyed. In 1964, nearly every bridge along the partially completed Cooper River Highway in Alaska was seriously damaged or destroyed. Seven years later, the San Fernando earthquake damaged more than 60 bridges on the Golden State Freeway in California. This earthquake cost the state approximately $100 million in bridge repairs. In 1989, the Loma Prieta earthquake in California damaged more than 80 bridges and caused more than 40 deaths in bridge-related collapses alone. The cost of the earthquake to transportation was $l.8 billion, of which the damage to state-owned bridges was about $300 million.

So much has been learned from these failures. The Engineering community has taken two approaches to improve the seismic resistance of highway bridges. The first approach requires considerable time, but is economically reasonable. Design guidelines are upgraded as more knowledge is gained about the response of specialized transportation structures to seismic activity. These new design guidelines can be applied to new construction as older bridges that are either structurally unsound or functionally obsolete are removed from service.

The second approach involves identifying those existing bridges that are important to the network and are susceptible to significant damage or collapse in the event of an earthquake. These structures can then be strengthened or retrofitted to enhance their response to seismic activity. Seismic retrofitting is a relatively new concept in bridge engineering and was motivated by the damage sustained by highway bridges during the 1971 San Fernando earthquake. The earthquake clearly pointed out the existence of a number of deficiencies in the then-current bridge design specifications. It also focused on the fact that numerous existing bridges may be expected to fail in some major way during their remaining life if subjected to strong seismic loads. However, because of the difficulty and cost involved in strengthening an existing bridge to new design standards, it is usually not economically justifiable to do so. This second approach thus requires significant capital expenditure; it consequently can prove economically infeasible in many cases.

A balance between these two approaches is needed to strengthen the highway system against seismic attack. This balance can be accomplished by upgrading those structures that form vital

links in the network and are vulnerable to damage, while at the same time imposing new applicable, geographically appropriate, seismic design standards on replacement bridges and new construction.

For the later approach, there are two common strategies that a bridge engineer adopts when encountered with a seismic retrofit project. One is based on conventional strengthening techniques and seeks to increase the capacity of the existing structure to meet the likely demand. The other is based on reducing the demand on the structure such that its existing capacity becomes sufficient to withstand the given earthquake. Generally speaking, retrofitting with a protective system such as seismic isolation has shown to be a cost-effective alternative to conventional strengthening. Base isolation systems include passive and active devices. Active systems are attractive only for longer span structures because the higher cost of active control and its required continual maintenance can be justified more easily for larger structures. Passive systems include mechanical devices, which simply dissipate energy and thus reduce response. Seismic base isolation is a relatively new method in earthquake-resistant design. The main concept is to uncouple the structure from the damaging action of an earthquake by using special devices placed under the structure.

There has been several studies in the past investigating the effectiveness of isolation devices for the seismic design of bridges. However, in many occasions, it is the cost that dominates the retrofit selection. Currently, there exists no universally accepted, comprehensive methodology to assist owners and engineers at the project-level to achieve this goal. To make the idea a reality only the cost of the retrofit and the bridge performance under the seismic excitation are compared to justify the cost-effectiveness. To seek the greatest benefit at the least cost a quasi-operational indicator is defined. An Economic Index (EI) is proposed to identify the most cost-effective solution when decision-makers face options like these. This article investigates the cost efficiency of the bridge retrofit using lead rubber bearings and the result is compared with a more traditional seismic retrofit strategy using neoprene bearings.

The Seismic Retrofitting Manual for Highway Bridges published by the Federal Highway Administration in May 1995 is referred to as the main reference in the study. The process of retrofitting bridges involves an assessment of a multitude of variables and requires the use of considerable judgment. The uniform load method as introduced in AASHTO DIVISION IA is illustrated in the study. It is essentially an equivalent static method of analysis which uses a uniform lateral load to approximate the effect of seismic loads. It is suitable for regular bridges that respond principally in their fundamental mode of vibration. Experiences have shown that all displacements and most member forces are calculated with good accuracy. To account for the cost-effectiveness of a lead rubber bearing, the uniform load method is used for ease of understanding the inter-relation between the variables. A numerical example is also provided herein to demonstrate the practicality of the study. The seismic load and displacement of the existing bridge is also calculated using the same method. The bridge performance after the retrofit using lead rubber bearings is also calculated similarly for comparison.

The seismic load combination used in this paper is stated in AASHTO Div.-IA Sec. 3.9, to account for the directional uncertainty of earthquake motions a combination of orthogonal seismic forces and moments resulting from analyses. Seismic forces and moments on each of the principal axes of a member shall be obtained by adding 100 percent of the absolute value of the member elastic seismic forces and moments resulting from the analysis in one of the perpendicular direction to 30 percent of the absolute value of the corresponding member elastic seismic forces and moments resulting from the analysis in the second perpendicular direction.

The retrofit alternative selection is obvious as the cost of each alternative is taken into consideration. The cost efficiency index, *EI*, consists of two major parameters: (1) seismic load reduction as a result of the rehabilitation, (2) retrofit costs. In this study, the cost-effectiveness of bridge seismically retrofitted using lead-rubber bearings is then calculated. First, calculate the *EI* value of the first alternative which uses the neoprene bearings. Then calculate the *EI* value of the second alternative which is the lead rubber bearings retrofit. The retrofit using lead rubber bearings is more cost effective if the *EI* is greater than that of the first alternative. Implementing the proposed approach a decision maker is able to rank the cost effectiveness of multiple retrofit alternatives. This paper offers a clearly defined, easy to implement process to standardize the cost efficiency of two seismic retrofit alternatives. Since the cost of each retrofit is different and the outcome of each retrofit is also different a priority ranking of retrofit alternatives is achievable using the Economic Index.

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A wireless sensor network for force monitoring of cable stays

G. Feltrin, J. Meyer & R. Bischoff
Swiss Federal Laboratories for Materials Testing and Research (Empa), Duebendorf, Switzerland

O. Saukh
University of Stuttgart, Stuttgart, Germany

1 INTRODUCTION

The objective of this paper is to investigate the performance of a wireless sensor network for force monitoring of cable stays. The network consists of a base station, a data logging and configuration unit that represents the data sink, and several remote measurement nodes attached to the cables that are the data sources. Each node is equipped with an acceleration sensor, a digital microprocessor and a radio transceiver. Since the nodes are powered by batteries, the network operates as a multi-hop communication network for reducing power consumption. Furthermore, data compression at the node level is an essential topic of wireless sensor networks. Network nodes operating for several months or even years from batteries have to perform a dramatic data compression in order to communicate as few as possible. This increase the node lifetime since compressing data needs less energy than communicating data. Therefore, the efficient use of wireless sensor networks in structural monitoring requires a substantially different data handling approach.

2 METHOD

A potential application of wireless sensor networks is cable tension force monitoring of stay cable bridges using measured natural frequencies. The accelerations of a stay are acquired with a capacitive MEMS sensor. The measured acceleration time series are processed directly in the nodes. Only the estimated natural frequencies are transmitted to the base station. Since the memory of the microprocessor is small, the natural frequencies are extracted from the acquired samples using a simple, 2-parameter AR-model.

The cable force is estimated by minimizing the error sum of squares between measured and computed natural frequencies that are based on an analytical cable model. It considers axial tension force, bending stiffness and sag of the cable. The parameter identification procedure is subdivided into three steps. This reduces the free parameter in each step to just one with the effect of considerably simplifying the nonlinear numerical optimization procedure. Furthermore, the method differs also from existing ones by working with eigenvalue equations instead of approximation formulas.

3 RESULTS

The wireless sensor network has been implemented on the Tmote platform and was validated with laboratory tests performed on the model bridge of Empa (Figure 1). The tests reveal that the natural frequencies could be estimated with an accuracy of approximately 1%. Figure 2 displays the estimations of the natural frequency of the second antisymmetric mode of the longest cable. The most probable error source is the observed variation of sampling rate (jittering) that produces additional noise and reduces the effective resolution of the data acquisition.

Figure 1. a) Laboratory stay cable bridge. b) Network node mounted on a cable with transceiver (top) and accelerometer (bottom).

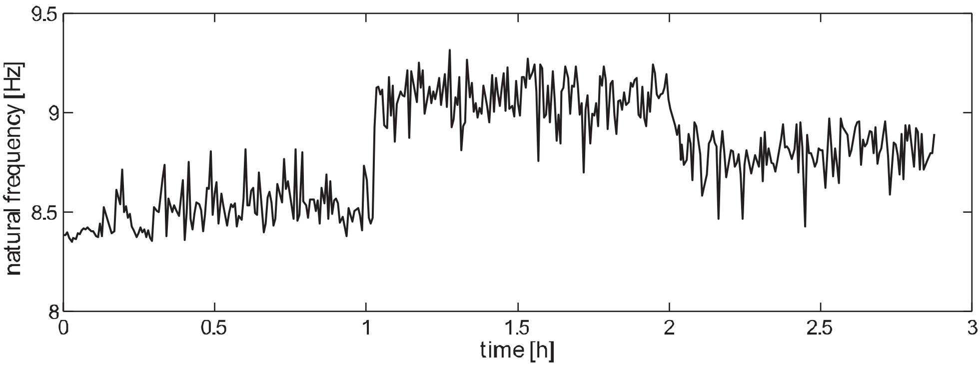

Figure 2. Time history of the estimated natural frequency of the 2nd antisymmetric mode.

However, much more critical is the choice of the cable model. The results show that the estimated axial tension force is very sensitive to changes of the boundary conditions. By changing the model from pinned to clamped ends, the tension force changes by approximately 10%. Furthermore, the model displaying the smallest error sum of squares between measured and computed natural frequencies (clamped edge) exhibits a tension force with approximately 9% error. Hence, a small error sum is therefore not equivalent with an accurate estimation. On real bridges, when independent measurements of cable tension forces are not available, the sensitivity to boundary conditions may produce significant systematic errors. In this case, the cable model with pinned ends provided a tension force estimation having only 0.9% error.

This investigation suggests that the method is suitable for estimating tension forces of stay cables with wireless sensor networks. An inadequate model choice represents the major error source. This error must be controlled by a calibration or by an improved cable model.

ACKNOWLEDGMENTS

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Dynamic testing of the Millau Viaduct

O. Flamand & G. Grillaud
CSTB, Nantes, France

ABSTRACT: The A76 Highway crossing through the mountain Massif Central is now the third direct link between Paris and the south of France. A lot of large viaducts have been built all along this motorway and the last one is the Millau Viaduct spanning a 2.5 km large valley 260 m upon the river.

A monitoring system was implemented on the viaduct during its construction by the SITES company. Once the viaduct achieved, using this instrumentation complemented by temporary additional sensors, the CSTB was commissioned by the EIFFAGE company, builder and concessionary of the Viaduct, to conduct the dynamic tests. These tests were carried out on November 24th and 25th 2004 in close collaboration with FEUP Porto, using both ambient excitation and pull and release tests in order to identify the vibration modes. Moreover, the measurement of the structural damping of the modes was a complementary goal of these ambient tests.

A series of measurements under ambient excitation (wind, works) was first performed, using two 3D autonomous Geosyg recorders successively set in 28 locations on the bridge, on the road pavement, when two other recorders were used as references and set in a fixed place. This gave a good definition of most of the modes identified on the viaduct. However, no information related to the damping of the various modes can be obtained by this technique. Finally the very small displacements used in the ambient excitation don't correspond to displacements as the structure exhibits under a major event like a strong storm or an earthquake.

For all these reasons a complementary test was performed under a more energetic, artificial, excitation, an impulse excitation obtained by cutting a cable taught with 100t between the ground and the deck.

Force balance accelerometers, which are more specially designed for seismic purpose, were used. 12 sensors from the monitoring framework and 9 additional ones were used during the pull and releases tests. Among these ones, two accelerometers C2, C4 were set on the same cross section of the span P6-P7, on each side of the deck, in order to catch by difference the torsion vibration properties.

The ambient excitation records allowed identifying the 10 first vertical modes, up to the 28th mode of the bridge. They gave results very close to the calculated ones.

The results from the pull and release tests were in perfect agreement with the ambient tests results in the lower frequency range but they also gave results for higher modes, up to the 36th one (4 additional vertical modes), more energy was transmitted to the structure.

The ambient excitation records allowed identifying 10 horizontal modes of the deck, up to the 20th mode of the bridge and the torsion mode.

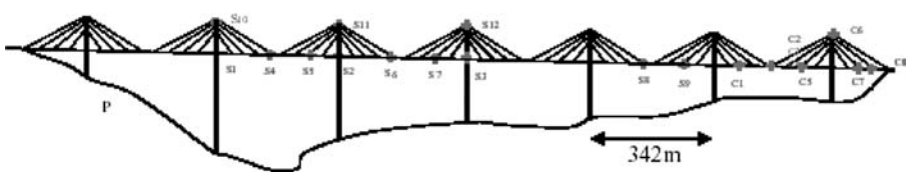

Figure 1. Location of the accelerometers along the bridge, the sensors belonging to the monitoring framework of the bridge are denoted "S", the additional sensors denoted "C".

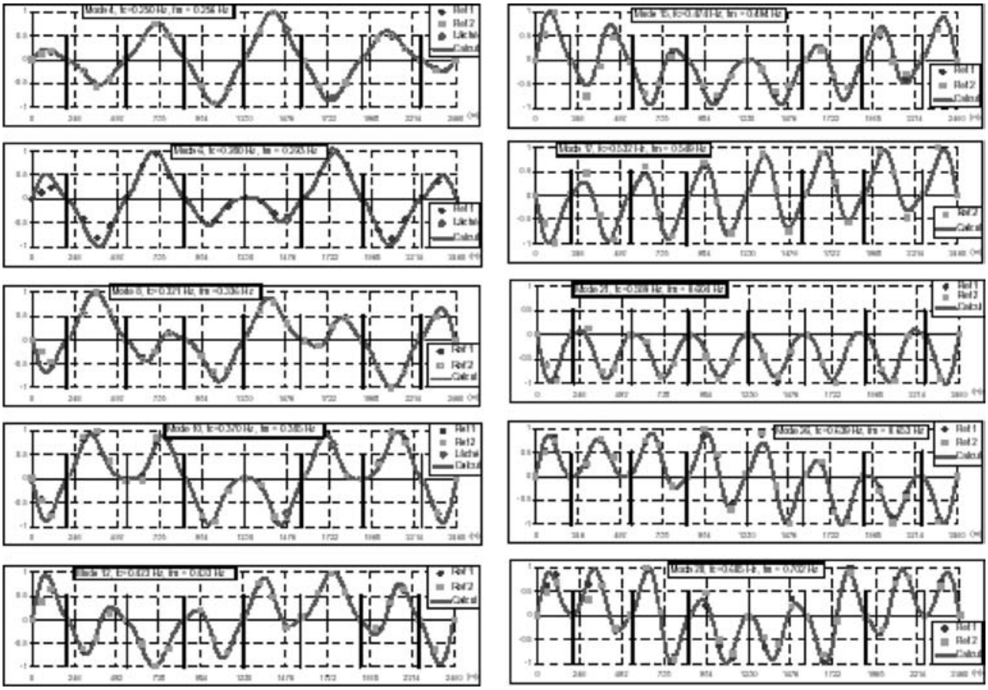

Figure 2. Comparison between the 10 first computed and measured vertical mode shapes.

Table 1. The damping level of the vertical modes.

Mode row	Frequency (Hz)	Average damping (%)
4	0.260	0.36
6	0.299	0.43
8	0.336	0.79
10	0.386	0.51
12	0.433	0.75
15	0.493	0.68
17	0.546	0.53
21	0.603	0.38
26	0.654	0.44
28	0.707	0.35
29	0.747	0.48
34	0.812	0.51
36	0.832	0.30

The damping level of each vertical mode, measured on the overall length of the recordings of the pull and release test, lies in the ranges 0.35% to 0.8% of critical. These values are in agreement with the expected ones.

The collaboration of the University of Porto in the performed dynamic tests, as well as the assistance and cooperation of MM. Vincent de Ville (GREISCH), Servant (EIFFAGE TP), Virlogeux (Consultant) and Brouillac (SITES) are gratefully acknowledged.

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Bridge displacement measurement system using image processing

J.W. Lee, J.S. Oh & M.K. Park
Dongyang University, Youngju, Korea

S.D. Kwon
Chonbuk University, Chonju, Korea

J.W. Kwark
Korea Institute of Construction Technology, Korea

ABSTRACT: The structures such as bridges, buildings and tall chimneys are subject to age as time goes and sometimes cause big disaster – such as breakdown or falling over. This kind of accidents can be estimated by measuring the displacement or vibrations of the structures. To solve this issue, bridge displacement/vibration measurement system using image processing with telescope lens to measure up to long distance (200 m) is developed and it can overcome the contact type system inefficiencies such as heavy, expensive property and temporary construction necessity. The system

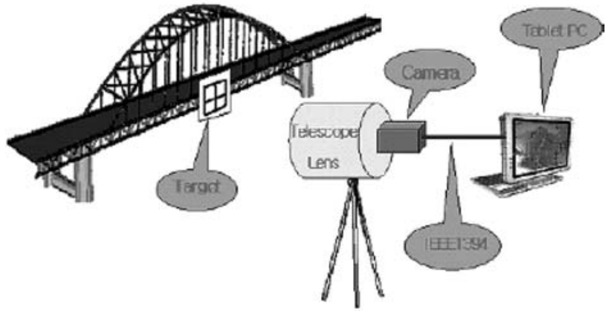

Figure 1. The system module and illustration how the system is used.

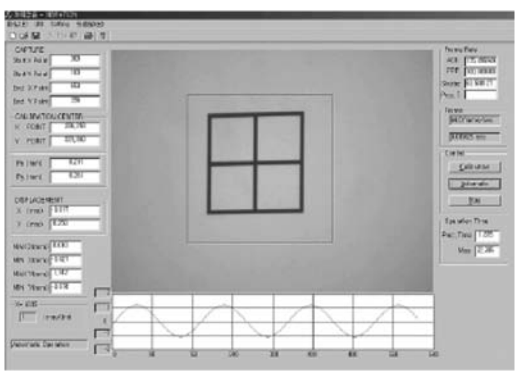

Figure 2. The GUI for the system setup, calibration and measurement.

Figure 3. The picture of the system usage.

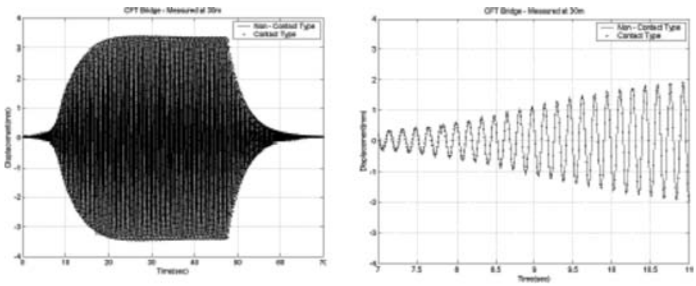

Figure 4. The vibration exciter experiment with 6.15Hz vibration for test bridge – measured at 30 m (the right graph is part of the left one – time from 7 to 11.

is composed of tablet PC with IEEE1394 digital camera, telescope lens and a target mark. The key idea is a self designed target mark with which the pixel to mm ratio can be calibrated by the program and image processing algorithm where all pixels are added up for each row (column) to find out the center point. The system can display the displacement/vibration history graph and max/min. The system is implemented and the experimental results are shown compared with a contact type system.

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Output-only modal identification of lively footbridges

F. Magalhães, A. Cunha & E. Caetano
Faculty of Engineering of the University of Porto (FEUP), Porto, Portugal

C. Butz
Institute for Steel and Lightweight Construction, RWTH Aachen University, Aachen, Germany

A. Goldack
Schlaich Bergermann & Partner, Stuttgart, Germany

ABSTRACT: This paper summarizes the dynamic tests recently performed in five German outstanding lively footbridges designed by Schlaich Bergermann & Partner, in the context of the European Project SYNPEX, and describes how the corresponding modal parameters (natural frequencies, mode shapes and modal damping ratios) were accurately identified on the basis of the application of modern and powerful stochastic modal identification techniques.

1 INTRODUCTION

During the last years, special attention has been devoted, by designers and local authorities, to the construction of modern footbridges in leisure and natural parks and urban areas, that often constitute attraction points or even landmarks, either from the structural engineering or the architectural point of view.

These bridges are submitted to relatively low static loads and frequently present considerable spans, and so they become easily lightweight structures, whose fundamental vertical natural frequencies can fall in the dominant frequency range of vertical pedestrian excitation (1.5–3.5 Hz). Moreover, a fundamental lateral natural frequency may also occur in the range 0.7–1 Hz, the corresponding mode of vibration being easily excited by the lateral human walking excitation, with an excitation frequency of one half of the stepping frequency. This is the case of the well known Solférino and Millennium Bridges, where excessive lateral vibrations were registered under the passage of a pedestrian stream, requiring the adoption and implementation of suitable control measures.

The proneness of this type of slender footbridges to suffer significant levels of vibrations led to the necessity to investigate the dynamic behavior of existing lively footbridges, as well as to develop simple but reliable design load models, that can take into account the dynamic effects induced by single pedestrians (walking, running or jumping), groups of pedestrians or continuous streams of pedestrians.

This research effort has been made by the authors in the context of the European Research Project SYNPEX, a significant number of dynamic measurement campaigns having been performed on existing lively footbridges in Germany, Portugal and France, with the purpose of: (i) accurately identify the most relevant modal parameters (natural frequencies, mode shapes and modal damping factors); (ii) validate finite element models for dynamic analysis; (iii) analyze the variation of damping with the level of oscillation; (iv) evaluate the levels of vibration induced by pedestrians and compare them with human comfort acceptability criteria; (v) develop enquiries characterizing the human reaction to the induced vibrations.

In this context, this paper summarizes the experimental work performed in five German outstanding lively footbridges designed by Schlaich Bergermann & Partner and describes how the corresponding modal parameters (natural frequencies, mode shapes and modal damping ratios) were accurately identified on the basis of the application of modern and powerful stochastic modal identification techniques.

2 DYNAMIC TESTING OF LIVELY BRIDGES

Figure 1. Eutinger Waagsteg in Pforzheim.

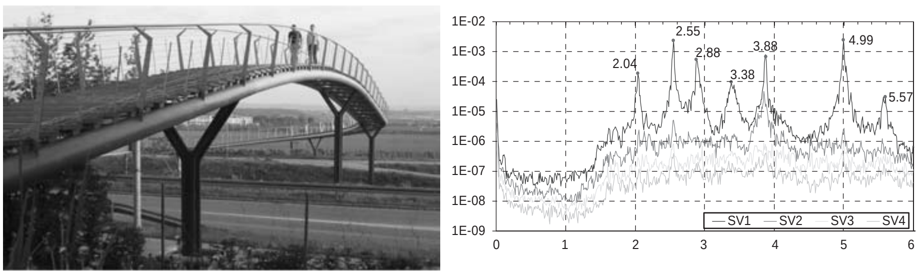

Figure 2. View of Ruit Bridge (left); Singular values spectra (right).

Figure 3. View of Kochenhofsteg Bridge in Stuttgart (left) and Glacisbruecke in Minden (right).

Figure 4. View of Katzbuckelbruecke in Duisburg.

3 CONCLUSIONS

The paper describes the extraction of the most relevant dynamic properties of five German lively footbridges, based on the application of stochastic modal identification techniques to an experimental database created during dynamic tests under pedestrian loading. The results obtained in terms of natural frequencies and damping values explain that lively behavior and allow the calibration of finite element modeling. However, though human comfort limit values mentioned in technical literature are exceeded in some situations, enquiries to pedestrians reveal that human induced vibrations are generally well accepted in such outstanding structures.

Seismic and dynamic analysis

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Comprehensive parametric study on the performance of seismic-isolated bridges

M. Dicleli
Middle East Technical University, Ankara, Turkey

S. Buddaram
Bradley University, Peoria, Illinois, USA

ABSTRACT: The concept of seismic isolation is essentially a method of controlling the seismic response of structures through yielding of the isolators possessing generally bilinear force deformation relationship. Recently, several studies have been conducted to identify important structural and ground motion parameters on the seismic response of simple structural systems with elasto-plastic and bilinear force-deformation relationships (Makris and Black 2004a; b). However, these research studies are broad and do not particularly address the practical issues concerned with the seismic performance of seismic-isolated bridges (SIBs). Several other researchers have attempted to study the performance of and identify the optimal design parameters for seismic-isolated buildings (Baratta and Corbi 2004, Fan et al. 1991). However, similar research pertaining to SIBs and the practical aspects of their design is scarce. Thus, further research is required to study the effect of isolator and substructure properties as well as the characteristics of the ground motion on the response of SIBs and identify critical issues with regard to the design of such bridges. Results from such a study may then be used by bridge engineers to arrive to a more sound and economical design of SIBs.

For this purpose a parametric study, involving more than 800 nonlinear time history (NLTH) analyses of simplified structural models representative of typical SIBs, are conducted. The effects of several parameters, such as bridge substructure stiffness, characteristic strength, post-elastic stiffness of the isolator and the properties of the ground motion on the performance of SIBs are considered. The effect of the isolator's elastic stiffness on the performance of seismic-isolated structures has been found to be negligible (Makris and Black 2004b).

In the presented study, the performance of SIBs is measured by the maximum isolator force and displacement (MIF and MID). The MIF represents the magnitude of the seismic force transferred to the substructures. Thus, it has a remarkable effect on the design of the substructures. The MID is generally used to determine the isolator size as well as the width and type of the expansion joints. In some cases, the widths of the substructures may be governed by the MID. Accordingly, for a given ground motion, smaller isolator force and displacement are indicative of superior seismic performance.

In the NLTH analyses, two sets of ground motions are used. The first set involves a suite of 15 earthquakes with peak ground acceleration to peak ground velocity (A_p/V_p) ratios ranging between 5.50 s^{-1} and 21.5 s^{-1} These ground motions are used for the examination of the performance of SIBs as a function of their frequency characteristics or A_p/V_p ratio. The second set consists of harmonic ground motions with various excitation periods, T_g where $A_p/V_p = 2\pi/T_g$. The main reasons for using such ground motions are (i) to validate the concept of A_p/V_p ratio for representing the frequency characteristic of the ground motion (ii) to have a clear understanding of the effect of the frequency characteristics of the ground motion on the performance of SIBs. Harmonic ground motions with A_p/V_p ratios having similar range as those of the seismic ground motions are considered to enable the comparison of responses from both.

The effect of the intensity of the seismic ground motion on the performance of SIBs is studied using a dimensionless term, A_pW/Q_d. This term represents the ratio of the seismic inertial force of a rigid bridge superstructure to the characteristic strength of the isolator.

The presented research study led to the following important conclusions and recommendations for the design of SIBs;

- The substructure stiffness may be ignored in the preliminary design of the isolators.
- The MID and MIF are highly dependent on the A_p/V_p ratio of the ground motion. This implies that the choice of the seismic ground motion according to the characteristics of the bridge site is crucial for a correct design of the SIB.
- The isolator design displacement capacity is generally governed by the minimum isolator displacement requirement of AASHTO (American Association of State Highway Transportation Officials, 1999) for ground motions with A_p/V_p ratios larger than 9. Thus, in the design of SIBs located in sites characterized by such ground motions, in addition to the isolator displacement capacity, the force transferred to the substructures must also be based on the same AASHTO provision.
- For bridge sites associated with ground motions with high A_p/V_p ratio, isolators with low characteristic strength should be used to minimize the forces transferred to the substructures while keeping the displacements within reasonable ranges. However, for bridge sites associated with ground motions with low A_p/V_p ratio, isolators with relatively higher characteristic strength may be used to minimize the MIDs while keeping the forces transferred to the substructures within reasonable ranges.
- For SIBs subjected to high intensity ground motions, using an isolator with a large characteristic strength may be more advantageous as it produces forces comparable to and displacements smaller than those of an isolator with relatively smaller characteristic strength. However, the designer needs to be aware that higher characteristic strength may adversely affect the design under service loading.
- In the selection of the isolator properties for design, the isolator post-elastic stiffness is an important factor that may affect the seismic response of the bridge. On the other hand, as proven by earlier research, the elastic stiffness of the isolator has only a negligible effect on the seismic response.
- For bridge sites characterized by ground motions with high A_p/V_p ratio, as the MIDs are very small, the smallest possible isolator post-elastic stiffness should be used to limit the forces transferred to the substructures. The same may be recommended for bridge sites characterized by ground motions with low A_p/V_p ratio. However, for such bridge sites, as the isolator displacements may become significantly large, the lower bound of the isolator's post-elastic stiffness may be determined based on a prescribed design displacement limit. Regardless of the A_p/V_p ratio of the ground motion, since isolators with very small or no lateral restoring force capability may experience very large displacements, the lower bound of the post elastic stiffness should be based on AASHTO's lateral restoring force requirements for keeping the MIDs within reasonable limits and for accommodating isolator installation imperfections.

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Proposed improvements to AASHTO effective damping equation for seismic-isolated bridges

M. Dicleli
Middle East Technical University, Ankara, Turkey

S. Buddaram
Bradley University, Peoria, Illinois, USA

ABSTRACT: An elastic analysis procedure for the design of seismic-isolated bridges (SIB) is presented in AASHTO (American Association of State Highway Transportation Officials) Guide Specifications for Seismic Isolation Design (AASHTO 1999). Since the behavior of seismic isolators is non-linear in nature, equivalent linear (EL) properties need to be defined for the elastic analysis of SIB. The EL properties of SIB are expressed in terms of an effective stiffness, an effective period and an effective damping (ED) ratio to account for the hysteretic energy dissipation of the isolators. Using these EL properties, an EL analysis procedure is followed to estimate the absolute maximum seismic responses in the isolators and in other components of the bridge.

The accuracy of the EL analysis results for SIB has been studied by several researchers (Hwang 1996; Franchin et al. 2001). Although these studies were very useful in identifying the imprecisions in the EL analyses results, they focused on specific design code procedures. Perceptibly, this type of an approach may not allow for a rational verification of the EL analysis results due to the approximations involved in the code procedures. Furthermore, most of these studies have not explicitly considered the effects of all the isolator, substructure and ground motion properties as well as the effective period of the bridge on the accuracy of the EL analysis results. Therefore, a comprehensive evaluation of the EL analysis results for SIB is required to rationally identify the deficiencies within the analysis procedure and to suggest improvements to AASHTO's ED equation for a more accurate prediction of the actual nonlinear responses.

The evaluation of the EL analysis results mainly involves the comparison of the seismic response quantities obtained from EL analyses (isolator displacement, d_E, obtained from EL analyses) with those obtained from nonlinear time history (NLTH) analyses (isolator displacement, d_{NL}, obtained from NLTH analyses). The NLTH analyses are conducted using harmonic and seismic ground motions with various frequency characteristics. The effect of several parameters such as bridge substructure stiffness, isolator and ground motion properties are considered in the evaluation of the EL analysis results. The effect of bridge superstructure mass on the ED ratio and hence on the accuracy of the EL analysis results is also investigated. This is followed by regression analyses of the acquired data to incorporate additional empirical relationships in AASHTO's ED equation for improving the accuracy of the EL analysis results. At the end, the accuracy of the EL analysis results using the improved ED equation is assessed and conclusions are outlined.

The force-displacement relationship of most isolators is idealized as bilinear for design purposes. The parameters defining this bilinear relationship is the elastic stiffness of the isolator, k_u, the post-elastic stiffness of the isolator, k_d, the yield force and displacement of the isolator F_y and d_y respectively and the characteristic strength, Q_d, of the isolator which is defined as the intercept of the post elastic stiffness slope with the vertical force axis. The isolators are generally designed to accommodate a maximum or design force (F_i) and displacement (d_i).

In the NLTH analyses, two sets of ground motions are used for the verification of the EL analysis procedure. The first set consists of harmonic ground motions with peak ground acceleration to peak ground velocity ratio, $A_p/V_p = 2\pi/T_g$, where T_g is the excitation period. The main reason for using

harmonic ground motions for the verification of the EL analysis procedure is to have a clear understanding of the effect of the frequency characteristics (or the A_p/V_p ratio) of the ground motion on the accuracy of the EL analysis results. Harmonic ground motions with A_p/V_p ratios ranging between $5.23\,s^{-1}$ and $20.0\,s^{-1}$ are used in the analyses. The second set of ground motions involves a suite of 15 earthquakes with A_p/V_p ratios ranging between $5.50\,s^{-1}$ and $21.5\,s^{-1}$. The details of the ground motions are presented in Table 1. These ground motions are used for further verification of the EL analysis procedure and for improving the AASHTO's ED equation for a more accurate prediction of the seismic response quantities.

The effect of the intensity of the seismic ground motion on the accuracy of the EL analyses results is studied using a dimensionless term, A_pW/Q_d (W: weight acting on the isolator), which represents the ratio of the seismic inertial force of a rigid bridge superstructure to the characteristic strength of the isolator.

The parametric study conducted for the evaluation of the EL analysis results yielded an improved ED equation for a more accurate prediction of the seismic responses of SIB. The summary of the findings from this research study is outlined below.

– It is observed that the effect of the substructure stiffness on the accuracy of the EL analysis results is negligible.
– The EL analysis generally produces unconservative estimates of the actual maximum displacement response of seismic isolators using AASHTO's damping equation.
– Analysis results have revealed that the difference between the EL analysis results and the actual nonlinear responses becomes larger for ground motions with high frequency content (high A_p/V_p ratio).
– It is also observed that for ground motions with larger intensity and lower A_p/V_p ratio, which produce larger effective periods; the EL analysis yields more reasonable estimates of the actual nonlinear responses.
– It is demonstrated that AASHTO's ED equation must involve the effective period of the bridge, dominant period (or frequency content) of the ground motion and k_d/k_u ratio of the isolator for a more accurate estimate of the actual nonlinear response of the SIB. Accordingly, an improved ED equation that incorporates the effective period and properties of the isolator is proposed.

The impacts and benefits of the proposed (improved) ED equation are as follows:

– The proposed damping equation yields more reasonable estimates of the actual ED compared to several other damping equations found in the literature.
– The proposed ED equation yields more reasonable estimates of the actual nonlinear responses regardless of the type of ground motion used in the analyses.
– Furthermore, the proposed ED equation reduces the dispersion of the d_E/d_{NL} data. This indicates that the proposed ED equation improves the reliability of the individual EL analysis results for various A_p/V_p and A_pW/Q_d ratios.
– Based on the above remarks, it may be stated that the proposed (improved) ED equation may produce more reliable SIB designs.

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Using opposing spirals to enhance seismic behavior of reinforced concrete bridge columns

W. Turechek & R.A. Hindi
Bradley University, Peoria, Illinois, USA

ABSTRACT: Through countless research projects, it was shown that spirals were the most effective means of lateral reinforcement for a column when large displacements need to be addressed. However, some limitations have been placed on lateral confinement by ACI 318-05 to aid constructability and provide certain minimum levels of ductility.

ACI 318-05 specifies a minimum spacing of 25 mm for spirals in all reinforced concrete columns. For the design of important structures that are located in areas of high seismic activity where maximum ductility is required, the 25 mm spacing may limit the ductility.

High-strength concrete has also been limited by this requirement. The acceptance of high-strength concrete has been slow because of its brittle manner of failure. This is especially true in seismic areas where high ductility is necessary. This level of ductility is not possible due to current codes and confinement techniques that limit the amount of lateral reinforcement needed to fully develop this potentially useful building material.

Also specified in the ACI 318-05 building code is a maximum spacing of 75 mm for spirals. This spacing, while not a problem for building columns, can cause difficulty in constructing large diameter columns, which are commonly used for bridges, where keeping spirals equally spaced is cumbersome. Piles would also benefit from increased spiral spacing by allowing more room for spiral slippage and movement without affecting the confining capability of the spiral. Finally, heavily reinforced beam-column connections would become less congested and more easily constructed if spiral spacing could be increased without affecting the load carrying capacity of the column.

It has been proposed (by the first author) that by using two opposing or cross spirals instead of a single spiral, the spacing can be manipulated to either increase the characteristics of the column or to enhance the constructability of the member. In areas where high ductility is necessary, two spirals can be used, each with a spacing of S. This would effectively double the volume of confining reinforcement, without adversely affecting the clearance for aggregate. For areas where constructability is a concern, a spacing of 2S could be used for each spiral. The net effect would be a column with two spirals each with half the standard volumetric ratio of confining reinforcement. In this situation, the load carrying characteristics are virtually unchanged, but it will be more likely that the reinforcement can be constructed within the given tolerances. This can become an issue for long and slender members where in-situ inspection is not possible.

Ductility of reinforced concrete columns is heavily influenced by the use of transverse or confining steel. Advancements in materials such as high-strength concrete and more stringent requirements for seismic areas have increased the need for high ductility in columns without compromising constructability. More efficient methods of confinement such as the proposed method can be used to meet the demand without creating unnecessary congestion in the section.

This paper investigates the cyclic behavior of reinforced concrete circular bridge columns confined using a new confinement technique. The new confinement technique uses two opposing spirals (cross spirals) to confine circular bridge columns in order to enhance their strength and ductility or to increase the spiral spacing (pitch) to facilitate the flow of concrete during construction. Twelve reduced scale reinforced concrete circular bridge columns with two different lengths and several spiral spacing and patterns were tested. The columns were subjected to constant axial load

(bridge self weight) and reversed cyclic lateral displacement (seismic load) to study the influence of the new confinement technique on the lateral strength and ductility of reinforced concrete circular bridge columns compared to columns confined with conventional single spiral. Six of the columns (1000 mm high and 200 mm diameter) were designed to study the flexural behavior, and the other six columns (500 mm high and 200 mm diameter) were designed to study the shear behavior. This paper reveals the effect of the new confinement technique on the cyclic behavior of reinforced concrete circular bridge columns.

A constant axial compression load of 100 kN was applied at the top of the column to simulate the self-weight of the bridge. This was based on 8% of the specified concrete strength times the gross concrete area ($0.08f'_cA_g$) of the column. The specified lateral cycling consisted of 3 cycles at a specified displacement. This loading was used to ensure that the recorded data could adequately be compared to previous tests. The loading history was based on the predicted flexural monotonic behavior of the columns, which was calculated using a layer analysis computer program. The yield displacement (Δy) was estimated as 2.25 mm and 9 mm for the short and long columns respectively. The loading history for both the short and long columns consisted of 3 cycles at each of the following multiples of the yield displacement, Δy: 0.75, 1, 1.5, 2, 3, 4, 5, 6, and 7. All of the short columns used a reduced loading scheme except for the first column tested, XS-70S. The loading for these columns skipped the cycles at 0.75, 1, and 2 yield displacements. At the conclusion of each cyclic loading test, the axial load was removed and the columns were either monotonically pushed to failure or to the limit of the actuator. If failure was not reached due to the limits of the actuator, they were then cycled to the extreme displacement until failure occurred.

Several conclusions were drawn from the experimental testing of these 12 reinforced concrete columns.

- Columns confined with regular spirals and cross spirals that have similar volumetric ratios of confining steel performed similarly during testing.
- Columns confined with the new technique of cross spirals did not have any failures due to spiral rupture. Spiral failure occurred in 3 of the 4 columns confined using regular spirals. Columns with cross spirals failed due to longitudinal bar fracture or bar buckling.
- Columns confined with cross spirals showed less shear deterioration compared to columns with regular spirals.
- Columns confined with cross spirals with twice the confining steel showed significant enhancement in ductility and strength.
- The new technique of confinement may be used to increase the ductile capacity or to increase constructability of columns and piles.
- By adding an additional (cross) spiral and keeping the same spiral spacing, ductility and strength can be increased. Alternately, by using two spirals with double the spacing, congestion in heavily reinforced sections can be avoided without lowering the ductility and strength capacities.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Effects of strong winds on bridge-vehicle interaction for long span bridges

M.S. Cheung, B. Noruziaan & C.Y. Yang
The Hong Kong University of Science and Technology, Hong Kong SAR, China

ABSTRACT: Stability of cable-stayed and suspension bridges under dynamic loads has been the subject of numerous studies. Safe passage of vehicles over these structures in presence of high winds and earthquakes is of utmost importance. The Smart and Sustainable Infrastructure Research Centre at the Hong Kong University of Science and Technology (HKUST) is currently studying various aspects of bridge-wind/earthquake-vehicle interactions. Among such studies are the stability analyses of vehicles on long and flexible cable bridges in presence of strong winds.

Hong Kong is located in a region with strong winds. High intensity typhoons are frequent in Hong Kong during typhoon season, i.e. July to October. In such cases the traffic management authorities impose special restrictions on vehicle movements on the bridges. A better understanding of bridge-wind-vehicle interaction will improve our understanding of the stability of vehicles.

In this preliminary study, the effects of moving loads as well as wind loads on dynamic response of a typical cable-stayed bridge will be studied. The overturning effects of wind on the stability of vehicles and the possibility of their slippage will be the focus of this research.

Computer software SAP2000 Nonlinear is used to carry out the analytical studies. "NLLINK" elements are used to model the connection of the tires with the bridge deck, as well as the suspension system under vehicles. The main purpose of this study is to establish the foundation for further investigations on the dynamic interactions between moving vehicles and long span bridges under the effect of strong winds.

The finite element model of a typical one span cable-stayed bridge is developed in this study (Fig. 1). In order to study the effect of wind on vehicles, box structures made of shell elements are used to model the buses (Fig. 2). The buses are connected to the deck using a serial combination of "Isolator2" and "Damper" NLLINK elements.

The bridge is first analyzed under the effect of static loads of gravity and pretension forces in the cables. The results are used as the starting point to carry out further analyses. In order to model the

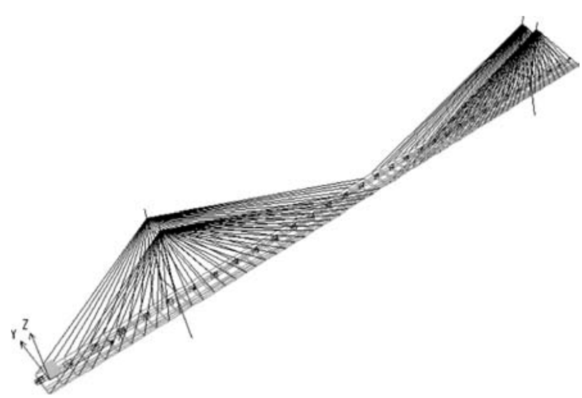

Figure 1. The finite element model.

effects of moving vehicles, constant vertical downward loads of 625 kN (corresponding to a heavy truck) are put on the deck at different locations along the traffic lane and at regular intervals. The time of appearance of such loads at various locations simulates the constant speed of the vehicles on the bridge. The wind load is applied horizontally on the windward side of the deck (push) as well as vertically upward (suction) on the surface of the deck. Dynamic response of bridge and stability of vehicle under different load combinations are investigated.

The results have shown that, due to the massive structure of the bridge, the moving vehicles are not significantly affecting the dynamic behavior of the bridge. The dynamic effect of the moving loads on the maximum deflection of the bridge is about 1.5%. Besides, under various vehicle speeds and in absence of wind, no separation between buses tires and bridge deck occurs, meaning that the bus will not lose contact with the deck surface (Fig. 3).

On the other hand, strong wind has significant effects on the stability of vehicles. Separation of the tires from the deck occurs frequently, resulting in extreme slippages. Since the gap elements can take no tensile force, their axial forces drop to zero as they open, simulating the separation between vehicle tire and bridge deck; which could occur frequently and sometimes for a rather long period of time (Fig. 4).

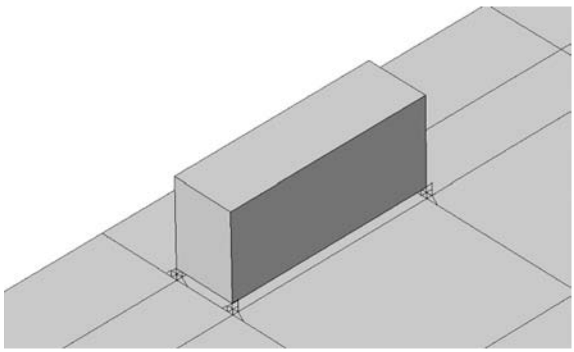

Figure 2. Shell boxes and NLLINK elements to model the buses.

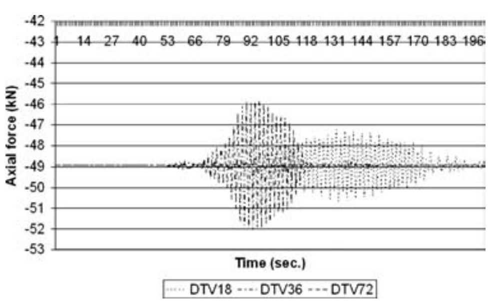

Figure 3. Axial forces in midspan bus suspension system, excluding wind effects.

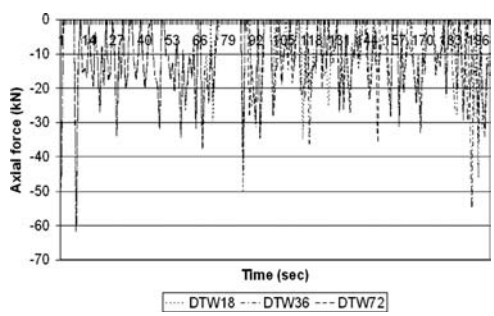

Figure 4. Axial forces in midspan bus suspension system, under strong wind effects.

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Seismic capacity assessment of cable supported bridge considering material nonlinearity

K.C. Lee, D.S. Moon & S.-P. Chang
Seoul National University, Seoul, Korea

S.H. Bae
Yongma Engineering Co, Ltd., Seoul, Korea

ABSTRACT: As the concept of seismic design is shifting from elastic design to limit state design, new analysis tools which can consider inelastic behaviors of the cable-stayed bridge under earthquake loads are necessary. In this study, the inelastic model of the Second Jindo Bridge, which can consider material nonlinearity as well as geometric nonlinearity, is investigated to define the ultimate behavior of the structure under earthquake loads and perform the nonlinear analysis. From the time history dynamic analysis with four different ground motions which are El Centro, Kobe, Taft, and Mexico earthquake, the analyses prove that responses of the bridge depend on not only peak acceleration but also other variables such as frequency characteristics and the duration time. Furthermore, the static push-over analysis of steel pylon shows that an unexpected local buckling, which can not be predicted in the inelastic beam analysis, could happen and lead to the sudden collapse of the cable-stayed bridge. Therefore, the new seismic capacity estimation method of the cable-stayed bridge that can take into account the local bucking effect of the steel pylon using the inelastic beam element is proposed. In addition, a seismic reinforcement method is suggested by filling the steel pylon with the concrete. From the experimental work this reinforcement method is concluded as an effective method.

1 INTRODUCTION

Preliminary researches have contributed in various fields to understand the behavior of long span cable-stayed bridges under seismic loading. Most studies focus on only the geometric nonlinearity and few researches considering material nonlinearity are conducted. In recent researches, nevertheless, material nonlinear resources are studied and it is identified that material ductile behavior performs higher seismic capacities at the structure under experiencing seismic loads. Therefore, it is necessary to evaluate seismic capacity of the cable supported bridge considering not only geometry nonlinearity but also material nonlinearity. If the ultimate behavior of cable-stayed bridges is fully understood, more valuable result can be derived.

2 ANALYTICAL RESEARCH

The nonlinear beam element model of the Second Jindo Bridge is investigated to perform the nonlinear dynamic analyses under the various earthquake ground motions. The isotropic hardening model is adapted as a steel elastic-plastic material model and the concrete damaged plasticity model is adapted to simulate the concrete inelastic behavior. In addition, a nonlinear spring is chosen to model the elastic shoe connection between pylon and girder.

Four ground motions are selected to perform the time history dynamic analysis: El Centro, Kobe, Taft, and Mexico earthquake. Each ground motion is multiplied by specified factors to investigate the damage sequence and level. The general failure mechanism of the elastic-plastic beam element

Table 1. Moment comparison between beam and shell element analysis.

	Damage moment	Ultimate moment
Beam element analysis	88,300	115,000
Shell element analysis	76,000	89,950

Table 2. Ductility index.

	SP0C	SP2C	SP5C
μ_m	1.62	2.08	2.38
μ_{95}	1.75	2.39	–

analysis model is failure of elastic shoes and concrete cracks at first, reinforcing bars yields at second, and damaging steel pylons at last. However, in the case of Mexico earthquake steel pylons yield before rebar yielding. It means that if we guarantee seismic safety from the design earthquake, it does not guarantee that the bridge never falls down from the earthquake whose PGA is smaller than the design earthquake. Therefore, we have to know the seismic capacity of bridge and how the bridge behaves when the strong earthquake strikes even if it is larger than design earthquake.

The damaged moment of steel pylons is about 88,290 KN·m and this can be considered as a seismic capacity of the steel pylons. Assuring the analysis accuracy, a detailed shell element model of the steel pylon is made to perform the static pushover analysis using RIKS method instead of beam element. The result proves that the elastic-plastic beam element analysis overestimates the seismic capacity of a steel pylon about 10% compared with detailed shell element model of pylon. It is same for ultimate moment as well as damage moment. Also we can find out that the bottom part of the steel pylon is a critical damage point of a steel pylon cable-stayed bridge.

3 EXPERIMENTAL RESEARCH

To define seismic capacity and verify the analytical result, an experimental work is carried out. Quasi-static test is performed to three specimens. One is steel pylon, and the others are concrete filled steel pylon at the bottom part. Two concrete filled specimens have different filling height. Concrete is filled up to twice and fifth heights of plastic hinge which points out the line that the local buckling failure happens. The result proves that the filled-in concrete improves ductility from 1.62 to 2.05 and 2.38.

4 CONCLUSIONS

Seismic design of cable stayed bridge in Korea has been performed according to ASD (Allowable Stress Design) method or WSD (Working Stress Design). This design method makes sure that the structure behaves elastically under the design load. However, this study shows that even if an earthquake whose PGA is smaller than design value, it can gives unexpected damage because of various characteristics of earthquake itself. Also, the design method shifts to limit state moreover PBD (Performance Based Design) which promises failure behavior for various level of loads. Therefore, we have to know the limit state of bridge and how it behaves after the limit state load or earthquake. This study gives an example for steel pylon cable stayed bridge.

This study concludes that the key point of this type of bridge for earthquake load is a bottom of steel pylon. For that reason, seismic capacity of the bridge is defined from the capacity of steel pylon. Although many bridge designers prefer to use beam elements to analyze structural behavior of bridge for simplicity, this research proves that the beam element over-estimates the seismic capacity about 10%. Therefore, it needs detailed shell element nonlinear analysis or experimental work to estimate an exact seismic capacity of steel pylon.

The experimental work of this research shows that the detailed shell element analysis agrees with experiment. Also, ductility of steel pylon is not enough to keep stable after a strong earthquake as 1.62. Therefore, the concrete filling is proved as a good reinforcement method to improve the ductility of pylons.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Seismic performance of hollow sectional columns with different portions of lap-spliced longitudinal bars

I.H. Kim, H.T. Yeo, C.H. Sun & J.S. Lee
University of Ulsan, Ulsan, Korea

1 INTRODUCTION

The amount of transverse reinforcement is specified proportional to the sectional area of RC piers. As a result excessive reinforcement is provided, which causes poor constructability. It is, therefore, one of urgent tasks to reduce the amount of lateral reinforcement. Making use of fewer amounts of lap spliced longitudinal bars is suggested as an alternative on the basis of the results in previous studies. To confirm the seismic performance of piers with such details the scale model tests consisting of 3 hollow rectangular sectional columns were carried.

2 TEST MODELS

Three specimens with hollow rectangular section were designed. One (Model C50S120) has continuous longitudinal bars with only 50% of lateral reinforcements specified in the code. Other two (Model L75S80 & L75S60) have alternate lap spliced longitudinal bars with 75% of lateral reinforcement but, these two models have different spacing of transverse reinforcement.

Considering test conditions, the models were designed with 1/4.2 scale. They are of hollow rectangular section with 1100 mm × 650 mm and 800 mm × 350 mm for the outer dimension and the inner one, respectively. The height from the base to the loading point is 2600 mm. The aspect ratio is, therefore, 4.0. The ratio of longitudinal bars is 1.3% in all models. For the models with lap splices (L75S80, L75S60), the lap length is designed in accordance with the ACI provision for tension lap splice, class B. The resulting lap length is 380 mm. Figure 1 shows the detail of scale model. Figure 2 shows the arrangement of transverse reinforcement including cross-ties.

3 TEST RESULTS

The reversed cyclic lateral load is applied with keeping 0.1 fckAg of vertical load. The envelope curves of the load-displacement hysteresis loops are compared in Figure 3. Very stable behaviour up to 118 mm corresponding to 4.5% of drift ratio is shown. The test results are summarized on Table 1. The displacement ductility ratios to maximum load carrying capacity are 6.36, 6.21 and 6.05 for model L75S80, L75S60 and C50S120, respectively. In seismic design the design force is determined by introducing R-factor. The required displacement ductility is not over 5 even if

Table 1. Test results.

Description	L75S80	L75S60	C50S120
Δ_y (Yield displacement)	19 (mm)	19 (mm)	19 (mm)
Δ_m (Displacement at P_{max})	122 (mm)	118 (mm)	115 (mm)
μ_m (at P_{max})	6.36	6.21	6.05
μ_m/H (Drift ratio)	4.7%	4.5%	4.4%

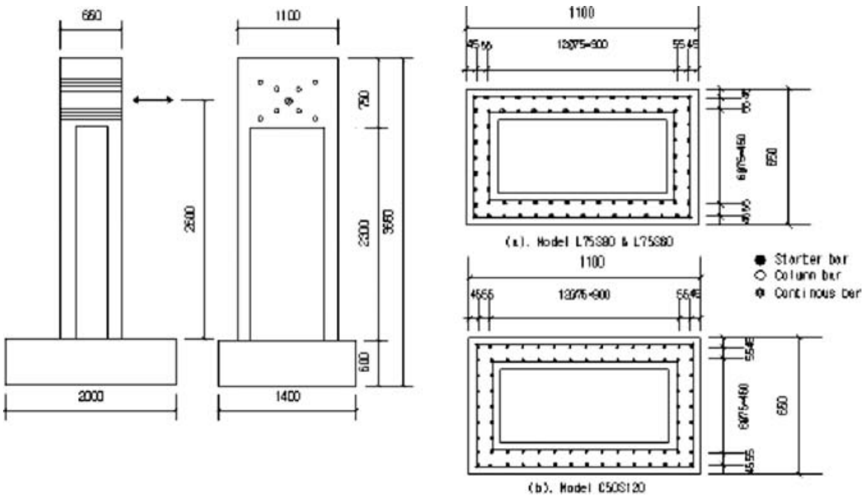

Figure 1. Details of scale model (unit; mm).

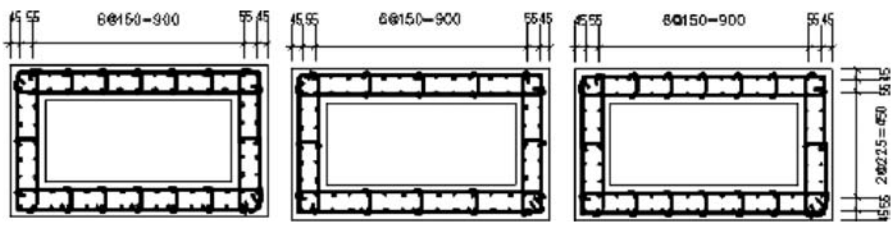

Figure 2. Details of transverse reinforcements (unit; mm).

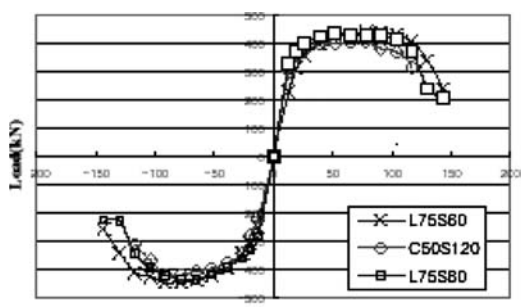

Figure 4. Comparison of envelop curves.

evaluated conservatively on the basis of an equal energy method when R = 3 is applied. In the view of this required ductility the models appear to have a sufficient performance.

4 CONCLUSIONS

In the tests all columns failed in flexure showing displacement ductility over 6, which appeared to satisfy the ductility demand in moderate seismicity zones. It is desirable in establishing the performance-based design to provide various seismic details meeting the performance demand so that designers can choose a rational detail considering construction condition. This study has been served basic experimental data to meet that purpose.

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Cost-effectiveness evaluation of MR damper system for cable-stayed bridges under earthquake excitation

D. Hahm, H.-M. Koh, S.-Y. Ok, W. Park & C. Chung
Seoul National University, Seoul, Korea

K.-S. Park
Dongguk University, Seoul, Korea

ABSTRACT: Over the last few decades, cable-stayed bridges have attracted great interest because of their aesthetics, structural efficiency, and economy of construction. However, these structures have a vulnerability to dynamic loads such as earthquakes and strong winds as a consequence of their structural flexibility and low damping characteristics. Among various strategies for the dynamic response mitigation of cable-stayed bridges, semi-active dampers have lately become a subject of interest due to its favorable features, i.e., inherent stability and minimal power requirements. Furthermore, the recent rapid progress of the related technology on magnetorheological (MR) damper system shows the great deal of promise for introduction of semi-active dampers to real structures. Thus, it would be a worthwhile subject to investigate the feasibility and cost-effectiveness of the MR damper system for cable-stayed bridges under earthquake excitation.

This paper presents cost-effectiveness evaluation of MR damper system for cable-stayed bridge with respect to the various seismic characteristics, such as magnitudes and frequency contents. The cable-stayed bridge used in this study is the Bill Emerson Memorial Bridge, which was adopted in the phase I benchmark control problem. The semi-active damper system is controlled by a modified clipped-optimal control algorithm on the basis of Linear Quadratic Gaussian (LQG) optimal controller. The cost-effectiveness of the MR damper system is defined by the ratio of life-cycle costs between a bridge structure with the semi-active control devices and a bridge structure with shock transmission units. The life-cycle cost function consists of the summation of initial construction cost and expected damage cost due to earthquake event. For the evaluation of the expected damage cost, the failure probability of a cable-stayed bridge system is estimated by use of the crossing theory and simulation methods.

The cost effectiveness index of MR damper (J_{MR}) is defined as the ratio of life-cycle costs between a bridge structure with MR dampers and a bridge structure with STU, as follows.

$$J_{\mathrm{MR}}(\alpha,\beta)=\frac{E[C_T]_{\mathrm{STU}}}{E[C_T]_{\mathrm{MR}}}=\frac{C_I+C_D P_{f,\mathrm{STU}}\,\dfrac{\nu}{\lambda}(1-e^{-\lambda t_{\mathrm{life}}})}{C_I+C_{\mathrm{MR}}+C_D P_{f,\mathrm{MR}}\,\dfrac{\nu}{\lambda}(1-e^{-\lambda t_{\mathrm{life}}})}$$
$$=\frac{1+\beta\cdot P_{f,\mathrm{STU}}\,\dfrac{\nu}{\lambda}(1-e^{-\lambda t_{\mathrm{life}}})}{1+\alpha+\beta\cdot P_{f,\mathrm{MR}}\,\dfrac{\nu}{\lambda}(1-e^{-\lambda t_{\mathrm{life}}})} \qquad (8)$$

where $\alpha(=C_{\mathrm{MR}}/C_I)$ is a ratio of installation cost of MR dampers(C_{MR}) to initial construction cost(C_I), and $\beta(=C_D/C_I)$ is a ratio of damage cost(C_D) to initial construction cost. $P_{f,\mathrm{STU}}$ is failure probability of a bridge with STU, $P_{f,\mathrm{MR}}$ is the failure probability of a bridge with MR dampers, ν is earthquake occurrence rate, λ is discount rate and t_{life} is lifetime of a structure. In this study ν, λ and t_{life} are assumed to be 0.1/year, 5% and 50 years, respectively. Note that the MR damper is cost-effective when the value of J_{MR} is larger than 1.

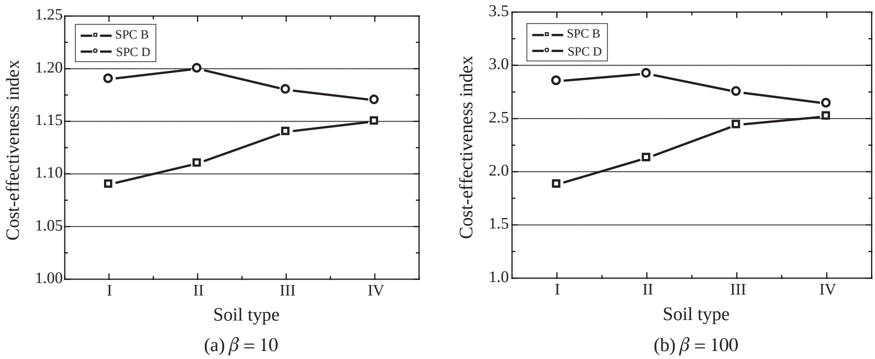

Figure 1. Cost-effectiveness of an MR damper system with respect to the soil profile types ($\alpha = 0.001$).

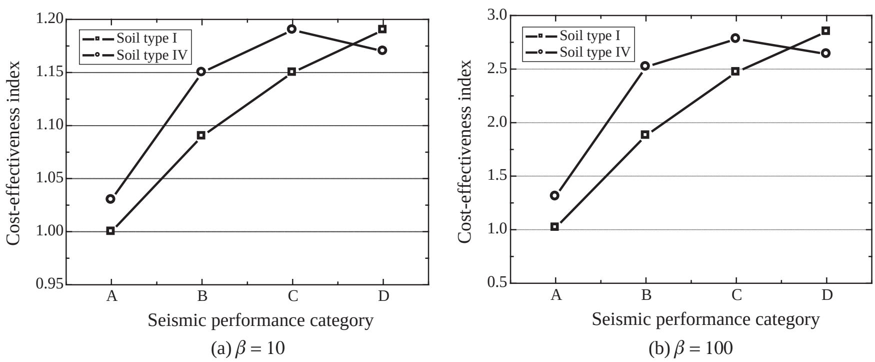

Figure 2. Cost-effectiveness of an MR damper system with respect to the earthquake intensities ($\alpha = 0.001$).

From the results of the cost-effectiveness evaluation for the MR damper system with respect to α, it was found that the cost-effectiveness index does not show significant variation, and the index is larger than 1.0. This implies that the MR damper system is cost-effective regardless of the damper cost. However, cost-effectiveness increases remarkably in accordance with an increase in β. Especially for the case of SPC D, the cost-effectiveness is very sensitive to variations in β. Thus, the damage cost of the structural system, i.e., the socio-economic effect due to the failure of a cable-stayed bridge, should be carefully assessed when designers apply MR damper systems to cable-stayed bridges that are located in regions of high seismicity.

Figure 1(a) and Figure 1(b) provide the cost-effectiveness of an MR damper system with respect to the soil profile types of the construction site when $\beta = 10$ and $\beta = 100$, respectively. Note that the scales of the cost-effectiveness index in each case are quite different although both cases show a similar tendency according to the soil profile types. In the case of ground motion with moderate seismicity, i.e., SPC B, the MR damper system shows a higher cost-effectiveness in soft soil profile types. On the other hand, in a region of high seismic intensity, SPC D, the cost-effectiveness is consistently higher compared to the case of SPC B for the overall soil profile types. Figure 2 presents evaluation results with respect to the intensities of earthquake ground motion. For SPC A and soil profile type I, the failure of a structural system even without control device is very non-probable, the benefit of the control system on failure probability becomes negligible, from a total life-cycle cost perspective. In soft soils, it should be noted that the MR damper system is more cost-effective for moderate seismicity, SPC C, rather than high seismicity, SPC D. The decrease in cost-effectiveness for a high-intensity earthquake may be caused by the limited capacity of the MR dampers, 1000 kN, used in this study. Therefore, a preliminary study to determine the appropriate level of MR damper capacity will be required when MR damper devices are applied to cable-stayed bridges, especially in the regions of high seismicity and soft soil profile types.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Performance evaluation tests of laminated rubber bearings for seismic isolation design of bridges

I.J. Kwahk, C.B. Cho, Y.J. Kim & J.W. Kwak
Korea Institute of Construction Technology, Goyang, Korea

ABSTRACT: Seismic isolation devices have been introduced and applied in Korea since more than 10 years ago. However, established design specifications and performance evaluation criteria for seismic isolation required to support such application are still lacking, and numerous examples where foreign criteria have been applied can be found in the construction fields. Such situation led researchers to persevere in providing economically efficient as well as rational seismic isolation design in order to secure the reliability of a seismic isolation design suitable to the domestic construction environment. Among these, a first step has been realized through the addition of a volume dedicated to seismic isolation in the 2005 revised Design Specifications for Road Bridges. However, performance evaluation criteria of the seismic isolation devices actually manufactured, delivered and built on field remain still unestablished in Korea. This leads the ordering authorities to require performance evaluation tests adopting foreign criteria, which often need to be performed overseas with substantial costs. Even if limited evaluation tests are conducted in Korea, the poor reliability accorded to the results sometimes delays the attribution of delivery approval of the seismic isolation devices. In order to establish performance evaluation criteria of seismic isolation devices, database relative to diversified evaluation tests of the performances of seismic isolation devices should be collected. However, such experimental research is still in its infancy in Korea. In addition, examination of the correct functioning of seismic isolation bearings actually installed on bridge piers is also insufficient.

In this study, performance level evaluation tests have been actually performed on laminated rubber seismic isolation devices produced in Korea to provide fabrication criteria and performance evaluation criteria for laminated rubber bearings. Accordingly, a large capacity test device has been designed and manufactured to implement the tests. The device selected for evaluation is a circular LRB actually applied in bridges. Evaluation tests were conducted using full-scale LRB with diameter of 851 mm in the rubber part and total height of 215 mm of which the effective horizontal stiffness and equivalent damping ratio have been measured during the experiments. Analysis of the results obtained through loading repeated 10 times with frequency of 0.3 Hz for the design seismic displacement of 40 mm of the LRB has been performed. The values of the effective horizontal stiffness and equivalent damping ratio among the results of the repeated loading test of which values for the first, second and tenth repetitions have been excluded are evaluated. Observation of the results reveals that all of the values of the effective horizontal stiffness and equivalent damping ratio are exhibiting stable characteristics without sudden variation even under repeated loading.

Evaluation tests to survey the dependence of LRB on the vertical loading, shear strain and input frequency have also been carried out. In order to observe the variation of the effective horizontal stiffness and equivalent damping ratio according to the variation of the vertical loading, design seismic displacement of 40 mm has been applied 10 times at speed of 0.3 Hz for each of 50%, 70% and 100% of the design vertical dead load. The effective horizontal stiffness and equivalent damping ratio are obtained through each of the vertical loadings. The effective horizontal stiffness and the equivalent damping ratio exhibited stable characteristics even under varying vertical loading. Tests on the dependence to shear strain have been conducted to verify the eventual variation of the effective horizontal stiffness and equivalent damping ratio according to the shear strain level of the seismic

isolation device. The variations of the effective horizontal stiffness and equivalent damping ratio are measured when varying the shear strain from 20%, 50%, 100% to 150%. Since the thickness of rubber in the sample LRB is 84 mm, shear strains of 20%, 50%, 100% and 150% correspond to horizontal displacements of $\pm$ 16.8 mm, $\pm$ 42 mm, $\pm$ 84 mm and $\pm$ 126 mm, respectively. The effective horizontal stiffness tends to decrease with increasing shear strains to finally converge, while the equivalent damping ratio seems to be practically insensitive to the variation of shear strain.

Because seismic isolation device also performs the role of bridge bearing, the device shall exhibit durability under ordinary displacements of the superstructure according to thermal variations during its lifespan. In the draft of the criteria redacted in the year 2004, 10,000 cycles were specified for the application of ordinary displacement. Following, the equivalent horizontal stiffness and damping ratio were measured after 10,000 cycles and compared to the values measured before test. Since no particular variation is observed between the characteristics before and after 10,000 displacement cycles, sufficient fatigue durability can be expected. Moreover, tests have already been performed to evaluate the dynamic characteristics and stability of LRB subjected to earthquake event as well as fatigue test under long-term repeated loading. Stable dynamic characteristics of the seismic isolation device during earthquake should be developed even under repeated seismic displacements. The number of repeated displacement cycles experienced by the seismic isolation device depends on the duration of the earthquake. An average of 50 effective cycles in one seismic time history seems to be reasonable regard to the researches of seismologists. Therefore, the design seismic displacement has been applied 50 times on the LRB specimen in order to investigate if stable dynamic characteristics are developed under seismic event. The effective horizontal stiffness and equivalent damping ratio have been measured at each cycle. The effective horizontal stiffness appeared to reduce with the number of cycles while the equivalent damping ratio seemed to be nearly not influenced.

Apart of its inherent performances, need is to evaluate also the seismic performance of the LRB installed in real piers. However, the absence of examples of seismically isolated bridge that experienced seismic loading in Korea impedes the verification of the actual performances of LRB. In addition, the seismic isolation design process also remains limited to the evaluation of performances through analytical methods. Therefore, necessity arises to inspect experimentally at least once the performances of a bridge equipped with LRB. To that goal, the LRB has been installed on a manufactured full-scale RC pier and the actual performances of the LRB have been evaluated by applying seismic loading. The bearings installed on the pier specimen have also been fabricated with full-scale like the elastomeric bearing, which presents a square section of 400 $\times$ 600 mm, a height of 181 mm, horizontal stiffness of 3,062 kgf/cm, and effective thickness of 105 mm. The lead-rubber bearing (LRB) exhibits circular shape with diameter of 560 mm, height of 276 mm, equivalent stiffness of 2,285.3 kgf/cm. The pier specimen, presenting aspect ratio of 5.81, was expected to exhibit flexural failure behavior under seismic loading. The input seismic load used for the pseudo-dynamic test is the acceleration records of El Centro earthquake (1940, NS). The relative displacement of the pier using RB and LRB exhibited significant reduction compared to the non-seismic design case using pot bearing. The equivalent damping ratios of RB and LRB computed using the load-displacement curves of the bearings are 6.81% and 21.68%, respectively. The maximum horizontal displacement of the superstructure was measured to be 118.3 mm for RB and 83.2 mm for LRB, which is smaller than the design ordinary displacement of $\pm$ 120 mm assumed for the seismic isolation device. Since LRB exhibits larger damping than RB, larger reduction of the maximum horizontal displacement of the superstructure and of the displacement of the pier can be seen to be obtained through the use of LRB.

Future tests shall evaluate the performances of seismic isolation devices with friction sliding surface. Larger database related to performance level evaluation shall also be collected for laminated rubber bearing presenting diversified sizes. Such process is believed to provide strong basis for the establishment of performance evaluation criteria for reliable seismic isolation devices.

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Effect of variability in response modification factors on seismic damage of R-C bridge columns

A. Mechakhchekh & M. Ghosn
The City College of New York & The Graduate Center of The City University of New York, USA

ABSTRACT: The seismic safety level implied when using the force based design method for proportioning bridge concrete columns is strongly related to the specified value of the Response Modification Factor $R\mu$ used during the design process to account for the nonlinear behavior of the structure. Recent investigations have shown that the presence of high variability in the ground motion characteristics at a site may severely compromise the safety margin in R-C bridge columns when these are designed using the mean value of $R\mu$. Therefore, there is an urgent need to develop a rational approach to account for the seismic uncertainties and material variability during the design of bridge structures. To achieve this objective, this paper proposes a probabilistic approach consistent with the LRFD philosophy to select appropriate $R\mu$ values for use during the design of reinforced concrete bridge columns.

The first step of the process consists of performing a hazard seismic analysis for a given site. The deaggregation technique narrows the selection of ground motions for structural analysis purposes into a number of (M,R) pairs associated with different weights which are equal to their contributions to the total seismic hazard. These weights reflect the probability of occurrence of an earthquake of magnitude M and source to site distance R that exceeds a prescribed ground motion amplitude.

Once the potential seismic threats are identified, the dominant modes along with their percentage contributions to the total hazard are exploited to produce a hundred synthetic ground motion records that represent the most damaging seismic activity at the site. These records implicitly incorporate the variations in the moment magnitudes, epicentral distances, intensities, and the random nature of the earthquake ground motion time histories for the selected site.

The Takeda model is used to perform the nonlinear analysis and account for the hysteretic behavior of typical bridge columns subjected to each of the 100 earthquake records that are collected for each site. The Takeda model defines and updates the stiffness and the strength of the column at every step of cyclic loading using three parameters α, β, and r that control the rate of strength degradation under cyclic loading. The means, standard deviations and probability distribution types of these random parameters are obtained from the results of twenty tests on typical bridge columns.

The damage to typical bridge columns subjected to the 100 earthquake time histories is estimated using the Park and Ang damage index for different limit states such as non-repairable damage and collapse.

Since many of the variables used in the estimation of a column's material properties and structural responses are random, a reliability model based on the First Order reliability Method in conjunction with the Response Surface Method is used to obtain the probability of failure and the reliability index for typical bridge columns. The calculations are executed for a representative site and a range of columns designed to satisfy current specifications for different values of response modification factor.

Table 1 presents typical results obtained for the reliability index, β rel, for the case where failure is defined as damage to the column beyond repair. The table includes columns designed using $R\mu$ values of 1.5 and 2.0 having natural periods $T = 0.5$, 1.0, and 2.0 sec. The table gives the reliability index for different exposure periods varying between 1 year to 100 years.

Table 1. Reliability indexes calculated for San Francisco site (37.46 N,122.26 W) for state damage beyond repair ($DI_{lim} = 0.4$).

Period(sec)	0.5		1.0		2.0	
R_{μ}	1.5	2.0	1.5	2.0	1.5	2.0
1 yr exposure	3.09	2.86	2.92	2.78	2.88	2.78
20 yrs exposure	2.07	1.74	1.83	1.62	1.77	1.62
50 yrs exposure	1.70	1.30	1.42	1.16	1.34	1.16
75 yrs exposure	1.52	1.09	1.22	0.94	1.14	0.93
100 yrs exposure	1.40	0.94	1.08	0.77	0.99	0.77

The results provided in Table 1 can be used to select appropriate response modification factors that satisfy a target reliability. For example, a R-C bridge column of period $T = 1$ sec will survive with only repairable damage all seismic excitations expected at the site within a 50-year return period with a reliability index $\beta rel = 1.3$ (which corresponds to a probability of failure $Pf = 9.7\%$) if it is designed with a response modification factor $R\mu = 1.73$. Similarly, in the case where the bridge column should resist collapse in a 50-year exposure period with a reliability of 1.7 ($Pf = 4.5\%$), then the adequate response modification factor should be $R\mu = 1.85$. For comparison purposes it is noted that the current AASHTO seismic design specifications require $R\mu$ values of 1.5, 2.0, and 3.0 for critical, essential, and other structures respectively. These appear to produce reliability index values on the order of 1.0 to 2.0 for a 75-year return period for the collapse limit state, which is herein judged to be quite acceptable when compared to risks from other threats.

Calculations similar to those outlined in this section can be executed in the future for a variety of sites and based on these results a rational set of response modification factors can be specified in future versions of seismic bridge design codes.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Influence of soundness degradation of railway viaducts on their dynamic response and site vibrations

K. Yoshida & M. Seki
Technology Research and Development Dept., Central Japan Railway Company, Komaki, Japan

M. Kawatani
Dept. of Civil Eng., Kobe University, Kobe, Japan

S. Yamaguchi
Graduate School of Science and Technology, Kobe University, Kobe, Japan

S. Nishiyama
Technical Dept., Nikken Sekkei Civil Engineering Ltd., Tokyo, Japan

ABSTRACT: Approximately 40 years have passed since Tokaido Shinkansen, known as "The bullet train" commenced the operation on October 1st 1964. The transportation system, connecting several principal cities in Japan, such as Tokyo, Nagoya and Osaka, can be regarded as one of the important infrastructures in the country. Thus it is inevitable that the railway structures should always be maintained in sound conditions.

When the structures are deteriorated by a certain reason, not only larger cracks and deformation are observed but also the dynamic response is changed under moving trains. For an appropriate maintenance of such structures, it is important to keep monitoring these changes and take proper countermeasures against them.

In this research, the feasibility is discussed of whether the soundness of railway viaducts can be evaluated by their dynamic response and site vibrations of the ground near the structures under moving trains. Dynamic percussion tests as shown in Fig. 1 are carried out for the soundness investigation of the viaducts located at the sites with high ground vibrations. The effect of degradation in rigidity caused by the deleterious factors such as increasing bending cracks on the dynamic response of the structures under passing trains is also investigated. Here the dynamic response is calculated through a three-dimensional analytical procedure to simulate the traffic-induced vibration of the bridges with parametric structural rigidity. A train set of sixteen cars (ordinary Tokaido Shinkansen train set) is modeled as a nine-DOF system. A standard viaduct bridges are modeled as 3D beam elements as shown in Fig. 2. The dynamic reaction force at the bottom of each viaduct column is obtained in this analysis, which is then used as external force to investigate the degree of the ground vibration adjacent to the viaducts.

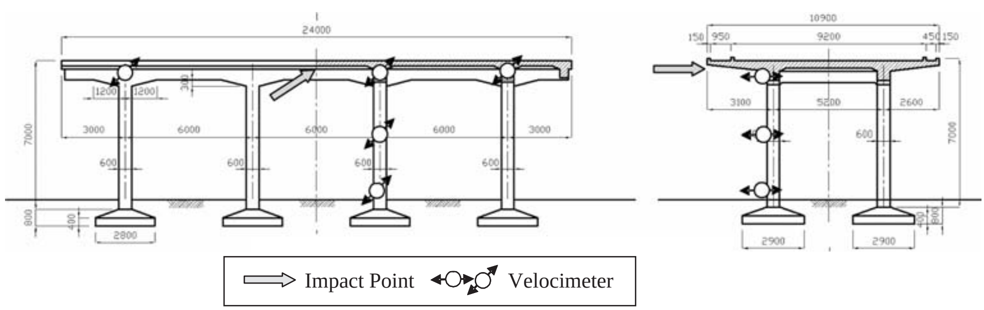

Figure 1. Dynamic percussion test on railway viaduct.

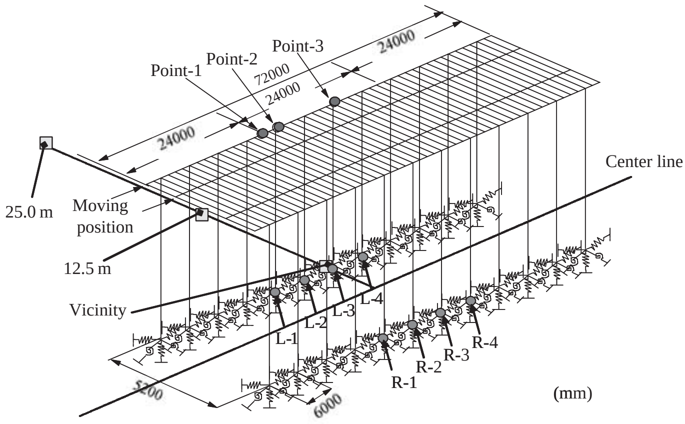

Figure 2. Analytical model of bridge.

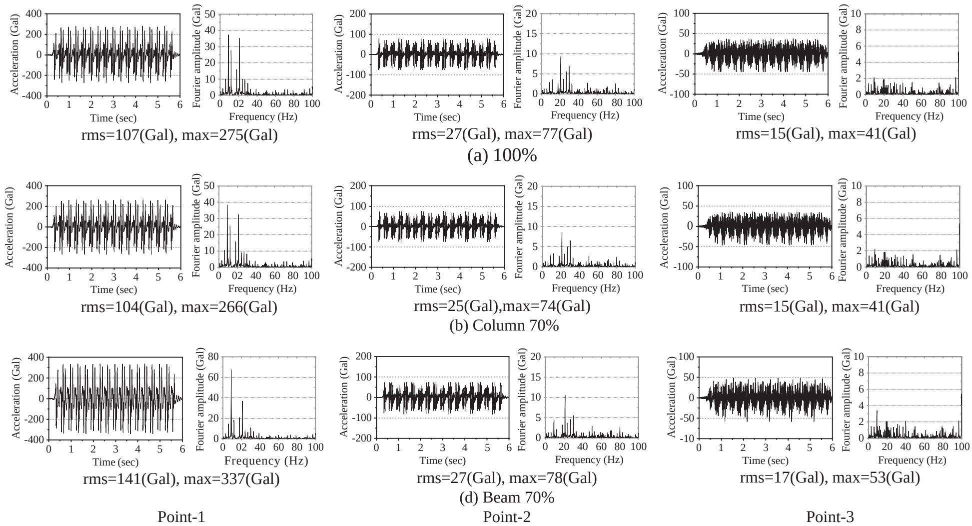

Figure 3. Acceleration of bridge in vertical direction.

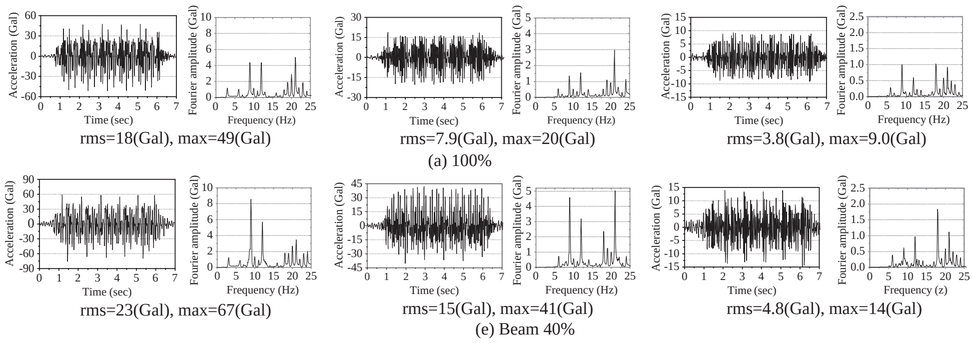

Figure 4. Site vibration in vertical direction.

As a coupled vibration analytical result of each case, the acceleration response and the Fourier spectrum are shown in Fig. 3. And the acceleration response and the Fourier spectrum as a site vibration analytical result of each case are shown in Fig. 4. It is found that the degradation in the soundness of structures results in different dynamic response and in a larger ground vibration around them. Therefore it can be concluded that the soundness of the railway viaducts may be estimated by monitoring their dynamic response and the site vibrations under moving trains.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Dynamic analysis of railway bridges with random vertical rail irregularities

B. Biondi
Dipartimento di Ingegneria Civile ed Ambientale, Università di Catania, Italy

G. Muscolino
Dipartimento di Ingegneria Civile, Università di Messina, Italy

A. Sofi
Dipartimento di Arte, Scienza e Tecnica del Costruire, Università "Mediterranea" di Reggio Calabria, Italy

ABSTRACT: The dynamic response of bridges subjected to the passage of high-speed trains has long been a topic of great interest in the field of railway engineering. A comprehensive review of the history and literature on this subject can be found in Frýba (1996). The problem of dynamic interaction between the train and the railway bridge deserves special attention when the safety of the bridge structure and the riding comfort of rail cars are to be assessed. The studies concerning railway bridge dynamics pointed out that the dynamic response of the bridge and rail cars is influenced by several parameters, such as rail irregularities, vehicle speed, number of coaches, damping of vehicles and bridge, elastic properties of rails and bridge deck, etc. In particular, the track irregularities (elevation, alignment, superelevation, gauge) are considered as one of the most important vibration sources during the passage of trains over railway bridges. The irregularities are usually modeled either as known quantities, exploiting measured data (see e.g., Biondi et al. 2004, Xia and Zhang 2005), or as stationary and ergodic Gaussian random processes characterized by an assigned Power Spectral Density (PSD) function (see e.g. Lei and Noda 2002).

The present paper focuses on the effects of vertical track irregularities on the vibrations of the coupled train-track-bridge system. Specifically, the dynamic response of a multi-span continuous bridge crossed by a running train is studied. The train-track-bridge system is modeled by applying a recently proposed substructure approach (Biondi et al. 2004, Biondi et al. 2005), which enables to handle simultaneously the dynamic responses of the vehicles, rails and bridge, taking into account the interaction among the three subsystems. The basic idea is to treat the running train, the rail and the bridge deck as three substructures. Vertical track irregularities are properly included in the model in order to investigate their influence on both bridge and vehicle vibrations. In particular, the rail irregularities are modeled as stationary and ergodic Gaussian random processes in space, characterized by an assigned PSD function. By properly coupling the discrete (train) and continuous (rail-deck system) substructures, a set of ordinary differential equations with time-dependent coefficients and stochastic excitation, governing the motion of the train-track-bridge system, is derived. In view of the stochastic variability of rail irregularities, the dynamic response of each substructure is described by a random process in time. In the present study, a second-order analysis of the response processes is carried out resorting to Monte Carlo simulation technique.

As case study, an Italian five-span continuous bridge crossed by a train consisting of a locomotive and five identical carriages is considered (Fig. 1). The time-varying mean and variance functions of both bridge and vehicle response are evaluated for different classes of vertical rail irregularities. As expected, numerical results demonstrate that track irregularities have a strong influence on vehicle vibrations, while their effects on bridge response are practically negligible.

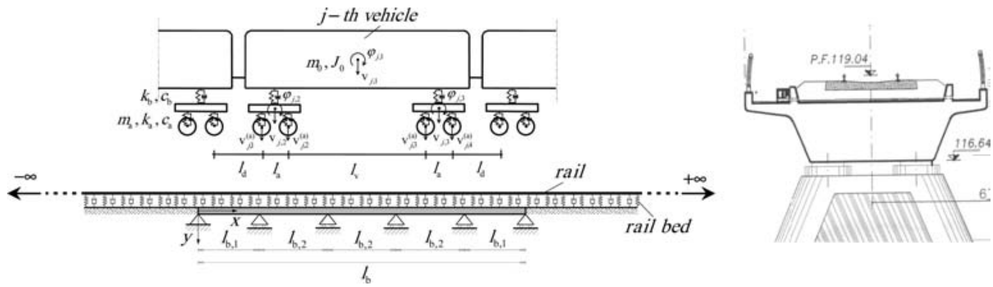

Figure 1. Case study: five-span continuous railway bridge located in Asti (Italy).

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Mitigation of buffeting response for a 800 m cable-stayed bridge during construction

H.K. Kim & S.W. Choi
Department of Civil Engineering, Mokpo National University, Muan-gun, Korea Rep.

ABSTRACT: A buffeting analysis is carried out to evaluate the effect of aerodynamic turbulences on the design of the pylon foundation of a 800 m steel cable-stayed bridge. Both completed and in construction structures are investigated in evaluating the along-bridge-axis bending moment at the tower leg and caisson. Based on current investigations, the maximum bending moment in construction particularly exceeds a design value invoking to find out a measure for suppressing wind-induced vibrations. This study examined a possibility of using buffeting cables or temporal piers as one of feasible measures.

1 INTRODUCTION

This paper evaluates the buffeting responses of a 800 m cable-stayed bridge particularly focusing on the along-bridge-axis bending moment for the pylon bottom and caisson top. Moreover, the seesaw motion of the bridge is expected to magnify the vulnerability to dynamic effect during construction by balanced free cantilever method. The pylon moment in construction usually exceeds that of the completed state even though the design wind velocity decreases somewhat for the bridge in construction.

2 EXAMINED BRIDGE

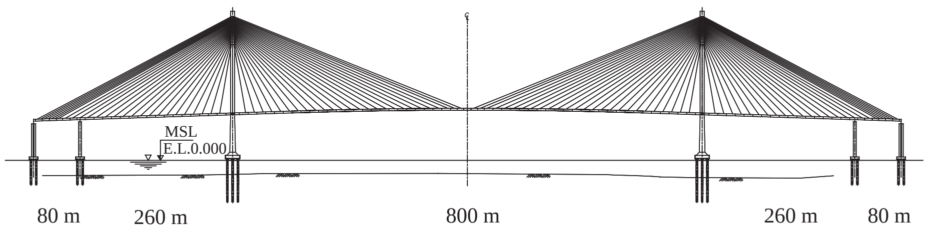


Figure 1. Examined bridge.

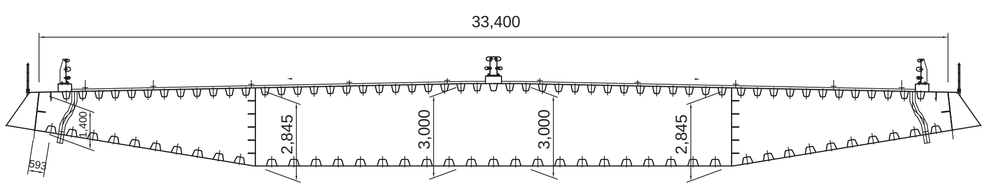


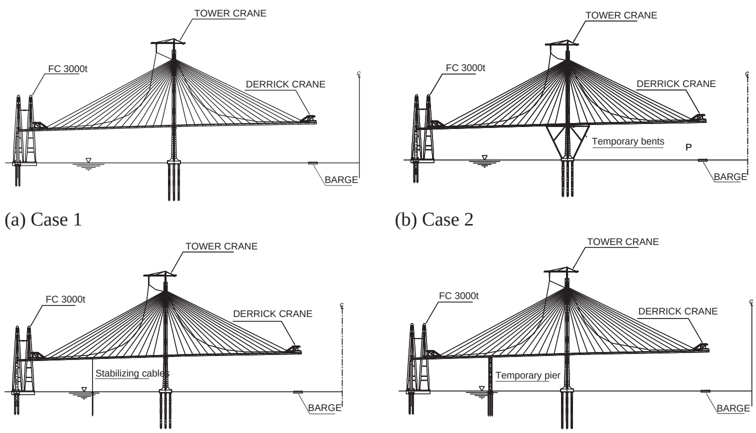
Figure 2. Cross section of stiffening girder (unit: mm).

3 STABILIZING MEASURES IN CONSTRUCTION

Stabilizing cables, temporal bents, temporal piers are basic measures to be considered. The maximum bending moment at the caisson top decreases up to 1.74($\cong$101,241/58,228) times when stabilizing cables are installed.

Table 1. Maximum bending moment (tonf·m) for each stabilizing measure.

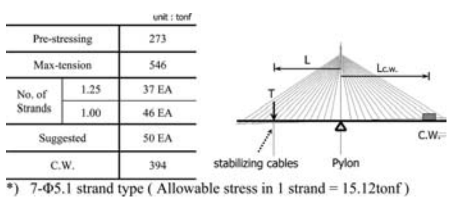
Section	Case 1	Case 2	Case 3	Case 4
Pylon	51,456	53,052	30,631	28,214
Caisson	**101,241**	113,781	**58,228**	53,125



(a) Case 1 (b) Case 2 (c) Case 3 (d) Case 4

Figure 3. Investigated case according to stabilizing device (before the closing of side span).

4 DESIGN OF STABILIZING CABLES



unit : tonf

Pre-stressing		273
Max-tension		546
No. of Strands	1.25	37 EA
	1.00	46 EA
Suggested		50 EA
C.W.		394

*) 7-Φ5.1 strand type (Allowable stress in 1 strand = 15.12tonf)

Figure 12. Proposed dimension of stabilizing cables.

5 CONCLUDING REMARKS

Dynamic member forces for the design of pylon foundation are evaluated for a 800 m cable-stayed bridge utilizing buffeting analysis. Based on the examinations, it is found that the along-bridge-axis bending moment at pylon bottom and caisson top is magnified a lot when the structure is in construction, particularly for the side span closure state. Since this moment is expected to exceed an allowable level, a measure was examined by utilizing a set of stabilizing cables. Due to the site condition, the stabilizing cables were considered only for the side spans and this requires the use of counter weight in main span. Through the current investigations, the authors intended to point out the importance of dynamic wind effect for a long-span bridge and reviewed the stabilizing cables as one of the possible measures.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Blast loading and earthquake effect on reinforced concrete structures

F. Shalouf & S.I. Ahmad
University of Ottawa, Ottawa, Ontario, Canada

ABSTRACT: In the past few decades, the earthquake effect on reinforced concrete structure has become important issue, where as in the recent time after the terrorist attacks on embassies, commercial building and industrial facilities, considerable emphasis has been given to the blast loading beside the earthquake.

The response of a structure to earthquake loading is different than blast loading. During the earthquake, all members of structure work as one unit and share the loads, whereas during the blast loading, some members may carry more loads than the others.

Performance studies of reinforced concrete structures subjected to bombs explosion shows that majority of structures suffered serious damages with blast loading. The non-ductile frame structures constructed prior to enforcement of modern seismic codes suffered more serious damages. It is also evident from such studies that blast resistance of some newly constructed buildings was not sufficient even under low levels of blast loading. The attacks on the World Trade Center Tower in New York City in 1993 and on the Alfred P. Murrah Federal Building in Oklahoma City in 1995, have underlined the importance of research in this area.

Bomb explosions may damage some of the structural elements and hence redistribute load to undamaged elements. For example, damage of any column in a structure can increase the span length of beam/girder which resulted in increase of applied shear and moment. Similarly the sign of applied moment also changes from negative to positive at the damage column location. Damage of column is considered as series damage, and likely leads to progressive building collapse. This aspect of damage due to blast loading requires more attention in design process. Proper detailing of reinforcement in reinforced concrete structures will increase the ductility during strong ground excitation and blast loading but will not prevent local columns failure.

In general, the internal columns can be protected against the bomb explosion by enforcement of security procedure that will limit the chance of internal bomb explosion but unfortunately the external columns are vulnerable to terrorist attack and need to be considered in design to prevent progressive collapse. The structure behaviors during blast explosion depend on; 1) type and size of the bomb, 2) distance between the structure and explosion point, 3) internal or external explosion, 4) strength and type of structure, and 5) location of the explosion i.e. inside or outside of the structure.

In this paper, a 5-storey reinforced concrete frame building was selected and it was designed on the basis of the National Building Code of Canada (NBCC 2005) for Vancouver. The structure was designed twice, the first time without the diagonal members and in the second time with the diagonal members but without external columns in the first three stories. The structural was analyzed under blast loads to see the effect of diagonal members.

From the analysis, the following conclusions can be drawn:

- Non-ductile reinforced concrete frames are likely to fail if the expose to blasting load generated by 500 kg of TNT.
- Reinforced concrete frames, designed according to NBCC-2005 will fail if they expose to blast loading generated by 500 kg and more of TNT for distance 10 m and likely to survive if they expose to 1000 kg if the distance increased to 20 m.
- The use of diagonal reinforced member is an effective method of bracing and strengthening the reinforced concrete frames against the seismic and blast loading. The degree of bracing is dependent on the axial strength of the members.

- The responses of reinforced concrete structure to blast loading are significantly affected by the distance of the explosion from the building. The interstorey drift ratios can be dropped from 6% to 0.35 and from 12% to 0.85 for 500 kg and 1000 kg respectively if the distance increased from 10 to 20 m.
- The drift demands decrease by adding diagonal members to the structures, the drift ratio can be reduced from 6% to 0.45% and from 12% to 0.7% for 500 and 1000 kg located at 10 m. And if distance increased to 20 m the drift ratio can be reduced from 0.35% to 0.035% and from 0.85% to 0.085% for 500 and 1000 kg respectively. In the past few decades, the earthquake effect on reinforced concrete structure has become important issue, where as in the recent time after the terrorist attacks on embassies, commercial building and industrial facilities, considerable emphasis has been given to the blast loading beside the earthquake.

The rational of this paper is to develop an appropriate structure design method which will not only ensure the resistance of structures to earthquake loading but also to blast loading and hence prevent progressive collapse.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Characteristics of lead rubber bearings for elastic response of bridges substructures

J.M. Jara & M. Jara
Division of Graduate Studies, Facultad de Ingeniería Civil, Universidad Michoacana de San Nicolás de Hidalgo, Morelia, Michocacán, MEXICO

ABSTRACT: The main objective of bridge isolation is the protection of the substructure by reducing the force demands and improving its energy dissipation capabilities for limiting the displacement demands. This paper presents the study of typical reinforced concrete bridges in Mexico aimed at determining the isolator parameters for obtaining elastic or limited inelastic behavior of the piers. A parametric analysis of the seismic behavior of short and medium span bridges with lead rubber bearings isolators, using the most common structural types built in Mexico was carried out. The inelastic seismic response of parameterized bridge models with lead rubber isolators was determined. A three-span continuous bridge fully isolated with lead-rubber bearings at each abutment and column bent was analyzed; the bridge deck is presumed rigid with transversal diaphragms in the middle and at the end of each span. The lengths of the spans 1 to 3 are assumed to be equal each other in the range of length from 25 to 40 meters with span length increments of five meters, which give four models to study. The roadway is 10 m width in all the analyzed models. For each bridge length, four possible periods of the structure is considered by varying the column heights in 4, 7, 10 and 15 meters.

The selected model bridges were subjected to three acceleration time history of earthquakes with epicenters in the subduction faults in Mexico. Two of them, Cales and Union stations, correspond to the September 19, 1985 earthquake ($M_s = 8.1$) with epicenter in the Michoacan Coast, and the last one, Manzanillo station, is an earthquake occurred in Manzanillo Coast in October 10, 1995 ($M_s = 7.5$). All stations are located in a 100 km ratio from the earthquakes' epicenter.

The strength and stiffness characteristics of the substructure and the isolation system were the basic parameters of interest in the study. Based on 3D non-isolated models, the substructure and superstructure of the bridges were designed using the SAP2000 program. Modal spectral analysis was carried out using the response spectra established on the seismic code regulations of the Federal Commission of Electricity (CFE 83) considering the bridges on hard soil areas and the zone of the highest seismic activity of the country (Zone "D"). The design of the models were carried out fixing the transversal section dimensions of the column and bent cap elements in the different bridge models analyzed, whereas the girders dimensions were modified according to the length of each bridge model.

Inelastic analyses of the isolated and non-isolated longitudinal bridge models were achieved selecting planar models and subjecting them to time history acceleration records using the DRAIN2DX program. The range of the isolator parameters values required to maintain the piers in the elastic branch of behavior was determined. The results can also be used as a guide for a preliminary selection of the best isolator parameters for bridges located in zones affected by subduction fault earthquakes. The parameter's combination with the four possible columns' height produces 400 isolated bridge cases for each seismic record that were analyzed.

Results of the analyses in longitudinal direction show that the increment of the stiffness ratio (K_{is}/K_{st}), which means stiffer isolator, increases the pier shear forces and the models show a strong dependence on the $\Delta_{Yis}/\Delta_{Yst}$ ratio presenting shear force increments as much as seven times when the strength ratio is changed from the lowest to the highest value.

It is also shown the device's ductility demands for comparing to common values of the manufactured isolators reported. Mean values of the isolator's ductility demands are suitable for strength and stiffness ratio greater than 0.5; however, the use of strength ratios greater than 1.0, conducts to small ductility demands that make either unattractive the inclusion of the devices. Contrary, the smallest strength ratio analyzed ($\Delta_{Yis}/\Delta_{Yst} = 0.1$) leads to high and impractical ductility demands on the isolators.

Substructure force demands are strongly reduced by adding isolation systems. The larger response reductions are obtained for small values of the stiffness and strength ratios. However, some values' combination conduct to impractical isolator ductility demands. It seems reasonable to use strength ratios ($\Delta_{Yis}/\Delta_{Yst}$) in the range of 0.2 to 0.5.

Finally, it was determined the boundary zones between the elastic and inelastic branch of behavior as function of the original period of the bridge, the ratio between the isolated to the non-isolated period and the strength ratio. As regards to the elastic behavior of the substructure, the non-isolated bridge models with a fundamental period below 1.0 sec, can be remained in the elastic region using strength values greater than 1.0. For other period values, several combinations are recommended to keep the substructure in the elastic range of behavior.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Seismic risk management of highway bridges

M. Dolce & D. Cardone
DiSGG, University of Basilicata, Potenza, Italy

L. Pardi
Autostrade per l'Italia S.p.A., Roma, Italy

ABSTRACT: The Italian research project S.A.G.G.I. (Advanced systems for the global management of infrastructures) is aimed at developing an integrated system for the effective management of the transport infrastructures. Bridges and viaducts can be considered as the critical elements of a road network, due to their own characteristics and the considerable consequences, in terms of both repair costs and circulation problems, implied by their degradation and damage. Besides the progressive decay of the structure, a sudden and heavy damage can occur due to an earthquake. Earthquakes of medium-high intensity have high probability of occurrence during the lifetime of a bridge in many regions, such as in Italy, where a new seismic zonation in 2003 has emphasised the seismic inadequacies of most bridges and viaducts. Moreover, the slow degradation of their structural materials can significantly change their strength and ductility and, then, increase their seismic vulnerability.

The above considerations justify the needs for setting up criteria and models to evaluate the seismic vulnerability and risk of bridges and define economically competitive maintenance strategies. That is why S.A.G.G.I. aims also at developing prediction models of the degradation and of the maintenance needs of bridges and viaducts in seismic areas. The research program will be accomplished in three steps:

I. Critically appraising the state of the art on the seismic vulnerability of bridges and viaducts and of the techniques for its reduction;
II. Setting up innovative models to evaluate the seismic resistance of the most common structural types, also validated by an extensive experimental investigation on new and deteriorated models;
III. Setting up operative models for the dynamic evaluation of the seismic vulnerability and of the associated seismic risk of highway bridges and viaducts and implementation of an algorithm exploiting the data base of the Autostrade per l'Italia Company.

The final objective is to develop a tool which is able to evaluate the seismic risk associated to the structure (i) as built, (ii) taking account of the current degradation state, (iii) taking account of the natural evolution of the decay process and the programmed maintenance and/or seismic upgrading interventions.

The second step of S.A.G.G.I., in particular, is related to the set up of an innovative model for the evaluation of the seismic resistance of the most frequent types of highway bridges and viaducts. Such a model shall permit to predict the expected damage for different intensities of the reference earthquake, as described in terms of peak ground acceleration and response spectrum.

The model will necessarily make reference to the structural types and to the geometrical characteristics which are more frequently found in the Italian highway network. These will be selected on the basis of the information contained in the database of Autostrade per l'Italia. Moreover, in the model the typical design situations of the selected structural types, especially concerning percent ratios and arrangement of steel reinforcement in reinforced concrete members, will be taken into

account. These data will be drawn through the critical appraisal of design documents of real bridges and viaducts belonging to the selected types.

The model for the evaluation of the seismic resistance will permit to evaluate the performance levels of the structure under seismic actions of different intensities, on the basis of the available data on the specific structure and of the typical structural characteristics. The model will also be able to account for eventual decay situations and for its time evolution, thus evaluating the consequent performance reduction (increase of vulnerability) of the structure under seismic actions.

The numerical results provided by the model will be compared to the results of experimental tests that will be carried out on large scale structural models, at the Laboratory of Structures of the University of Basilicata. The experimental models will reproduce structural pier-deck sub-assemblages in $^{1}/_{4}$ scale. For some of them the decay conditions will be reproduced with equivalent decay conditions, obtained by suitably reducing the strength characteristics of the materials or the local reinforcement section. They will be subjected to cyclic tests, with increasing amplitude, to evaluate their lateral strength and ductility capacity. Also seismic simulation tests using the pseudodynamic method will be carried out on some of them, to evaluate their actual seismic resistance. The deck will be treated as a rigid mass concentrated at the top of the pier, connected to the pier through bearing devices.

The design of the pier of the models will be referred to the design documents of the pre-selected real structures. The structural models will be also tested in the retrofitted configuration, obtained by introducing different seismic isolation devices, instead of the bearings, between the pier and the deck.

The third step of S.A.G.G.I., in particular, is related to the set up of an operative method for seismic vulnerability and risk evaluations. Such a method will be developed with reference to the most common bridge and viaduct types, previously selected. The method shall take account of both the structural characteristics of the bridge and of the modifications of strength and ductility due to the decay of the materials and/or rehabilitation interventions and/or seismic retrofit interventions. The method will estimate the damage produced by earthquakes of given intensity, described in terms of peak ground accelerations and response spectra, on the bridge under different hypotheses of decay progress, starting from the no decay condition. Therefore it will be possible to examine different situations, in relation to the time evolution of the state of degradation, namely; (i) seismic risk associated to the current state of the structure, (ii) seismic risk associated to the evolution of the decay according to the predicted progress, (iii) seismic risk associated to the evolution of the decay and to programmed rehabilitation interventions, (iv) seismic risk associated to the previous conditions with additional seismic retrofit interventions.

Also the algorithm of evaluation of the seismic risk will be purposely developed for the most frequent previously selected types of highway bridges and viaducts. Once the algorithm is translated into a software, the seismic risk evaluation algorithm will permit to make predictions of the expected damage, in one year or for a given time interval, of a bridge or viaduct in a given site. It will account for the actual state of degradation of the structure and its probable evolution, as well as of eventual rehabilitation and/or seismic retrofit interventions. Therefore it will permit to define an "optimal" maintenance and upgrading strategy with respect to the basic and decay-induced seismic vulnerability problems.

The algorithm and the software will be referred to the database of Autostrade per l'Italia.

In the paper, the first results of the research project, related to the state on the seismic performances, assessment and retrofit of existing bridges, as well as the main aspects of the research developments expected in the research project are described.

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Performance-based design considering ageing of bridge rubber bearing

H.S. Gu & Y. Itoh
Nagoya University, Nagoya, Japan

1 INTRODUCTION

Since the 1995 Hyogoken-Nanbu earthquake (Kobe earthquake) the effectiveness of bridge rubber bearing has attracted the interests of many engineers in Japan. It is because the laminated rubber bearings can not only provide satisfying vertical stiffness, but also sufficient horizontal flexibility. In recent years, the ageing problems have been realized too. It is clarified that ageing causes rubber performance to drop, causing for instance an increase of hardness, and a decrease in tensile strength as well as elongation at break. Thermal oxidation is found to be the most significant factor deteriorating the rubber materials. The heterogeneity of material properties in rubber bearing during ageing process is also studied. It is estimated that from the coldest area to the hottest area in Japan the equivalent stiffness of rubber bearings will increase by 5–25% after 100 years.

However, usually the long-term behavior of rubber bearing is not considered. Since the increase of rubber bearing's shear stiffness will reduce the global natural period, which consequently increase the earthquake force acting on the pier, it is needed to investigate the seismic response of the pier considering the ageing of rubber bearing. This paper focuses on single steel piers with base-isolated rubber bearings. A two-degree-of-freedom model is adopted in dynamic analysis, and the lowest acceptable global natural periods of base-isolated steel bridge piers are proposed considering the increase of rubber bearing's shear stiffness due to ageing.

2 PERFORMANCE REQUIREMENT OF SINGLE STEEL BRIDGE PIERS

The current *Design Specifications of Highway Bridges of Japan Road Association* proposed a two-level seismic design method for moderate (called Level 1) and major (called Level 2) earthquake. Japanese Society of Steel Construction proposed a modified required performance matrix with respect to the earthquake level and the structural importance, as shown in Figure 1.

3 ANALYSIS MODEL OF BASE-ISOLATED STEEL PIERS AND RESULTS

In this study, emphases will be laid on concrete-non-filled thin-walled stiffened steel box piers. In the dynamic analysis, the isolated steel pier with lead rubber bearings (LRB) is modeled as a two-degree-of-freedom (2DOF) model as shown in Figure 2(a). A two-parameter model is adopted as the hysteresis model of the steel bridge pier, as shown in Figure 2(b). As for base-isolated rubber bearing, a bilinear model is used, as shown in Figure 2(c).

Firstly, various global natural periods are assumed and the appropriate rubber bearings are designed. Then time history analysis is carried out with the corresponding accelerogram inputted with respect to each earthquake type and ground type. The maximum displacement of the pier in each case is computed and compared with the performance requirement. The lowest global natural period, which is able to make the maximum displacement satisfy the criteria, is called the lowest acceptable global natural period. Next, the equivalent stiffness K_{Be} of rubber bearing is changed, and in this paper the increase of 10%, 20% and 30% are investigated. The same procedures are

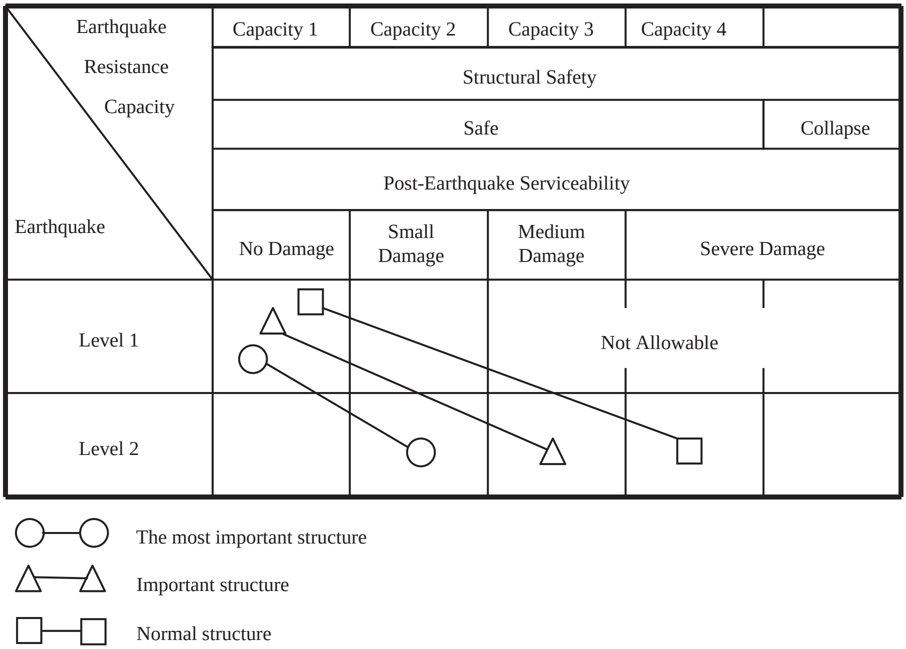
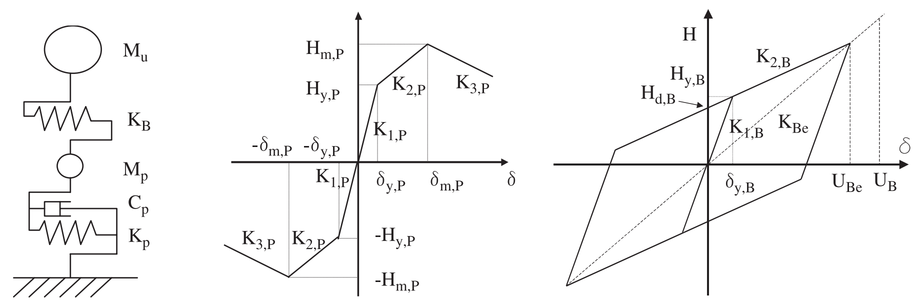


Figure 1. Required performance matrix (JSSC, 2004).



(a) 2DOF model (b) 2-Parameter model of pier (c) Bilinear model of lead rubber bearing

Figure 2. Analysis model of a base-isolated steel pier with lead rubber bearings.

repeated and the corresponding lowest acceptable global natural periods are illustrated for different piers of important and the most important structures.

4 CONCLUSIONS

There illustrations will be helpful to determine the global natural period with the ageing of rubber bearing taken into consideration. And from these illustrations it is found:

– With the equivalent stiffness of rubber bearing increasing due to ageing of rubber material, the lowest acceptable global natural period should be longer.
– For important structure on Ground Type I there is no need to consider the ageing of rubber bearing when attacked by Level 2·Type I earthquake.
– When the most important structure on Ground Type II is attacked by Level 2·Type I earthquake, there is no way to design a rubber bearing unless modify the design of the pier.

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Seismic retrofitting of bridges using slide bearings with bending-type anchor bars

H. Namiki
Kyobashi Corporation, Osaka, Japan

H. Suzuki
Meisei University, Hino, Japan

H. Matsuhisa
Kyoto University, Kyoto, Japan

M. Kimura
West Japan Railway Company, Osaka, Japan

ABSTRACT: The bending-type anchor bar as schematically shown in Figure 1 is expected to have a large energy dissipation ability based on the plastic deformation of mild steel, thus providing an effective device for retrofitting bridges to protect them from seismic attack. In this report, theoretical and experimental studies are presented for implementing seismic retrofitting of bridges.

1 DESIGN CHART BASED ON EXPERIMENTAL STUDY

Three-point bending tests were performed using double-length round bars to provide symmetrical deformation as shown in Figure 2. Bending-type anchor bar have bi-linear load-displacement characteristics as shown on Figure 3. Design loads (displacement = ±75 mm) obtained from the bending tests are plotted in Figure 4, and these points agree well with the double value of the plastic design curve shown by the solid line. Based on these results, a bridge retrofit design chart is obtained as shown in Figure 5.

2 RETROFITTING USING BENDING-TYPE ANCHOR BAR

A numerical time-history analysis using ground motion from the Hanshin-Awaji earthquake with and without a bending-type anchor bar was carried out. Comparing Figure 7 and Figure 8, the

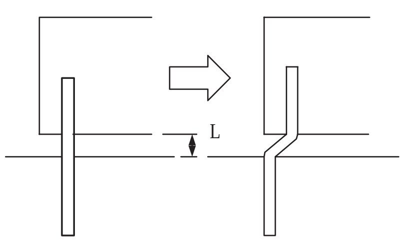

Figure 1. Bending-type anchor bar.

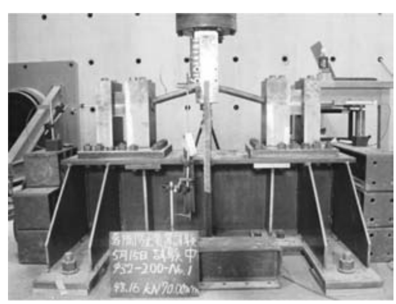

Figure 2. Three point bending test apparatus.

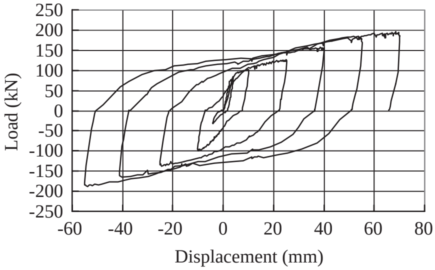

Figure 3. Load-displacement hysteresis ($\phi = 48\,\text{mm}$, $L = 200\,\text{mm}$).

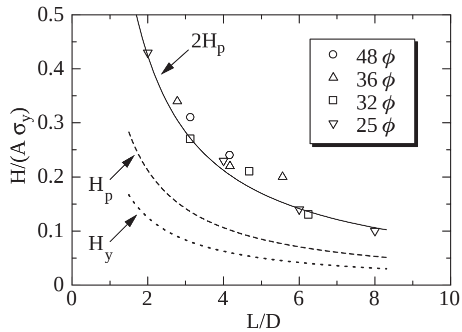

Figure 4. Design load of anchor bar.

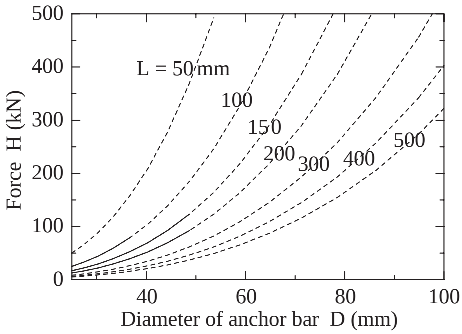

Figure 5. Bridge retrofit design chart.

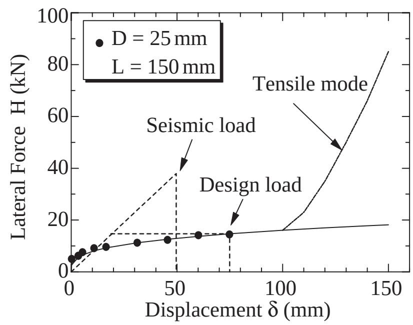

Figure 6. Behavior of a free-ended anchor bar.

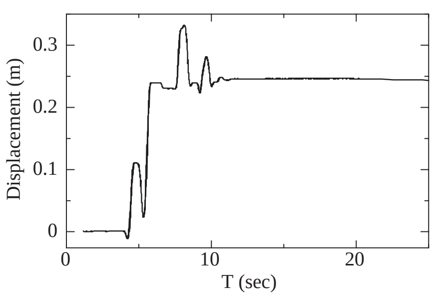

Figure 7. Seismic response. (without anchor bar, $\mu = 0.2$).

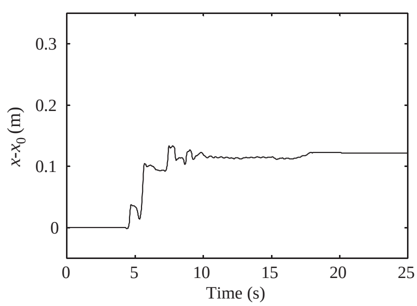

Figure 8. Seismic response. (with bending-type anchor bar, $\mu = 0.2$).

maximum displacement decreased from 0.365 m to 0.133 m (about 1/3). This result demonstrates that bending-type anchor bars can effectively reduce seismic displacement of bridges. The final fracture mode of bending-type anchor bars is also investigated. The energy dissipation of elastic and plastic design for a ductility factor $\mu = 4$ is represented by the area enclosed by the broken line in Figure 6. The black circles show experimental data that coincides well with the broken line based on the capacity design principle. By using free-ended anchor bars with suitable boundary conditions, the loading mode changes from a bending mode to a tensile mode. The maximum displacement increases infinitely, and the lateral load increases nearly to the tensile strength of the bar.

3 CONCLUSION

Bending-type anchor bars made of mild steel are proposed to reduce seismic displacement. Theoretical and experimental studies were carried out and the following results were obtained.

1) Theoretical analysis of earthquake-induced vibration shows that a bridge supported by slide bearings experiences only a few cycles of large relative displacement. Therefore a simple mild steel anchor bar can be used as an effective device for seismic retrofitting of bridges.
2) The design load of bending-type anchor bars is formulated based on a ± 75 mm displacement. This value is in agreement with twice the calculated value using a plastic design model, and can be treated as a nominal yield load.
3) A bridge retrofit design chart is presented, and retrofitting by selected size of bending-type anchor bars can decrease the seismic displacement of a bridge between 1/2 and 1/3.
4) By using free-ended anchor bars with suitable boundary conditions, the loading mode changes from a bending mode to a tensile mode.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Numerical modeling and dynamic behavior of a railway concrete arch bridge over the Vindel River in Sweden

G.J. He
Department of Civil and Environmental Engineering, Luleå University of Technology, Luleå, Sweden; Civil Engineering and Architecture College, Central South Forestry University, Changsha, Hunan, P.R.China

A. Bennitz, O. Enochsson & L. Elfgren
Department of Civil and Environmental Engineering, Luleå University of Technology, Luleå, Sweden

B. Paulsson, B. Töyrä, P. Olofsson & A. Kronborg
Banverket, Borlänge & Luleå, Sweden

ABSTRACT: The Vindel Railway Bridge in northern Sweden is composed of a concrete deck, columns and an arch with a span of 110 m and a height of 22 m. In order to check the possibility to increase the axle load from 225 kN to 250 kN, the properties of the bridge needed to be evaluated. Especially, the dynamic behavior of the bridge is an important factor as it is affecting the load bearing capacity and the serviceability. In this paper, two types of three-dimensional (3-D) finite element models have been developed. One uses shell elements and the other uses beam elements. Based on the 3-D FE models, the effects of nonstructural mass, side spans and connection between columns and deck on the dynamic behavior are discussed. Furthermore, field test data from passing through train are compared with the results calculated by the models. The numerical simulations of the dynamic response coincide reasonably well with the field test data.

1 INTRODUCTION

The bridge situated along the Swedish main northern railway line, crosses the Vindel River near Vindeln some 60 km NW of Umeå. It is a concrete arch bridge with a span of 110 m and a height of 22 m, which was constructed in 1952, see Figure 1. The box rib arch carries a concrete deck with columns. The section of the arch is a box with two cells. The width and height of the section varies with the distance from the abutment to middle span.

2 DISCUSSION AND CONCLUSIONS

In order to investigate the possibility to increase the axle load from 225 kN to 250 kN, the dynamic behavior of the Vindel bridge has been analyzed by using different FE Models. Since a proper selection of finite elements is a key to decrease the simulation errors, different types of finite elements were used to model the bridge structures, i.e. a 3-D shell model using shell elements and three 3-D frame models using beam elements. Based on the comparisons between the results obtained by using several finite element models and available field test results, the following conclusions can be drawn:

1. A shell model and a beam model yield the same shapes and approximately the same frequencies for the first ten global vibration modes. The shell model is more suitable and reliable for analysis of the dynamic behavior. However, the beam model is simpler to use and provides a rational tool

Figure 1. Bridge over the Vindel River in northern Sweden.

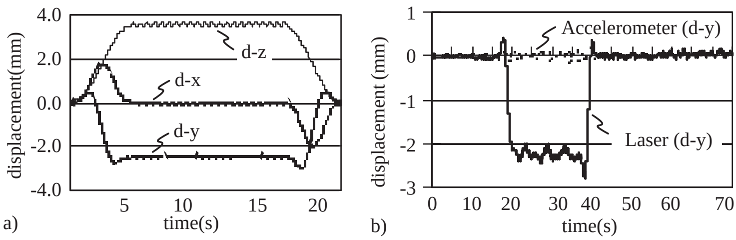

Figure 2. Displacement time-history in transverse (d-z), longitudinal (d-x) and vertical direction (d-y) in middle span of the deck for a train with an axle load of 225 kN and a speed of 90 km/h according to a) a 3-D shell model of the bridge and b) field measurements.

for the understanding of the behavior of the bridge and for the choice of proper placements of vibration sensors if only a few are going to be used.

2. The non-structural mass from the ballast and the rail equipment results in a reduction of the vibration of the frequencies by up to 10%. The side span structure has a negligible effect on the natural frequencies of the beam model. However, the effect of the side column on the dynamic behavior (frequencies and mode shapes) is significant. The frequencies of the bridge will increase without the side column, especially in relation to the lateral bending mode shapes. The superstructure reduces the magnitude of the frequencies.
3. The calculated lowest natural frequency of the Vindel Bridge is 1.13 Hz. The corresponding mode shape is bending vibration of the beam and the arch in the transverse direction. The frequency corresponding to the first mode in the vertical direction is 1.76 Hz. Thus, the bridge is affected more easily by a lateral loading than by a vertical loading, especially under dynamic load cases. Measured values are 1.11–1.15 and 1.77–1.90 Hz respectively (the higher values for a frozen structure). This is very close to the model. The measurements indicate that the stiffness of the bridge increases somewhat for temperatures below zero.
4. From the results of the dynamic response by using a shell model with different train speeds, the largest vertical displacement is at the 3/4 length of the arch span, up to 6.31 mm with a velocity of 90 km/h and an axle load of 225 kN. The maximum transverse displacement is 3.65 mm at the middle span of the deck. In situ measurements on the bridge gave information about its static and dynamic behavior that coincides well with the FE-results from the shell model.
5. The calculations and the preliminary measurements indicate that it ought to be OK to increase the axle load from 225 to 250 kN. However, more analyses and measurements should be carried also with the higher axle load to check this.

Assessment of bridge repair and strengthening

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Bond quality survey of loaded RC beams with CFRP-plate repair using impulse-thermography

R. Helmerich, C. Maierhofer, M. Röllig & R. Arndt
Federal Institute for Material Research and Testing, Berlin, Germany

J. Vielhaber
University of Applied Sciences, Potsdam, Germany

ABSTRACT: The proposed quality system for near surface structural rehabilitation measures of concrete bridges uses active thermography. The transient temperature development during the cooling down process is recorded with an infrared camera and visualizes damages. Validation tests on specimens with defined failures confirmed the accuracy of the procedure. Active thermography, amended by computer simulation, is appropriate to detect both, defects caused during CFRP-application or in-service damages. The procedure for heating the inspected surface was optimized and the measurement procedure was automated during the project Sustainable Bridges. The set-up of a thermography-prototype was tested in four-point bending test.

1 INTRODUCTION

Reinforced concrete bridges in general are not maintenance-free for 120 years, as was widely anticipated during their erection. Many of them show severe damages already after being 25 to 30 years in service, either due to insufficient construction quality, low maintenance level, extreme loading, moisture in the concrete or because of harsh environmental conditions. Deteriorated bridge structures need cost-intensive rehabilitation to remain in service. To be sure about the efficiency of strengthening measures, Non-destructive testing (NDT) can be applied to survey the quality of workmanship and the in-service performance of reinforcing plates, e.g. made of carbon fiber reinforced polymers (CFRP), bonded to the concrete surfaces as external reinforcement.

NDT is the still the exception to the rule in civil engineering, albeit NDT has been fully accepted in industry since many years. The European Commission is funding the development of repair and strengthening measures for railway bridges in order to increase axle loads and sustainability of railway bridges within the project Sustainable bridges. For the evaluation of external reinforcement of concrete structures, the specific parameters of the inhomogeneous concrete require specific investigation.

2 LABORATORY TESTS

In the project, active thermography was chosen for detection of CFRP-plate debonding. Impulse-thermography is a fast and non-contact introduced non-destructive testing method for damage detection in near surface regions. The surface of a structure is heated by using either an internal or external heat source. Different heat sources as radiator, halogen light and flashlight were applied and the results compared. Investigated parts of specimens with designed defects were heated up and the transient heat flux was observed by recording the temperature change at the surface as a function of time. Flashlight was selected as the most effective heating variant.

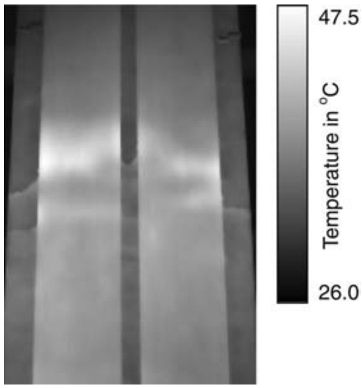

Figure 1. Thermogram recorded after the third load cycle, directly after switching off the radiator (heating source in the first load test). Displayed area: $0.3 \times 0.5\,m^2$.

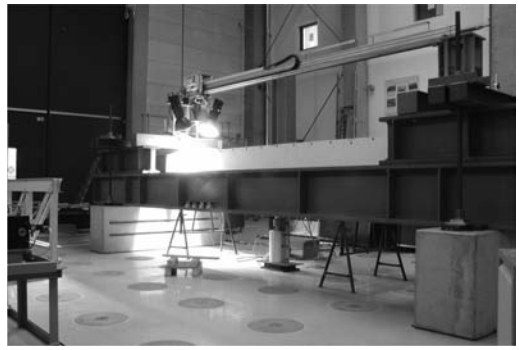

Figure 2. New developed automated scanning system for the survey of the plate bonding under increased loading, displayed area: $0.3 \times 0.3\,m^2$.

The differences between temperature transient curves at surface positions above non-defect regions and above a inhomogeneity are expected to include information about the defect parameters like depth, lateral size and the type of material. The specimens were used for the validation of the method and estimation of its accuracy.

For the assessment of the in-service performance of beams with external CFRP-plate bonding, an automated scanning system for the application of active thermography was developed.

3 CONCLUSIONS AND OUTLOOK

Flash heating and thermographic measurement of debonded areas is a fast and non-contact method for the quality control and evaluation of strengthening and rehabilitation measures using external CFRP-reinforcement. A prototype of an automated scanning system has been developed and was tested with success. The testing method does not require traffic interruption. Even small defects from 1 cm^2 are detectable. The procedure was applied to concrete beams in laboratory and can be tested in a next step on concrete bridge decks or box girder walls.

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Chloride determination for condition assessment and quality assurance by LIBS

F. Weritz, A. Taffe, D. Schaurich & G. Wilsch
Federal Institute for Materials Research and Testing, "Non-Destructive Testing and Environmental Measurement Methods", Berlin, Germany

ABSTRACT: Concrete bridges are damaged by chloride ingress, especially in marine environments and by use of de-icing salts. Chlorides initiate pitting corrosion of the reinforcement and cause a great risk for the durability of a bridge.

Laser-Induced Breakdown Spectroscopy (LIBS) allows the quantitative determination of chloride contents (total chloride content, comparable with acid-soluble chloride). We present results from reference samples and samples from real sites, e.g. car parks, railroad tie, marine structures, showing the potential of the method for condition assessment and quality assurance.

For condition assessment, depth profiles with mm-resolution are available from measurements on concrete cores. The fast measurement technique allows for more tests and thus a better assessment of large structures is possible.

For quality assurance during removal of Chloride contaminated concrete, measurements can be accomplished directly on the surface and information about sufficient removal is immediately available. The success of the work can be documented, and costs for further repair due to insufficient removal of chloride contaminated concrete can be avoided.

1 TESTING PROBLEM, LASER-INDUCED BREAKDOWN SPECTROSCOPY (LIBS)

Chloride ingress into reinforced concrete is a major damage mechanism. High chloride concentrations occur in marine environments and in structures exposed to de-icing salts. There is a large amount of infrastructure, which has to be inspected regularly, such as bridges, structures in a marine environment and parking decks. Damage assessment and quality assurance in the field of concrete repair needs a fast and portable method. So far, these analyses are performed on taken concrete cores by chemical methods in the laboratory.

Laser Induced Breakdown Spectroscopy (LIBS) is a spectroscopic method providing information about the elemental compositions of materials. Basics and applications are described in e.g. [Radziemski 1989]. The advantages of LIBS compared with other techniques have been reported to be the use in harsh environments and on-site, no sample preparation and real-time measurements. LIBS allows for a simultaneous multi-element analysis including main and trace elements.

Applications of LIBS in civil engineering are the analysis of cement and concrete, the detection of hydrophobic coatings, quantitative determination of sulfate and chloride content.

The determined chloride contents compare with total chloride contents determined with other methods, whereas the acid-soluble chloride content is used as reference method.

The sample preparation, experimental set-up and evaluation procedure is described elsewhere in detail [Weritz 2005].

2 PRACTICAL APPLICATIONS

Results from concrete cores originating from a marine structure, a sample taken from a roadway of a car park and a railroad tie with a crack exposed to NaCl-solution are presented. The depth

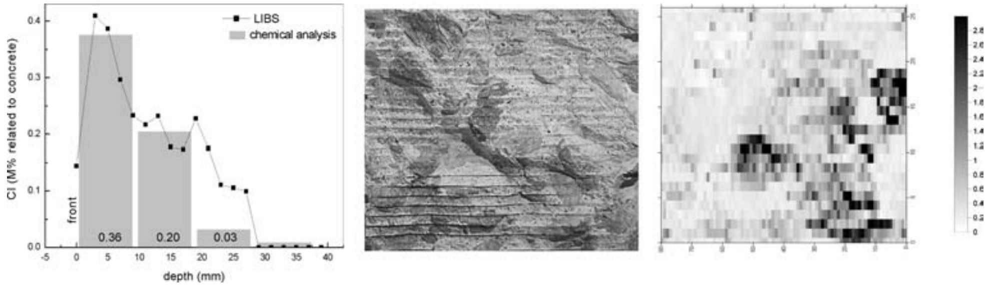

Figure 1. Left: Comparison of chlorine depth profiles measured on a concrete core originating form a marine structure by LIBS and chemical analysis results. Right: Photograph of measured surface (crack layer) and chlorine distribution. Cl-contents in M%/cement.

profiles obtained from LIBS measurements and chemical analysis on a concrete core from a marine environment are shown in Figure 1 (left) proving the good agreement of both methods. In Figure 1 (right) the chlorine distribution measured on the crack sample is shown with the underlying photograph of the measured surface. The chlorine distribution shows a preferential direction of ingress. Chlorine contents are ranging from 0 to 5 M%/cement, whereas the chlorine contents beside the preferential ingress direction are ranging from 0 to 1.5 M%/cement.

3 CONCLUSIONS

LIBS allows for the quantitative determination of chlorine (and sulfur) contents in concrete, element distributions and detailed depth profiles with millimeter-resolution. In principal, the quantitative determination for further elements like sodium, potassium or nitrogen is possible. Results obtained on samples from a marine structure, a car park and a railway tie with crack exposed to NaCl solution are presented. The spatial resolution of 1 mm/spectrum and the fast measurement (measurement frequency 10 Hz) allows for more measurements and thus for statistically more reliable results with respect to a single core and the complete structure. Element distribution can be determined for areas in the centimeter scale, so that e.g. the preferential ingress direction along crack layers can be visualized. The spatial resolution closes a gap between chemical analysis and microscopic methods e.g. RFA, SEM. The simultaneous multi-element analysis allows further conclusions about the cement to aggregate ratio in the investigated sample volume, the identification of materials (e.g. aggregates, coatings) or material changes (e.g. corrosion products) and correlations between elements. The robustness of the method against environmental conditions like dust and vibrations and the possibility to measure without sample preparation predestine LIBS for onsite applications. A portable set-up is under development. Thus, LIBS has a great potential for damage assessment, quality assurance, as input for the calculation of chlorine diffusion coefficients, service life and durability considerations.

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Nonlinear analysis of RC beams with externally bonded plates

J.-G. Park, K.-M. Lee & H.-M. Shin
Department of Civil and Environmental Engineering, Sungkyunkwan University, Jangan-gu, Suwon, Kyonggi-do, Korea

Y.-J. Park
Chief of Structural Division in S & Q Engineering Co, Ltd, Seoul, Korea

1 GENERAL INTRODUCTION

A finite element analysis was carried out to investigate the nonlinear behavior of RC beams strengthened by externally bonded steel or FRP plates in the programs. The Coulomb criterion with a constant failure surface was adopted as the bonding failure criterion for the concrete-epoxy interface where the premature failure may occur. An interface element employing the Coulomb criterion was implemented in a nonlinear finite element analysis program for the simulation of the bonding failure behavior. A Numerical example is presented to demonstrate the validity and efficiency of the present method. Comparison of analysis results with experimental ones shows good agreement. It can be concluded that a reliable strengthening design can be achieved by predicting the premature failure loads of strengthened RC beams using the developed program.

2 COMPARISON OF ANALYTICAL AND EXPERIMENTAL LOAD-DEFLECTION CURVES

Figure 1–3 shows the load-deflection curves obtained by finite element analysis for the three Series I, II and III beams. Premature failure due to debonding is quite effectively predicted for beams S3-150 and S6-190, where the coefficients of cohesion were set to 5.0 MPa and 4.7 MPa, respectively. For beam S3-190, the coefficient of cohesion for the concrete-epoxy interface of beam S3-190 should be greater than 2.9 MPa. Analytical and experimental results show bonding failure following flexural failure when the coefficient of cohesion was set to 5.5 MPa. For the beam 205, the analytical result where the coefficient of also cohesion was set to 5.5 MPa, matched well with the premature load. In case of beams Cplp227 and Cplp221, analytical results show bonding failure following flexural failure when the coefficient of cohesion was set to 5.2 MPa and 5.3 MPa. For the beam Cplp213, the analytical result where the coefficient of cohesion was set to 5.0 MPa, matched well with the premature load.

3 PREMATURE LOADS AND BONDING PROPERTIES

For beams with premature failure due to debonding, the corresponding coefficient of cohesion value was found to be between 4.5 and 5.6 MPa when the angle of internal friction was set to 45°. These numbers fall within the range found from the diagonal shear test. Taking into account many variables, such as the thickness and the bonding length of the steel or FRP plate, the coefficient of cohesion for beams with externally bonded steel or FRP plates ranges between 4.5 and 6.0 MPa.

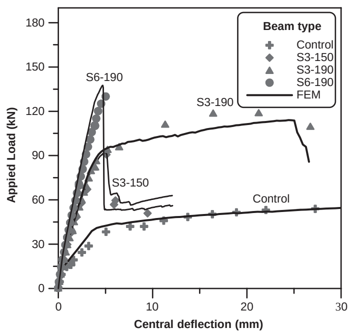


Figure 1. P-D curves for series I beams.

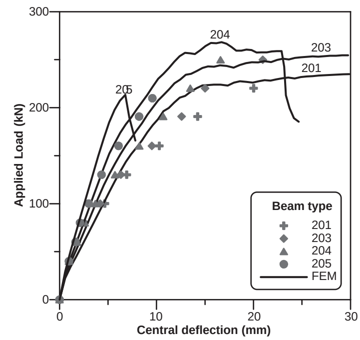
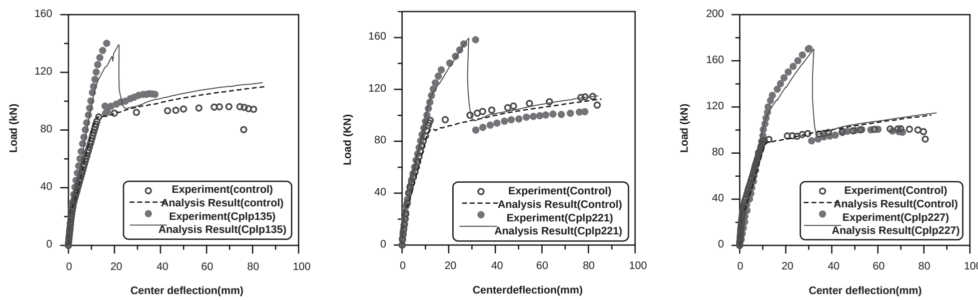


Figure 2. P-D curves for Series II beams.



(a)CPLP135beam

(b)CPLP221beam

(c) CPLP227 beam

Figure 3. P-D curves for Series III beams.

4 CONCLUSIONS

This paper presented (a) bonding properties of concrete-epoxy interface through the diagonal shear testing; and (b) analysis and experimental resulting on RC beams strengthened by externally bonded steel or FRP plates, confirming that the interfacial characteristics are a significant factor in determining the loading capacity and ductility of the strengthened beams.

An interface element employing the Coulomb criterion as a debonding failure criterion was implemented in a nonlinear finite element program to analytically examine the premature failure of strengthened RC beams. Analytical models employed in the program involve the nonlinear material models for constituent materials and the characteristics of the concrete-epoxy interface. Simulation of experimental results was also attained using the developed program.

A reliable strengthening design can be achieved by predicting the premature failure loads of strengthened RC beams using the bonding properties and the finite element program.

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Parametric evaluation of CFRP patch effectiveness in fatigue repair

E.S. Aggelopoulos, T.D. Righiniotis & M.K. Chryssanthopoulos
School of Engineering, University of Surrey, Guildford, Surrey, UK

ABSTRACT: Over the years, there has been an increased research interest on the use of adhesively bonded composite reinforcements for upgrading metallic structures. Although most of this work has concentrated on the use of composites for the fatigue repair of aircraft components (e.g. Baker & Jones 1988), recently, a small number of studies has focused on the use of bonded composite plates for strengthening metallic bridge members (e.g. Tavakkolizadeh & Saadatmanesh 2003a). However, the emphasis of this research has been on the strengthening of deteriorated or degraded steel members with only a handful of cases, where the strengthening of fatigue damaged steel bridge components was investigated (e.g. Tavakkolizadeh & Saadatmanesh 2003b).

In this paper, the problem of a steel plate with a fatigue surface crack growing in the thickness direction, reinforced with an adhesively bonded composite (CFRP) patch is analysed using the finite element method. Parametric 2D finite element (FE) analyses are performed and the effect of a number of parameters, such as the crack depth and the properties of the patch and the adhesive on the interfacial shear and peel stress distributions, the crack tip stress fields and the fatigue life of the repaired plate is investigated.

Results for the shear and peel stresses developed at the steel/adhesive interface indicate that severe stress concentrations occur at the extremities of the patch at the near-crack and end-of-patch positions. Greater peel stresses are found near the end-of-patch region, while the region near the crack is found to be dominated by large shear stresses. The shear stresses in the latter region are found to be several times greater than those near the end-of-patch region.

The beneficial effects of crack patching are observed through the considerable reduction in the stress magnification factor Y. A parametric study whereby the Young's modulus and the thickness of the patch (E_c and t_c) and the adhesive (E_a and t_a), normalised by those of the steel plate (E_s and t_s), are varied in a systematic way, demonstrates that the reduction in Y is influenced primarily by the properties of the patch (see Fig. 1).

Fatigue life calculations based on the crack growth model by Paris & Erdogan (1963) show a significant increase in the fatigue life of the damaged plate, when this is strengthened with a patch. This is shown in Table 1, where the ratio of the fatigue life for a patched plate N_p to that for an unpatched plate N_u is presented for different patch configurations. Typically, the fatigue life is increased by approximately 2.6 to 3.6 times for cracks growing between 10% and 50% of the plate's thickness.

Although these preliminary results appear to be encouraging they do not take into account debonding, which generally decreases the patch effectiveness. Further work in this direction is currently in progress.

ACKNOWLEDGMENTS

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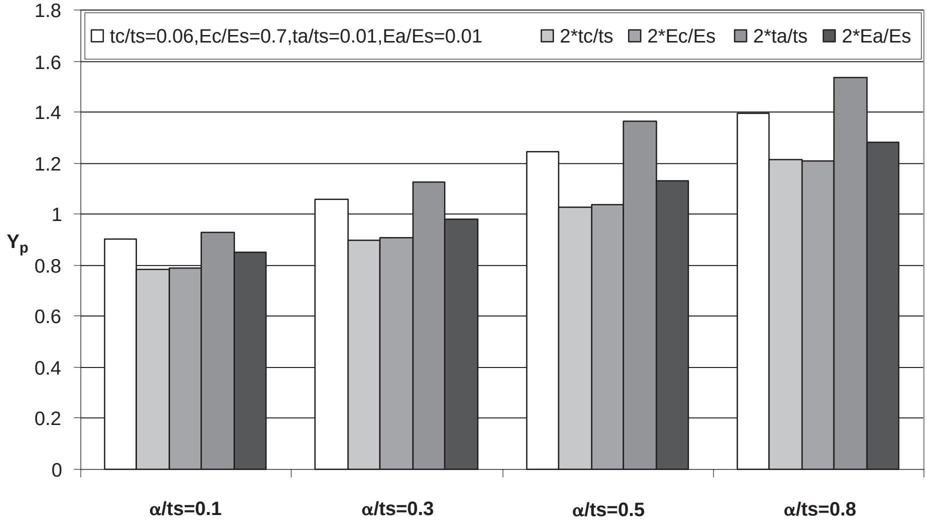

Figure 1. Effect of patch and adhesive properties on Y_p.

Table 1. Values of N_p/N_u for varying patch and adhesive properties.

Case	N_p/N_u
$t_c/t_s = 0.06$, $E_c/E_s = 0.7$, $t_a/t_s = 0.01$, $E_a/E_s = 0.01$, $\ell_c/t_s = 10$*	2.69
$2t_c/t_s$	3.55
$2E_c/E_s$	3.57
$2t_a/t_s$	2.59
$2E_a/E_s$	2.71

*where ℓ_c is the length of the patch.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Sustainable bridges: A European funded project for higher load and speed on railway bridges – WP6 repair and strengthening

B. Täljsten
Luleå University of Technology, Luleå, Sweden and Denmark Technical University, Lyngby, Denmark

R. Helmerich
BAM Federal Institute for Materials Research and Testing, Berlin, Germany

ABSTRACT: European railway bridge stock consist mainly of 4 major bridge types, with age ranging form extremely old masonry arch bridges, middle-age metallic bridges and newly built concrete and composite (steel/concrete) bridges. Small span lengths, less than 10 m, are dominating. Furthermore railways typically assess serviceability as rout bases. Traffic interruptions need to be avoided almost entirely. Many of the existing bridges are in need prolonged life considering the design life when built. In addition it is not uncommon that the owner wishes to increase the speed, weight and traffic volume on the already busy routes. If these situations occur a thoroughly structural investigation is needed. First the remaining capacity is calculated, preferable with methods that consider real material data and loads. If uncertainties regarding for example boundary conditions exist monitoring might be needed. Nevertheless, if calculations and monitoring shows that the load carrying capacity is not enough strengthening can be one alternative to replace the structure. There are numerous different methods to strengthen existing structures of concrete, metal or masonry and the strengthen method chosen is largely dependent on the environment, type of original design, existing object, estimated future use and so on. In a sustainable society, the transportation work carried out by rail ought to be larger than today. In order to enable such an increase, the capacity of existing railway bridges needs to be increased too. This is also the objective of the project "Sustainable Bridges – Assessment for Future Traffic Demands and Longer Lives". There are three specific goals:

1. Increase the transport capacity of existing bridges by allowing higher axle loads (up to 33 tons) for freight traffic with moderate speeds or by allowing higher speeds (up to 350 km/hour) for passenger traffic with low axle loads.
2. Increase the residual service lives of existing bridges with up to 25%.
3. Enhance management, strengthening, and repair systems.

A consortium consisting of 32 partners is carrying out the project. The gross budget is more than 10 million Euros. The partners represent the whole supply chain from user to producer/designer/developer. This paper presents mainly the part considering repair and strengthening systems for railway bridges.

Many of the railways in use today were once built for completely other conditions than those we are facing today, especially when it comes to train speed, axis loads, and traffic intensity. Authorities, the Industry, and also the EU today require the train speeds and axis loads to be possible to increase. As a direct impact, existing railways must be assessed and possibly strengthened in order to meet the requirements on stability, settlements and induced vibrations. The following criteria's have been set up within WP6 – they follow mainly criteria's that have been set up by the Swedish railway authority Banverket. Strengthening works under traffic conditions must comply with regulations from the rail authority. Design of the strengthening should be carried out with reference to the function of the construction, e.g. to improve stability conditions, to reduce settlements or to reduce induced vibrations. Strengthening works should be possible to carry out under "on-going traffic conditions” with minimal impact on accessibility to the railway tracks and without, or with only marginal, reduction of train speed and axis loads. Strengthening should have minimal impact on the position of the railway tracks. Strengthening methods must be cost effective. Strengthening methods must be as harmless as possible to the environment. Strengthening works shall be carried out without damaging existing constructions, e.g. tracks, ties, ballast material, under ballast material, electric wires, signals, drainage equipments, etc. Each strengthening method must have a control program in which precautions, safety aspects, control measurements during installation, and verification after installation are covered. Strengthening should reduce the necessary amount of maintenance work during the life time of the construction, e.g. due to changes of the position of the railway tracks.

If strengthening works should be carried out from the track (work within the track area), the following additional criteria apply: Installation should be carried out on railways closed for traffic and under limited time (may vary from authority to authority). Machines to be used must be adjusted to comply with “Free space along the railway line”. Strengthening works must be possible to carry out without removing the existing tracks, ties, ballast material, electric wires, signals, drainage equipment, etc. In WP6 the strengthening methods studied has the above mention criteria’s in common, even though in some cases deviations might exists.

Work package 6 – *“Repair and Strengthening of Railway Bridges”*, focus on a “toolbox” for Repair and Strengthening methods. WP6 consists of three main deliverables:

“D6.1 A guide for the use of repair and strengthening methods for railway bridges in Europe”. In this deliverable a guide how to repair and strengthen existing railway bridges in Europe will be put together. Existing processes, systems and methods will be included in the guideline. In addition, also new developed method together with best practice methods will be addressed. Furthermore, emphasis is placed on workmanship and quality control during the repair and strengthening process. The second main deliverable is *“D6.2 Research report regarding repair and strengthening of railway bridges in Europe”*. In this deliverable a summary of research and testing together with state of the art reports are conducted. The majority of the research is focused on new and innovative repair and strengthening methods. The last main deliverable is *“D6.3 Field testing regarding strengthening of an existing railway bridge”*. Besides the results from the field tests, also a guides for implementation and assessment will be presented. In WP6 we are 13 partners from all over Europe; From Sweden; Luleå University of Technology, Sto Scandinavia, Chalmers University of Technology, Skanska Teknik AB, Swedish Geotechnical Institute (SGI) and Banverket. From Norway; Norut Teknologi AS, from United Kingdom; Salford University and City University, from Germany, Federal Institute for Materials Research and Testing (BAM) and Rheinisch Westfälische Tech. Hochschule (RWTH), Switzerland is represented by Swiss Federal Laboratories for Materials Testing and Research (EMPA) and finally from Denmark, COWI AS. All partners have different roles in the project and form sub-groups working together. In the coming sections are the content of the deliverables and consequently, work carried out in WP6 briefly described.

CONCLUSION/DISCUSSION

In this paper the work carried out in the European funded project Sustainable Bridges and in particular in WP6 Repair and Strengthening of Railway Bridges has been presented. The project is still ongoing and the presented findings are still preliminary even though a large extent of the planned laboratory testing already has been carried out. However, one important finding in the project is that only minor priorities have been given methods that do not disturb the rail traffic – traditional methods are often used. In this project focus has been placed on new and innovative methods that do not disturb the traffic flow and at the same time are environmental friendly and cost effective. In addition to this a quality assessment manual and a Best Practice Manual are put together. At the moment the WP6 project is in the phase of planning full scale testing on real bridges.

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Lessons learnt from underwater FRP repair of corroding piles

Rajan Sen, Gray Mullins, Kwang-Suk Suh & Danny Winters
Department of Civil and Environmental Engineering, USF, Tampa, FL, USA

ABSTRACT: The poor durability of conventional repairs has led to increased interest in the application of fiber reinforced polymers (FRP) for repairing corroded concrete structures. Over the past decade, several highway agencies in Canada and US have conducted demonstration projects to evaluate the application of FRP for repairing corrosion damage. In most instances, repairs were limited to bridge elements that had corroded due to salt water runoff from faulty expansion joints or salt spray from passing vehicular traffic. These FRP repairs have held up well and show little sign of deterioration.

The development of resins that can cure in water has led to interest in the possible application of FRP for the underwater repair of corrosion damaged piles. Over the past two years, USF has completed three demonstration projects in which corroding reinforced and prestressed piles in two very different tidal regions were repaired using FRP. In the first case, the piles were wrapped in relatively shallow waters where access was possible using ladders. In the remaining two cases, piles wrapped were in deeper waters requiring alternative strategies. In this case, a simple, modular, lightweight scaffolding system was developed to accomplish the task. In all the demonstration projects, two different materials – carbon and fiberglass – and two different epoxy systems were investigated.

An important element of the study was the attention paid to instrumentation to allow monitoring of the long term corrosion performance of the piles. Three different types of probes were used. This included a commercial probe, one developed by the Florida Department of Transportations and a USF probe that eliminated the need for wiring and junction boxes. In addition, the long term bond between FRP and concrete was evaluated from pullout tests conducted on selected specimens.

This paper provides an overview of the three demonstration projects and discusses some of the important lessons that were learnt.

Handling uncertainty in analysis design

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Probabilistic evaluation of model uncertainties in concrete structures

D.L. Allaix, V.I. Carbone & G. Mancini
Politecnico di Torino, Torino, Italy

ABSTRACT: The effect on the structural safety of a damage related to the strength class of concrete is studied in this paper. The reliability method FORM (First Order Reliability Method) is used, a method included within the probabilistic level II. The reliability index β is used as alternative parameter for the structural safety measure with respect to the evaluation of the probability of failure P_f. The structural safety is evaluated using single random variables describing the mechanical and geometrical properties of the structural system.

1 SEMIPROBABILISTIC APPROACH OF EN 1990 AND EN 1992-2

The EN 1990 (CEN 2002) suggests two expressions for the the design values of actions effects E_d and for the design resistance R_d. The safety verification is written in terms of internal actions and external action effects. The limit state function for the generic design situation is a function of the variables R and E.

2 PROBABILISTIC MODEL DESCRIPTION

The probabilistic model used in this paper deals with the geometrical and mechanical properties of the structural system under investigation. The description of the mechanical parameters is based on the process recently envisaged (Mancini 2002), (Bertagnoli, Carbone, Giordano, Mancini 2004), (Bertagnoli, Giordano, Mancini 2005) and accepted in EN 1992-2 (CEN 2005). The concrete peak stress (f_c), both yielding stress (f_y) and ultimate strength (f_t) of the reinforcement steel are described probabilistically. The base (b) and the height (h) of the concrete section are the geometrical properties described within the probabilistic model. The standard deviation's values adopted are based on the standard tolerances for concrete dimensions (Ostlund 1991). The uncertainties related to the actions applied to the structure are taken in account by means of the partial safety factors $\gamma_G = 1.35$ and $\gamma_Q = 1.5$.

3 APPLICATION EXAMPLE

The application has been applied to a simply supported beam and the design has been performed with a linear analysis.

3.1 *Probabilistic analysis*

By means of the HLRF (Haldar, Mahadevan 2000) and the gradient projection methods (Liu, Der Kiureghian 1991), the structural safety with respect to the flexural limit state is evaluated in the midspan section of the beam. The analysis is carried out considering that the concrete strength in the construction phase could be lesser than the prescribed value in the design. The values of f_{ck} equal to 15, 20, 25, 30 and 35 MPa are considered. The results of the reliability analysis are described in terms of the reliability index β and the sensitivity vector α. For each parameter the omission sensitivity factor is used to evaluate the error in the reliability index when the parameter is assumed as a deterministic quantity. The results, in terms of reliability index β, are presented for both the HLRF and the gradient projection methods.

4 CONCLUSIONS

The effect of a damage related to the strength class of concrete is studied in this paper. A comparison is performed in terms of reliability index. The percentage variation of this index is up to 45% when the limit state related to the midspan section is considered. Useful observations related to the possibility of reducing the probabilistic model size without loosing accuracy in the estimate value of the reliability index are addressed.

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Handling uncertainty in reliability analysis of concrete structures

Fabio Biondini
Department of Structural Engineering, Politecnico di Milano, Milan, Italy

Franco Bontempi
Department of Structural and Geotechnical Engineering, University of Rome "La Sapienza", Rome, Italy

Pier Giorgio Malerba
Department of Structural Engineering, Politecnico di Milano, Milan, Italy

1 INTRODUCTION

Reliability-based concepts are nowadays widely accepted in structural design. However, before such concepts can be effectively implemented, the actual design problem often needs to be considerably simplified. This is mainly due to two reasons. First, in their simplest formulation reliability-based procedures require the structural performance to be represented by explicit functional relationships among the load and the resistance variables. But, unfortunately, when the structural behavior is affected by several sources of nonlinearity, as always happens for concrete structures, such relationships are generally available only in an implicit form. Second, for structural systems with several components, a complete reliability analysis includes both component-level and system-level estimates. Depending on the number and on the arrangement of the components, system reliability evaluations can become very complicated and even practically impossible for large structural systems.

The actual role played by these limitations may strongly depend on the criteria adopted to model the uncertainties related to the geometrical and mechanical properties involved in the structural problem. Basically, uncertainty modeling can be approached by a probabilistic or by a fuzzy formulation. The probabilistic approach assumes the intrinsic stochastic variability of the random variables as known. In the practice of structural design, however, it is very frequent that a lack of information occurs about such randomness. Consider for example a beam imperfectly clamped at one end. This link is usually modeled through a rotational spring having uncertain stiffness. The translation of this problem in probabilistic terms is not simple, since no information are usually available about the random distribution of the stiffness value. Conversely, it appears more direct and reasonable to consider a band of situations in between the hinged and the clamped ones, which defines a design domain large enough to include the actual one under investigation. This makes the fuzzy approach, in which the uncertain parameters are bounded between suitable minimum and maximum value, more meaningful for a consistent solution of the problem. Situations of weak structural coupling are very frequent in structural engineering, as for instance happens for structures built in subsequent phases, for large span cable supported bridges and for high rise buildings.

In previous works the authors proposed theoretical formulations and numerical procedures for reliability analysis of concrete structures by using a probabilistic approach (Bontempi et al. 1998, Biondini et al. 2004*a*), or a fuzzy approach (Biondini et al. 2000, 2001, 2004*b*). Such methodologies are based on detailed and representative mechanical models of the structural behavior, and are able to handle implicit formulation of the performance relationships and to perform system-level evaluations even for large structural systems. This study presents the main aspects of the proposed procedures and compares the results of both the probabilistic and fuzzy modeling of the uncertainties with reference to the reliability analysis of a prestressed continuous beam. The capability of the presented approaches to handle complex structural systems is also shown through the application to an arch bridge and a cable-stayed bridge.

2 PROBABILISTIC AND FUZZY APPROACH TO RELIABILITY ANALYSIS

The probabilistic approach assumes the intrinsic stochastic variability of the random variables as known. In this way, by detoting with $f_\theta(\theta)$ the density probability function of the safety factor θ, the probability of failure can be evaluated by its integration within the failure domain $D = \{\theta|\theta < 1\}$. In practice the density function $f_\theta(\theta)$ is not known and at most some information is available only about a set of n basic random variables $\mathbf{x} = [x_1\ x_2 \ldots x_n]^T$ which define the structural problem (e.g. mechanical and geometrical properties, dead and live loads, prestressing actions, etc.). Since the functions $\mathbf{y} = \mathbf{y}(\mathbf{x})$ which describe the structural response (e.g. stresses, strains, etc.) is generally only available in an implicit form, a numerical approach is required. The problem can be approached by Monte Carlo simulation, where repeated analysis are carried out with random outcomes of the basic variables $\mathbf{x}$ generated in accordance to their marginal density functions $f_x(x_i)$. Based on the sample obtained through the simulation process, the density function $f_\theta(\theta)$, or its cumulative $F_\theta(\theta)$, can be derived for each given limit state, and the corresponding probability of failure $P_F = F_\theta\ (\theta^*)$, as well as the reliability index $\beta = -\Phi^{-1}[F_\theta(\theta^*)]$, can be evaluated (Bontempi et al. 1998, Biondini et al. 2004*a*).

A fuzzy membership function $\mu = \mu_{\tilde{A}}(x)$ assigns to the parameter x a degree of membership varying in the closed interval [0, 1]. In other words, a fuzzy membership function is a *possibilistic* distribution suitable to describe uncertain information, when a probabilistic distribution is not directly available. Therefore, analogously to the probabilistic case, in fuzzy structural analysis the membership functions of the uncertain data $\mathbf{x} = [x_1\ x_2 \ldots x_n]^T$ must be processed in order to achieve the corresponding membership function of the limit multipliers θ. To this aim, it is useful to discretize the continuous fuzzy variables by choosing some levels of membership $\alpha \in [0, 1]$, called α-levels, which represent different levels of uncertainty. In this way, the relationships among fuzzy sets can be studied by using the concepts of classical logic and the methodologies of the interval analysis, applied to each α-levels. However, such problem may be not straightforward, since the response interval $[\theta_{\min}; \theta_{\max}]$ corresponding to $[\mathbf{x}_{\min}; \mathbf{x}_{\max}]$ cannot be simply obtained from $\theta(\mathbf{x}_{\min})$ and $\theta(\mathbf{x}_{\max})$, but needs to be find by solving an anti-optimization problem (Biondini et al. 2000, 2001, 2004*a*).

3 CONCLUSIONS

The results presented in this work show that the proposed procedures for reliability analysis of concrete structures are effective both in easily handling implicit formulation of the performance relationships, as well as in performing system-level evaluations even for large structures. It is also outlined that probabilistic and fuzzy approaches should not be considered as alternative each to the other, given that the two methods account for different aspects of the same problem. However, an autonomous approach to reliability assessment, like the probabilistic formulation proposed by the codes, should find a higher rationality in a fuzzy approach which, due either to the real nature of the involved uncertainties, or to a higher simplicity of the mathematical formulation, seems to be more suitable for design purposes.

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Reliability of simplified analytical models for the analysis of FRP reinforced masonry frames

U. Ianniruberto & Z. Rinaldi
Department of Civil Engineering, University of Rome "Tor Vergata", Rome, Italy

ABSTRACT: Masonry arch bridges are very important structures of the international building heritage. Therefore their structural assessment and the analysis of the possible strengthening interventions is a very interesting research theme. The changes in service conditions and the seismic risk often require these structures to be strengthened or repaired. The influence of the strengthening technique on the load bearing capacity should always be evaluated.

The use of innovative systems to retrofit and/or reinforce structures has been widely developed in the field of civil engineering. Specifically the use of composite sheets applied at masonry structure surfaces represents a very attractive solution in the field of practical applications. This choice is supported by the fact that the composite material is light, easy to install, without any types of corrosion problems and with a high specific strength.

Many experimental investigations (see for instance Briccoli Bati & Rovero 1999, Faccio et al. 1999, Valluzzi et al. 2001) showed the efficacy of the technique in improving the strength of the structures and the problem of evaluating analytically the failure load of a strengthened arch has been addressed, with some simplifying assumptions, in many papers (Como et al. 2000, Faccio et al. 1999, Valluzzi et al. 2001). In Como et al. (2001) a simplified general method for calculating the failure load of a single arch completely reinforced at intrados or extrados and subjected to vertical loading at the crown or at the haunch has been proposed. In Fabiani et al. (2004) a comparison between analytical and numerical analysis of FRP reinforced arches is shown.

This paper focuses on the problem of the evaluation of the effects of the FRP reinforcement at the intrados of a masonry frame. Indeed, in the case, the limit design theorems can lose their validity, since the FRP sheets prevent the formation of the hinges required by the mechanism formation. Local failure can occur and then the mechanical properties of the materials (masonry, FRP and interface) are involved and have to be considered in the analysis.

A simplified analytical model to evaluate the strength increase provided by the FRP strengthening intervention has been already developed by the authors (Ianniruberto & Rinaldi 2004) and this paper aims at the validation of the model through a comparison with non-linear numerical analysis.

A reference masonry frame, made by two piers and an arch, which can be considered the reference module of typical bridge structural schemes, is considered and subjected to both vertical (V) and horizontal (H) concentrated forces. The frame has a square cross section whose side is equal to 0.42 m.

The application of the simplified analytical model (Ianniruberto & Rinaldi, 2004) is useful for the construction of the failure interaction domain (H, V) of the frame.

The model is based on the analysis of the behaviour of the frame at failure through the definition of structural statically determined schemes corresponding to different loading processes characterized by different values of the ratio $\eta = H/V$.

For each phase it is possible to evaluate internal forces and internal stresses as a function of the vertical and horizontal load at every section of the structure and the collapse domain in the plane (H,V) for any failure criterion (masonry crush, FRP debonding or failure), can be drawn.

The model is validated with numerical analyses, based on fiber discretization of the sections.

In agreement with the assumptions of the analytical model, the masonry is simulated as a no-tension material, with the non-linear compression behaviour. The FRP sheets are considered unable to sustain any compressive stress and are characterised by an elastic behaviour.

The analyses have been performed both in displacement and force control, by increasing the external action up to the structural failure.

The comparison between numerical and analytical results is effectively summarized through the drawing of the different collapse loci, and through the analysis of the thrust lines, together with the evaluation of the stress and strain values in the masonry and FRP materials. In each case the analytical model gives results that are very close to the numerical ones.

Finally the results referred to the analysed case show that the main assumptions of the analytical model are true and that the model is very reliable for the evaluation of the strength of the FRP reinforced structure.

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The role of monitoring in the management of uncertainties and residual life of existing structures

A. Del Grosso & F. Lanata
Department of Structural and Geotechnical Engineering, University of Genoa, Italy

ABSTRACT: It is widely recognized that extension and efficiency of infrastructure are amongst the factors that highly influence the competitiveness, the potential for economic growth and the quality of life both in developed and in emerging countries. As a consequence, the construction of new and the management of existing infrastructure are today posing complex economical and technological problems to owners, governmental bodies and societal organizations worldwide.

The economical effort of building, replacing or retrofitting of infrastructure is particularly important in industrialized countries, where the existing infrastructure stock is subjected to ageing and obsolescence. For example, the majority of the transportation infrastructure existing in the Europe and in the United States has been built from the 50's to the 70's, when the characteristics of the traffic intensity and loads were significantly different from that of today and when some of the phenomena related to material degradation were not fully discovered or understood. Moreover, the problem is not confined to transportation infrastructure, but also includes other types of infrastructure like energy production and distribution systems, water supplies and wastewater treatment plants, communication facilities, schools, hospitals, homeland defense installations, etc.

The development of infrastructure management systems able to optimize the costs needed to keep existing facilities at an acceptable level of performance has been therefore the subject of extensive research activity in the past few decades, leading in some cases to standardized codes of practice but also giving rise, especially as concerning the statement of global approaches, to challenging interdisciplinary problems that the scientific community has come to face in the more recent years and that represent a very promising research field in civil engineering.

Basically, to establish a global approach to the management of existing infrastructure, an optimal strategy for programming maintenance and retrofitting interventions shall be found in such a way to minimize some "cost" function while keeping some "performance" or "condition" measure (index) within a specified acceptable bound.

Maintenance interventions may be decided on the basis of time or condition, cost and cost-condition relationship to the aim of preventing excessive degradation of the performance indices and delay retrofitting activities that have the scope of carrying the performance indices back to their target values.

There are many parameters that influence the decision making process, like the law of degradation and the effect of the interventions on it. Essentially these parameters are of a stochastic nature and the effect of uncertainties renders this decision making process very cumbersome. Uncertainties derive from the complex phenomena that govern material degradation, unknown factors concerning the building process, the gap between design hypotheses and real conditions, from cost evaluations and from the definition of performance measures and corresponding limit states with respect to the conditions that in the reality render the infrastructure unusable or unsafe.

Many practical procedures already in use by infrastructure owners and state agencies for the definition of performance indices and strategies are based on heuristics; these procedures have revealed to be effective in the management of an infrastructure containing a large number of homogeneous critical structures (e.g. bridges) essentially to the aim of prioritizing interventions.

However, many recently proposed approaches are based on a more scientific background and take advantage of algorithmic procedures derived from the context of mathematical optimization and from the control theory.

The paper is aimed at discussing some of the relevant problems arising from the conception, design and installation of a monitoring system able to perform a permanent "health monitoring" function and to provide the information needed to reduce the effect of the uncertainties in view of the application of policy selection procedures for infrastructure management.

After the type of monitoring has been selected, the different components of the system shall be properly designed. For designing a sensory system, the scope of monitoring i.e. the physical parameters that shall be taken under control and the phenomena that need to be traced, shall be clearly understood. The type and number of sensors, their characteristics and their locations are strictly dependent on the above considerations and will greatly affect the economy and durability of the system. The physical quantities that may be measured include mechanical (strains, displacements, etc.) chemical, thermal quantities, as well as environmental (wind or wave characteristics, etc.) or loading (weight-in-motion).

To design a sensory system able to perform economically, redundancy of measurements shall be reduced to a minimum. This requires a careful study of the features of the structural response that are to be extracted from the measurements. It has to be recalled that the cost of the data carriers and data loggers depends on the number of sensors and that, in a continuous monitoring system, an excessive number of sensors dramatically increases the amount of data to be processed.

Maintainability of a sensory system may represent a big issue. To date, we have no information on the expected life of many of current sensor technologies compared to the expected duration of monitoring. In view of a continuous monitoring extended over the entire operational life of the structure, the problem of updating the technology of the sensory system shall be considered. Accessibility to critical components of the system shall always be ensured.

It should be pointed out that present experience demonstrates that sensors have been continuously used on real structures for more than ten years without encountering significant problems. However there is no clear evidence that the systems will continue to operate for the expected life of the structure, i.e. for the next 50 or 100 years.

Reliability of sensors, data carriers and data loggers for the long periods of observation expected for as continuous monitoring is also very important to minimize the down times and the consequent loss of data. The possibility of measurement failures shall also be considered.

Flexibility in the sensory system means that the possibility of reconfiguring the sensor network during the operational life of the system shall also be appropriately considered. Flexibility can be obtained through a careful study of the data carrier/data logger subsystems. For large systems the identification of sub-networks, the use of conventional or wireless platforms, of distributed data acquisition modules and web-based applications shall also be considered.

In designing the data management subsystem for a continuous monitoring solution, the following issues are very important. The system shall be able to form and render easily accessible the time series of the readings of each instrument in the system. These time series shall be always referable to unique time origins, processed for eliminating errors and filling the gaps due to temporary malfunctioning of some subsystem or instrument and for filtering out the effects of the phenomena that will not be considered in the subsequent processing.

Data analysis and interpretation is the subsystem that is responsible for feature extraction and damage identification.

Relatively extensive experiments and practical applications have been performed by using model-based techniques associated with static and dynamic monitoring. Non-model-based techniques, especially if applied to continuous static monitoring, have been mainly the subject of research studies and it has to be stated that, for the time being, there is no clear evidence from real experiences that damage detection and identification is a reliable outcome of their application.

Sensory systems are also discussed in the paper, because current and near-future technologies provide a wide range of new possibility for the monitoring of structures. Among these technologies, the capabilities of fiber optic sensors, GPS and wireless integrated platforms are shortly presented.

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Excessive deflections of concrete bridges affect safety, maintenance and management

V. Křístek & A. Kohoutková
Czech Technical University of Prague, Prague, Czech Republic

ABSTRACT: Apart from durability, the most important factor in the whole life design of prestressed concrete bridges is the Service Limit State. Various factors affecting increase of deflections (creep, shrinkage, shear effects and arrangement of layout of prestressing tendons) are assessed. It is recommended that bridge design should be performed on two different levels, including two equivalent parts – not only common stress analysis, but also optimisation of prestressing tendon layout should be compulsorily performed to reach acceptable deflection variations.

The most important factor in the whole life design of reinforced and, in particular, prestressed concrete bridges, is serviceability, durability and appropriate reliability of such bridges. From the point of view of the Service Limit State, prestressed concrete bridges are very sensitive to long-term increase of deflections. This phenomenon has paramount importance for serviceability, durability and long-time reliability of such bridges. The costs of reduced service life of structures due to excessive deflections can be enormous.

There are many reasons for the deflection increases, which usually are coupled together. Three of them are briefly discussed.

1. Realistic prediction of concrete creep and shrinkage is of crucial importance for achieving good durability and long-time serviceability of concrete structures. Shrinkage – besides the axial shortening of the bridge beams – can also affect the box girder deflections due to nonuniform development of shrinkage resulting from different thicknesses of flanges. It has been found that the curvatures of a box girder bridge in the cantilever stage first increase over a long period, but then they reach a maximum and afterwards they decrease. The result is a delay in the onset of significant downward of deflections of box girders, which gets shifted to a much later period than would be expected according to common level of understanding. Schematically, these tricky interactions are shown in Fig. 1.
2. It has been proved that neglecting shear deformations and the shear lag can result in underestimation of deflections due to action of *vertical loads. The prestressing effects*, being accompanied by no or minor shears, are, on the other hand, predicted by elementary bending theory rather

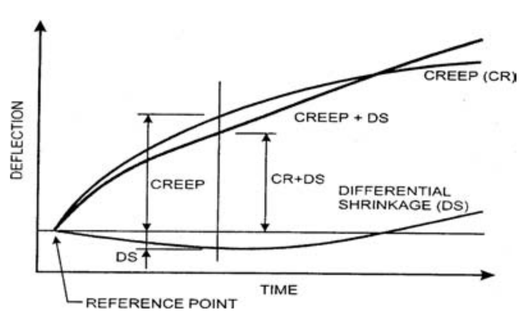

Figure 1. Deflection variation affected by differential shrinkage.

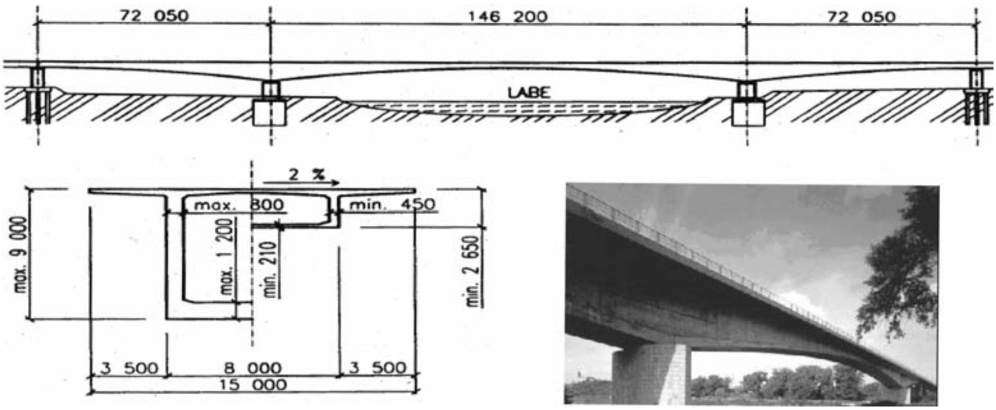

Figure 2. Bridge over the river Vltava at Mělník.

satisfactorily. This is why *the concept of effective widths*, if they are assumed the same for evaluation of effects of vertical loads and prestressing, is in the case of prestressed concrete bridges *completely wrong*.

3. Efficiency of prestressing to reduce deflections is very significantly affected by the layout arrangement of tendons. Among the important issues concerning this efficiency belongs the question in what manner the bridge long-term deflection are influenced (in *the final structural system*) by the prestressing layout arrangement applied *during the construction stages*. It can be shown that low deflections of the bridge *during the cantilever construction stages* do not automatically result also in acceptable deflections during the bridge service life.

To elucidate significance of the tendon arrangement layout, the bridge over river Labe in mělník (fig. 2) is considered and analysed from this point of view. the main task is to discover a possible unsuitable arrangement of the tendon layout that can result in harmful effect – such tendons cause long-term *increase* (instead of reduction) of the midspan deflections.

It can be concluded that 22% of total of prestressing tendons affect the investigated bridge unfavourably, contributing to increase of deflections. Especially, the tendons located at the bottom surface of the first and third spans were proved to be absolutely harmful, since all of them produce deflection increase in the central region of the main span of the bridge. From the tendons located at the top surface, applied during cantilever erection, the straight tendons, which are passively anchored in the vicinity of internal supports and follow the top surface, are harmful. In the discussed bridge, the unfavourable tendons in the first (or in the third) span are anchored.

Typically at distance approximately 15 m from the ends of the bridge, the unfavourable tendons in the main span are anchored typically at distance approximately 30 m from the midspan.

Lessons from assessment of existing bridges can be learned: bridge design should be performed on two different levels, including two equivalent parts – not only common stress analysis, but also optimisation of prestressing tendon layout should be compulsorily performed to reach acceptable deflection variations. Practicing engineers, for added convenience, can benefit in the design from a program (the web address http://creep.fsv.cvut.cz/test) that makes the assessment of the tendon layout immediately accessible.

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Characterization of the structural performance of existing r.c. bridges and basic criteria for rehabilitation and refurbishment: Experiences in Northern Italy

C. Modena, P. Franchetti & M. Grendene
Department of Structural and Transportation Engineering, University of Padova, Padova, Italy

ABSTRACT: The r.c. bridges built in Italy in the post-WWII period show currently dimensional, structural and functional deficiencies caused by different factors.

Their inadequate structural performances are mainly due to the intrinsic weakness of some structural components, to deterioration phenomena and to the updating of structural codes, especially in relation to seismic classification.

The intrinsic weakness is an obvious consequence of the original design criteria (lower design loads and "performances levels", poorer technical solutions, etc.).

Physico-chemical degradation finds particularly favourable conditions where damage and micro-cracking occur in over-stressed concrete and steel. Moreover, the lack of adequate maintenance causing unevenness of the road surface further increases the effects of loads due to the dynamic amplification effects.

Rehabilitation is usually coupled with refurbishment interventions aimed to improve the safety and comfort of road users. This is obtained, in particular, by widening the deck, eliminating the expansion joints (which are also one of the most critical elements for durability), modifying the approach spans and ancillary structures, laying life-lines on pavements, adapting road markings, safety barriers and parapets, and lighting.

The design of rehabilitation and refurbishment interventions of existing r.c. bridges require a very complex and comprehensive approach, deeply involving the use of conventional and unconventional structural analyses and technologies. The assessment of the current and the prognosis of future structural performances, first of all, requires an appropriate campaign of experimental and theoretical investigations. In addition to the more commonly addressed issues of strength and stability, consideration might also be given to a broad range of factors including stiffness, dynamic behavior, long-term deformations, durability and possibly even aspects. This requires the use of such tool as long term monitoring and dynamic identification, which can, when appropriately applied, give substantial informations on the overall structural behaviour of the bridge, allowing, via model updating procedures, for appropriately selecting the most efficient intervention strategies and for controlling the efficiency of the applied interventions.

The subsequent choice of the proper intervention (in terms of both material and application technique) is then dependent on several factors, the main ones being: the type of structural or non structural components (e.g. deck, piers, abutments, bearings, joints, water-proofing systems, etc.), the structural solutions (e.g. simply supported, continuous deck, arch bridges, etc.) and the type of action (static, cyclic, dynamic).

The paper describes the practical application of basic criteria for rehabilitation and refurbishment to three cast in-situ r.c. bridges (fig. 1) crossing the river Adige, all part of the massive works of post-war rebuilding in northern Italy. Their structural typologies make their rehabilitation and refurbishment interventions suitable for considerations of a general nature. In these cases, the decision to intervene was well justified by the numerous aspects of limited efficiency and performance of the bridges, with respect to current bridges codes.

Figure 1. View of the Sega bridge: (a) prior to the upgrading; (b) after the upgrading. View of the Albaredo bridge: (c) prior to the upgrading; (d) after the upgrading. View of the Zevio bridge: (e) prior to the upgrading; (f) after the upgrading.

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Effective framework for seismic analysis of cable-stayed-bridges, Part 1: Modeling of the structure and of the seismic action

L. Sgambi & F. Bontempi
Department of Structural and Geotechnical Engineering, Rome, Italy

G. Santoboni
Structural Engineer, Rome, Italy

ABSTRACT: The spatial variability of the earthquake strong ground motion (SVEGM) leads to significant aspects in the structural response of long-plan structures, such as long-span and medium-span bridges.

The main purpose of this study is to investigate the effects of the asynchronism of the ground motion on the structural response of a medium-span cable-stayed bridge, and in doing so the SVEGM takes only into account the time delay between the arrival of the shear waves, without taking into account their incoherency.

The studied structure is the bridge over the Cuiabà River; it is a prestressed composite cablestayed bridge, built up from 2000 to 2001 in Mato Grosso, Brazil, with three spans of 71.75 m, 157 m, 71.75 m, for a total length of about 300 m. The piers have an height of 51.8 m over the foundation plinths.

The seismic input has been assigned by the application of different spostograms, obtained by the double integration of 10 accelerograms, spectra-compatible with the ones proposed by the Eurocode 8 for an "A-type" soil; the accelerograms have been generated with the Simqke software (Gasparini, Vanmarcke, Liu, 1976) and are opportunely filtered by the use of the Butterworth filter, applied with the Seismosignal software (Seismosoft, 2004).

It has been considered different velocities of the seismic waves *Vs* in the soil. The applied spostograms have been translated in function of the time delay; the asynchronism is taken into account only in the longitudinal direction.

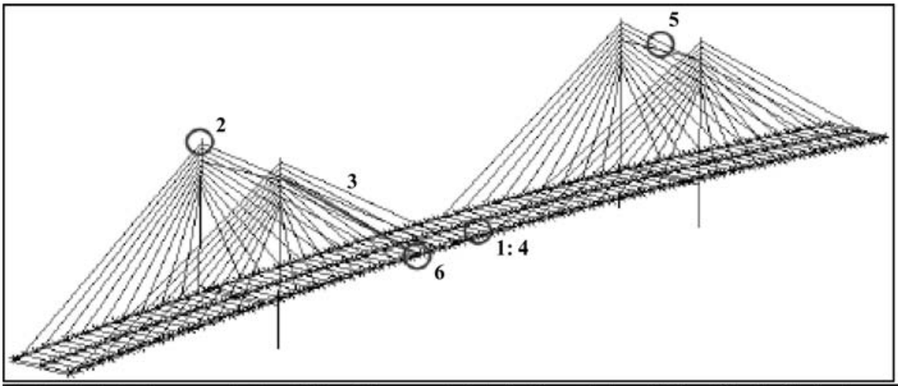
The analysis, conducted with the finite element program ANSYS, is a nonlinear time stepping analysis, done with the Newmark's method, with a Newton-Rhapson approach.

After having compared the results obtained, under a same seismic event, in the cases of different time steps, one has chosen to use a time step of 0.05 s.

Although the studies are principally conducted by a 3-D model of the structure, which one will call "frame model", developed using the finite element software Ansys, the structure has been studied using another model, which one will call "shell model", in order to evaluate how the differences in the modeling lead to different results. The substantial difference between these two models is how the bridge's slab is modeled: a gridwork in the frame model and a 2-D continuous in the shell model. The geometric characteristics (area and inertia) of the gridwork have been evaluated by the application of the kinematics criterion: applying forces equipollent to the ones applied on the continuous, the corresponding deformations should be equal (Toniolo, Malerba, 1981).

The comparison of the results obtained in both the models shows the similar behaviour of the two models, confirmed by the small differences in percentage of the values reached by monitored static and kinematical.

Figure 1 shows an image of the main 3-D used model and a list of the monitored parameters.



1. Vertical displacement in the middle of the span	4. Axial force in the middle of the logitudinal beam
2. Transversal displacement of the pier	5. Axial force in the upper transverse
3. Axial Stress of the 7^{th} cable	6. Moment at the connection of the 7th cable

Figure 1. 3-D model of the bridge and monitored parameters.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Effective framework for seismic analysis of cable-stayed-bridges, Part 2: Analysis' results

L. Sgambi & F. Bontempi
Department of Structural and Geotechnical Engineering, Rome, Italy

G. Santoboni
Structural Engineer, Rome, Italy

ABSTRACT: The spatial variability of the earthquake strong ground motion (SVEGM) leads to significant aspects in the structural response of long-plan structures, such as long-span and medium-span bridges.

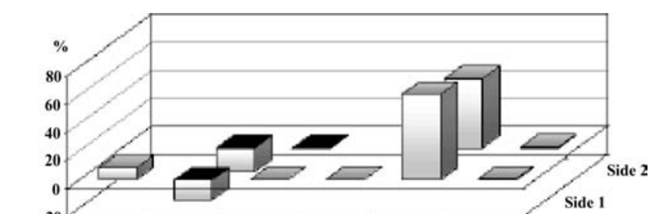
The main purpose of this article is to investigate the effects of the asynchronism of the ground motion on the structural response of a medium-span cable-stayed bridge, and in doing so the SVEGM takes only into account the time delay between the arrival of the shear waves, without taking into account their incoherency.

Referring to the article named "Effective framework for seismic analysis of cable-stayed bridges, pt.1: modeling of the structure and of the seismic action" (Bontempi, Sgambi, Santoboni, 2006), in which the studied structure and its modeling are discussed, once having established a list of the monitored parameters of the bridge, it has been done the evaluation of their values in a statistic way, in order to evaluate if the number of the considered seismic events may be enough to obtain reliable results.

One has considered the estimation of the average, variance and standard deviation of the results of both the synchronous and asynchronous motion. Histograms have been evaluated, showing, for established intervals, the number of the registered events; the gaussian distribution has been drawn, considering the obtained values as continuous variables.

The statistic reliability of the obtained results has been evaluated by the comparison of the shape of the gaussian distribution with the histograms, by the use of the parametrical χ^2 Pearson's test, and doing some physical evaluations.

An evaluation has been made of the differences in percentage of the averages of the maximum values reached by each monitored parameter, during the time histories, in both the cases of synchronous and asynchronous motion. Figure 1 shows these differences.



	Us middle	Uy pies	Cable 7	N upp t.	N middle	My
□Side 1	8,19	-14,46	0,20	0,23	60,54	0,75
□Side 2		-16,66	-0,03		49,60	0,60

Figure 1. Differences in percentage between the averages of the maximum values of the parameters in both the cases of synchronous and asynchronous motion.

Since the time delay depends on the shear wave velocity, the response of the structure under different values of the velocity has been studied, with the purpose to establish the dependence between the wave velocity and the structural response and to evaluate the possible existence of a critical value, which may show a particular behavior of the structural response. With those purposes, the response of the structure has been evaluated under a range of velocity which goes from 200 m/s to 1000 m/s, even if the lower values are not compatible with a "Eurocode type A" soil, used for the evaluation of the response spectra.

The main results of these studies are:

1. Neglecting the effects of asynchronous ground motion can lead to serious mistakes also into the evaluation of the structural response of medium-span bridges. Considerable differences in the observed parameters are registered only in the axial force in the middle of the longitudinal beam, with an underestimation of 60%;
2. Considering the structural response under different values of the shear wave velocity, from 200 m/s up to 1000 m/s, shows that the differences at the lower velocities are really high; the trend of the values reached by the monitored parameters shows a nonlinear behavior of the structural response;
3. The statistical evaluation of the results shows that the reliability of the results can be improved by the use of a higher number of considered events.

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*Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Reliability-based life cycle assessment for civil engineering structures

R. Schnetgöke, C. Klinzmann & D. Hosser
Institute of Building Materials, Concrete Construction and Fire Protection, Braunschweig University of Technology, Braunschweig, Germany

ABSTRACT: A modern lifetime oriented design of structures includes inspection and monitoring strategies. Structural health monitoring guarantees that the load bearing capacity, the serviceability and the durability of the structure remains ensured. One main focus of the collaborative research center (CRC) "Life cycle assessment of structures via innovative monitoring", funded by the DFG at Braunschweig University of Technology, is to optimize methods of structural health monitoring. In this article the framework for reliability-based system assessment developed by project field A1 of the CRC is described. The framework can be used to identify critical weak points or failure paths of a structure. It bases on methods of system and reliability theory. The gained knowledge about the structural system is used to formulate a probabilistic model with a fault tree, limit-state equations and information about the random variables for the structure. Using the results of the reliability analysis, further decisions concerning an inspection and monitoring design can be made.

For the illustration of the framework a bridge example is used. The main focus in the article lies on the ultimate limit states for the flexure and fatigue failure of the bridge structure. The structure is a multi-span plate-girder bridge over two fields with two girders and a span width of 25 m for each field. The bridge is pre-stressed with post-tensioning tendons. The assessment starts with a survey of possible weak points. First information about weak points of structures can be extracted from analyses of past damages. These analyses are the basis to determine failure scenarios. Further information about possible weak points and failure scenarios can be derived from the structural design process. In the example, the weak point analysis is based on the structural design.

The structural design for the bridge structure shows high action-effects in the cross sections at midspan and at the support. After a failure of the cross section in one of these areas, the structure does not yet fail. A plastic hinge arises and a redistribution of the applied load is possible. This means that the collapse of the structure depends on the failure of more than one component. All failure paths of a structure will be identified on the basis of an event-tree analysis which is an element of the framework. In the event tree, which is shown in the article, the failure in point 1 represents a cross-sectional failure at midspan of the first bridge field. The failure in point 2 represents a cross-sectional failure at the support and the failure in point 3 represents a cross-sectional failure in the second bridge field. A system collapse would occur after a formation of two plastic hinges.

For the reliability analysis the event tree will be transformed into a fault tree. Therefore, a schematization for the design of the fault trees was developed (Klinzmann et al. 2005). In the first step, a relation between the structure and the abstract elements in the fault tree is established. This is done with the so-called "failure points". A failure point of a structure can be any weak point of the structure where a failure can occur, e.g. the points 1–3 in the bridge example. The type of failure occurring at a failure point is called "failure mode". The failure mode combines the different causes for a failure. In point 1–3 a failure can occur due to flexure failure or due to fatigue failure. A failure path normally consists of more than one failure mode, which needs to occur for the system to collapse. All failure modes in a failure path are linked in parallel systems. Per definition, the failures have to occur at different failure points. All failure mechanisms of a structure are linked, using a serial system, because they are all single reasons for system failure. In case that a failure

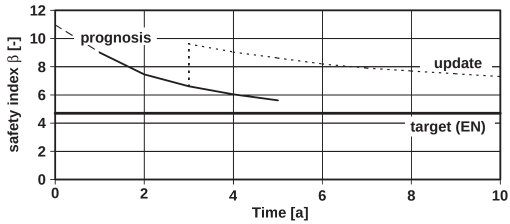

Figure 1. Results of the reliability analysis.

mode already would lead to a failure of the system, this failure mode is as well a failure mechanism. The limit state functions are modeled in the next step of the setup of the probabilistic model.

The limit states in the example are formulated using analytical models. The flexure capacity of the cross section can be calculated from the equation of the equilibrium of internal forces. Apart from the flexure capacity, also the limit state for fatigue failure of the tendons is to be analyzed. The describing model for fatigue failure is based on the Palmgren-Miner-hypothesis.

The first reliability analysis contains a prognosis based on the initial stochastic model and an assumed deterioration. The prognosis of the reliability is carried out for the next five years (Fig. 1). A further result of the reliability analysis is the influence of components on the system reliability, the so-called sensitivity of components. In this bridge structure the fatigue failure at midspan of both fields have a predominant influence on the system reliability. Based on these results a continuous monitoring of the action-effects due to loading is reasonable. The adaptation of the first prognosis on the current state of the bridge should be carried out after three years because there is a sufficient safety margin between the prognosis and the target reliability of structural codes (e.g. $\beta = 4.7$ from Eurocode). For this purpose a monitoring of the pre-stressed tendons is recommended. The dotted line in Figure 1 shows a possible development of the reliability with consideration of the monitoring results. This procedure is part of the developed structural evaluation and assessment process (Schnetgöke et al. 2005).

The article provides an overview of the framework of reliability-based life cycle assessment, which was developed to optimize the structural health monitoring process. Further, the framework gives support to civil engineers when they have to plan monitoring measures. Further research will focus on the modeling of systems and the extension of the usage of results from the monitoring in the assessment process of structures. Especially the consideration of deterioration models within the probabilistic model is of great significance.

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Probabilistic durability of concrete bridge structures in Korea

J.S. Kim
Department of Civil Engineering, Seokyeong University, Seoul, Korea

S.H. Jung, J.H. Kim & K.M. Lee
Department of Civil Engineering, Sungkyunkwan University, Suwon, Korea

S.H. Bae
Department of Civil Engineering, Andong University, Andong, Korea

Y.G. Kim
Byucksan Engineering CO. Ltd., Seoul, Korea

ABSTRACT: Traditionally, the durability problems in concrete structures are treated by implicit provisions, such as minimum concrete cover, maximum water/cement ratio, and minimum cement contents, etc. However, it is not possible to give a controlled durability and long-term performance of concrete structures with such deem-to-satisfy rules. Recently, research works have shown that the probabilistic approach, which is based on the theory of structural reliability, would be very valuable for durability analysis. The sampling data related with concrete bridges in Korea were selected from detailed field investigation and the probabilistic durability analysis based on a Monte Carlo Simulation was performed. The probabilistic properties of some design variables, such as diffusion coefficients of concrete and surface chloride concentration were newly determined using some experimental data in this country. The relative contribution of each design variable to the service life of concrete bridge structure was investigated through sensitivity analysis. By applying a probabilistic durability analysis to an integral structural design, the durability performance of concrete bridge structures would be remarkably improved.

1 INTRODUCTION

In Korea, a new bridge code needs explicit presentation of performance not only for ultimate limit states but also serviceability limit states such as durability to give an explicit relationship between performance and service life. Especially in the 1990s, there was increasing interest in setting requirements for the service life of concrete structures. The total costs comprised of both construction and maintenance is also increasingly emphasized. As a result, durability and service life aspects have been stressed in contract briefs. New methods for more accurate durability design of structures have been demanded. Fortunately, the vast research on concrete durability has produced reliable information on deterioration process (Duracrete, 2000; Takewaka, 1988), which makes it possible to incorporate durability even in the mechanical design of concrete structures. But they have not been fully utilized in the existing 'deem-to-satisfy' rules.

In this paper, a new method based on probability theory is established to overcome the above problems in durability design of concrete structures. It can present the explicit relation between service life and durability and include the uncertainties of the variables in the relation. With this, the service life of concrete structures in respect to the durability can be predicted with a probability also.

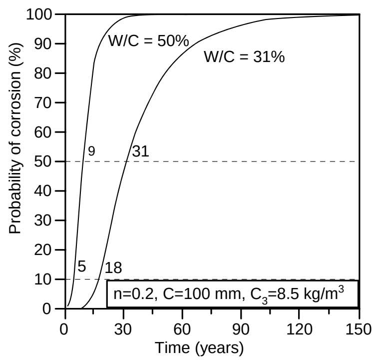

Figure 1. Probability of corrosion vs. time with concrete bridges at the west coast in Korea.

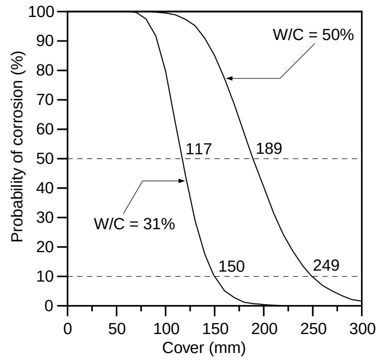

Figure 2. Calculated concrete cover with concrete bridges at the west coast in Korea at 50 years.

2 SERVICE LIFE OF CONCRETE BRIDGE STRUCTURE IN KOREA

The probability of corrosion with varying time and cover depth are given in Figure 1 and Figure 2 respectively. It can be seen that the probability increase for higher w/c ratio is much greater than that of lower one as expected and that the required cover depths are also much greater for higher w/c ratio concrete.

The times to reach 50% probability of corrosion are 9 years and 31 years for w/c ratio of 50% and 31% respectively. Since the corrosion time with 50% probability coincides with the deterministic prediction methods of service life, it appears that the service life of 31% w/c ratio may be estimated as 31 years if the probabilistic methods are not adopted. But by the probabilistic methods the corrosion time of concrete structure with 10% probability, which is the usual probability of failure for serviceability limit state, results in 18 years, respectively.

Figure 2 shows that the required covers of the 50 years service life based on 50% corrosion probability are 117, 189 mm for 31%, 50% w/c ratio respectively. The 10% of corrosion probability requires 150, 249 mm cover for 31%, 50% w/c ratio respectively.

3 CONCLUSION

In this paper, the development of a new procedure for probability-based durability analysis in chloride containing environment is established and applied for real conditions in Korea. It is observed in the sensitivity analysis that the probability of corrosion is much affected from the mean value of both chloride diffusion coefficient and surface chloride concentration, but less from the COV's of those. From the probabilistic analysis using chloride penetration data from some experiments, the corrosion time of 50% probability is 9 years and 31 years for w/c ratio of 50% and 31% respectively. But, the corrosion time of 10% probability is reduced by a half comparing that of 50% probability. Therefore it may be concluded that the probabilistic procedure for durability analysis of concrete structures in chloride containing environments is useful and valuable in order to insure the reliability of service life.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Performance analysis of a bridge – degradation, assessment and reliability modeling

A. Strauss & K. Bergmeister
Institute for Structural Engineering, Department of Civil Engineering + Natural Hazards, University of Natural Resources and Applied Life Sciences, Vienna, Austria

R. Pukl
Červenka Consulting, Prague, Czech Republic

D. Novák
Faculty of Civil Engineering, Brno University of Technology, Czech Republic

ABSTRACT: A complex approach to life cycle reliability assessment of concrete bridges is presented. It consists of three basic parts: nonlinear modeling of concrete, reliability calculation, and degradation aspects. The software system SARA – Structural Analysis and Reliability Assessment – integrates the nonlinear finite element software ATENA with stochastic and reliability program FREET into an advanced engineering tool which can be used for reliability assessment of structures based on an advanced nonlinear computer simulation. Degradation phenomena can be examined in an extended tool of FREET. The methodologies are applied to an existing bridge structure of the motorway highway A22.

1 GENERAL

Life cycle reliability assessment is needed by administration of transport infrastructure network operators. The decision-making tools should intercept the degradation and retrofitting process in order to support the maintenance of engineering structures.

Numerous approaches like Bayesian Updating Monte Carlo Simulation and Asset Management using multinational Genetic Algorithm have been used for Life Cycle Cost analysis. Since the stochastic models behind these procedures are rather demanding and time consuming, these approaches are commonly based on simplified mechanical models or formulas. More realistic reliability analysis can be achieved using nonlinear FEM modeling. The life-cycle analysis is a complex task and requires an interdisciplinary approach. It should combine modeling of

- nonlinearities in material
- uncertainties and
- degradation phenomena.

The particular methodologies for the use of probabilistic-based assessment are available and have been proven to work in practice.

The reliability calculation of structures from the stochastically obtained structural resistance and expected load distribution is a transparent and easily understandable concept. The stochastic response requires repeated analyses of the structure with random input parameters to reflect randomness and uncertainties in the input values. A nonlinear computer simulation should be utilized for realistic prediction of structural response and its resistance. As the nonlinear structural analysis is computationally very demanding, a suitable technique of statistical sampling should be used to allow for a relatively small number of simulations. Special attention should be paid to the modeling of degradation phenomena, like carbonation of concrete, corrosion of reinforcement, chloride attack, etc.

The main expected results are an estimate of structural reliability using a reliability index and/or theoretical failure probability during the degradation/retrofitting processes.

In order to perform a complete life-cycle analysis, a wide spectrum of methods should be used and combined, including nonlinear FEM modeling, statistical and reliability techniques and degradation phenomena modeling. The problem is rather complex; it requires an interdisciplinary approach and should be complemented by a health monitoring system. The approach presented in this contribution differs from the previously mentioned approaches mainly by the use of nonlinear models describing the real structure. The goal is a more realistic modeling of the structural behavior, and consequently of the health index (e.g. efficient realistic nonlinear modeling of structures). The method permits a direct link between nonlinear degradation models and nonlinear material behavior at the "mesoscale", see Fig. 1.

2 STRUCTURAL ANALYSIS WITH INTEGRATED MONITORING AT A REAL OBJECT

On a cantilever beam bridge in Italy with a total length of 1,000 m this method has been performed to evaluate the ultimate limit state, the service limit state and the sensitivity parameters regarding the structural responses at defined monitoring points. The bridge was built 1969 and is a fully post-tensioned box-girder. The measurements of selected values – monitoring points placed on the structure – allow for verification of the service ability level and the ultimate limit level of the structure. The monitoring points have been located along the structure in agreement with sensors already mounted on the structure. During the simulation process, for each incremental increase of line load, the monitoring points recorded the structural response – mainly the longitudinal strain or the vertical deflection.

The data achieved by the monitoring, serves mainly for the customization of the input quantities of the nonlinear simulation.

3 DEGRADATION CONSIDERATIONS

The probabilistic simulations also serve as a base for degradation considerations. There are some models for describing degradation processes, linear models considering environmental conditions, linear models combined with conditioned information from inspection routines, inspection routines based degradation processes combined with weibul functions etc.

Some of these models are applied on the Colle Isarco Viaduct. These considerations were also the base for strengthening actions.

4 CONCLUSIONS

The method introduced shows the feasibility of nonlinear probabilistic calculations. Creating ingenious simulation processes the simulation number can be reduced and the correlations between the basic variables can be taken into account. Therefore it is possible to use sophisticated nonlinear material models and highly developed nonlinear simulation techniques within acceptable time expenditure at complex structures like the Colle Isarco bridge. The introduced method permitted to support in an efficient engineering way lifetime considerations and preservation measure planning. Selective inspection data, permanent monitoring, Santa (2004), and degradation models can also be included in the process. The concept is that powerful that inverse material detection algorithm to deviate time dependent material conditions from monitored deflection lines are too supported.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Uncertainties in probabilistic modeling of the load carrying capacity of bridges

A. Stenlund
Swedish Road Administration; Luleå University of Technology, Luleå, Sweden

L. Elfgren
Luleå University of Technology, Luleå, Sweden

E. Rosell
Swedish Road Administration, Borlänge, Sweden

ABSTRACT: Uncertainties in probabilistic modeling of bridges are exemplified with the bending moment capacity of the edge beam of a concrete bridge deck. First Order Reliability Method (FORM), Failure Mode Effect Analysis (FMEA) and Event Tree Analysis (ETA) are used.

1 INTRODUCTION

There is a great economic potential in finding useful methods for the assessment of the structural capacity of existing bridges. If a bridge has been damaged or if the load carrying capacity is insufficient, strengthening or replacements are options that have to be looked into. An analysis often starts with a calculation based on deterministic methods. If the required traffic load capacity cannot be met, then the use more sophisticated methods is an option. The use of probabilistic methods is one such possibility. The object of this paper is to evaluate and exemplify the uncertainties and risks with these kinds of methods. As an example, the bending moment capacity has been studied for the edge beam of a concrete deck resting on steel beams in a four span bridge situated in Piteå in Northern Sweden. The uncertainties and risks are evaluated with a First Order Reliability Method (FORM), a Failure Mode Effect Analysis (FMEA) and an Event Tree Analysis (ETA).

2 CASE STUDY – BRIDGE OVER PITEÅ RIVER IN NORTHERN SWEDEN

The E4 Bridge over Piteå River is a continuous steel beam bridge with a concrete deck, see Figure 1. It has four spans (52 + 70 + 70 + 52 m) and total length of 257 m. The width is 13 m. There is no structural interaction between the bridge deck and steel beams. Some results from a study of the moment capacity of the edge beam are given in Table 1 and Figure 2.

Figure 1. Elevation of the E4 Bridge over Piteå River in Northern Sweden.

Table 1. Failure Mode Effect Analysis. Effect of variation of variables. Risks.

Variable	Parameter, y	Tested parameter variation, (Δy)	Calculated β-index variation, $(\Delta\beta)$	Sensitivity in β-value for change in y, $\Delta\beta/\Delta y$	Corresponding probability, $p_f(\mathrm{G} < 0)$	Estimated probability of incorrect value of parameter $p_f(y \text{ wrong})$	Risk with parameter
A_s	μ	382–405 mm^2	4.45–4.96	2.22%/mm^2	$\approx 10^{-6}$	10^{-5}–10^{-6}	Minor
A_s	σ	6.0–12.0 mm^2	4.93–4.80	2.17%/mm^2	$\approx 10^{-6}$	10^{-4}–10^{-5}	Minor
f_{st}	μ	455–475 MPa	4.54–5.01	2.65%/MPa	$\approx 10^{-5}$	10^{-5}–10^{-6}	Minor
f_{st}	σ	25–45 MPa	5.30–3.84	2.70%/MPa	$\approx 10^{-5}$	10^{-4}–10^{-5}	Minor
f_{cc}	μ	45–65 MPa	4.86–4.90	0.20%/MPa	$\approx 10^{-6}$	10^{-5}–10^{-6}	Minor
f_{cc}	σ	4.5–7.5 MPa	4.89–4.89	0%/MPa	$\approx 10^{-6}$	10^{-4}–10^{-5}	Minor
d	μ	385–400 mm	4.62–4.96	2.27%/mm	$\approx 10^{-5}$	10^{-5}–10^{-6}	Minor
d	σ	7–27 mm	4.96–4.16	4.00%/mm	$\approx 10^{-5}$	10^{-4}–10^{-5}	Minor
C_R	μ	1.1–1.3	5.03–4.64	139%/0.2	$\approx 10^{-4}$	10^{-5}–10^{-6}	Minor
C_R	σ	0.10–0.20	5.22–3.62	160%/0.10	$\approx 10^{-4}$	10^{-5}–10^{-6}	Minor
M_{tr}	μ	35–51 kNm	5.99–3.50	12.6%/kNm	$\approx 10^{-4}$	10^{-2}–10^{-3}	Major
M_{tr}	σ	2.5–7 kNm	5.77–3.70	18.0%/kNm	$\approx 10^{-4}$	10^{-1}–10^{-2}	Major

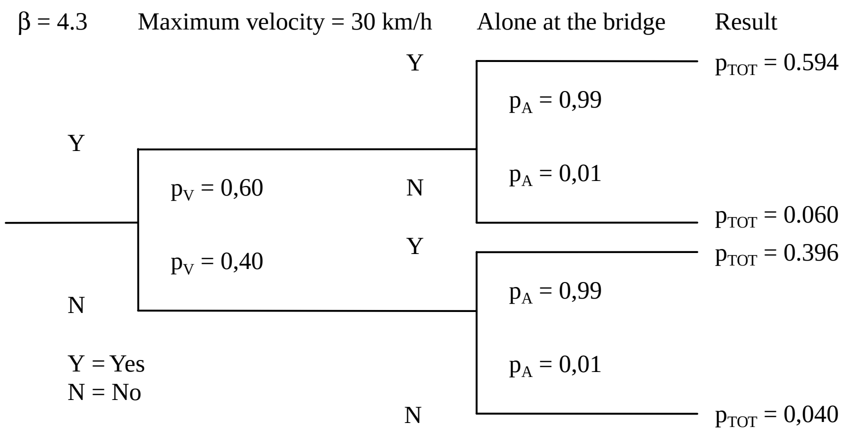

Figure 2. Event tree studying the influence of velocity and other traffic on the bridge.

3 CONCLUSIONS

It can be concluded that Failure Mode Effect Analysis, FMEA, and Event Tree Analysis, ETA are two methods that can be used as a complement to First Order Reliability Methods, FORM, for evaluating uncertainties and risks in probabilistic modeling of the load carrying capacity of bridges. However, it is difficult to estimate the probabilities to be used in FMEA and ETA and a FORM analysis may have to be performed first. In this paper the edge beam and its bending moment of capacity has been studied. The FORM and the FMEA show that the bending moment of traffic is the variable that has the highest risk or uncertainty. The ETA shows that one of the conditions for allowing a specific heavy load vehicle to pass the bridge, the velocity, has a high influence on the bending moment of traffic and of course on the safety index β. This strengthens the result of the FORM and FMEA analyses, and justifies the recommendation to strengthen the edge beam.

Probabilistic characterization and analysis of the properties of materials used in bridges

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Reliability based assessment of prestressed concrete bridges subject to creep using a coupling procedure

W. Raphael, F. Geara & F. Kaddah
Ecole Supérieure d'Ingénieurs de Beyrouth (ESIB), Université Saint-Joseph, Riad el Solh, Lebanon

A. Chateauneuf
LaMI/IFMA-UBP, Campus des Cézeaux, Aubière cedex, France

ABSTRACT: Creep and shrinkage strains of concrete can have prejudicial consequences in prestressed structures built by phases. But these uncontrolled strains appear, with values often clearly more important than the expected ones. In fact, there is not yet a physical explanation perfectly satisfactory of creep and the codified descriptions of this phenomenon are always unreliable.

In this work, a large collection database has been constituted for creep testing in many research centers in Europe. With the constitution of this database and the observation of real structures' behaviour, and also with the progress made in probabilist, statistical and reliability calculation, a critic of current codes of deferred deformations' calculation of concrete becomes possible. This database allows us to carry out statistical analysis in order to define the influence of different parameters, such as: concrete strength, prestress force, ambient humidity, scale effect, loading date. . .etc.

At first, a comparison is performed between the database experimental results and the design codes of practice: BPEL, CEB 78, CEB 90, EUROCODE 2 and ACI. For all these codes, the creep deformation is, more or less, underestimated. For long-term creep, the error reaches sometimes more than 300%. This result has been confirmed by reliability analysis which emphasizes the weakness of the usual design codes.

In France, a bridge built from 1989 to 1991 has a steel central span, supported on two prestressed concrete cantilevers. The vertical displacement measurements at the cantilever free ends have shown unexpected excessive deformations, reaching more than three times the predicted one; noting that BPEL has been used in the design of this structure. These observations confirm the fact that design codes lead to lower prediction of creep, in most cases.

In order to understand the possible reasons for the code deficiency, a reliability based assessment is carried out for this bridge. A coupling between the finite element tool ST1[1] and the reliability software PHIMECA[2] is applied to analyze the reliability of the bridge. It shows the role of creep parameter variability and hence to justify the gap between predicted and observed creep deformations. In this reliability assessment, the EUROCODE 2 gives the best results compared with other codes. However, the probability that prediction meets reality is still very low.

This study shows the interest of reliability considerations in the qualification of design codes and allows the designer to take account for creep model uncertainties. Actually, it seems that improved prediction has to be included in the present formulations of different codes.

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Probabilistic creep model by Bayesian updating for design codes

D. El Asmar Selouan & W. Raphael
Ecole Supérieure d'Ingénieurs de Beyrouth (ESIB) – Saint-Joseph University, Ain Saadé, Lebanon

A. Chateauneuf
LaMI – UBP & IFMA, Campus des Cézeaux, Aubière cedex, France

ABSTRACT: Creep plays an important role – sometimes crucial – in the in-service behavior of concrete structures, especially for long term integrity of prestressed concrete. Incorrect or inaccurate prediction of creep deformation leads to undesirable consequences for all involved parties, including designers, constructors, owners, users, insurers, financial institutions, etc. The informed knowledge of creep behavior is thus mandatory for *lifetime management* of concrete structures, which is a fundamental need for sensitive structures such as bridges. So, even when creep does not affect the ultimate strength of the component in which it takes place, its effect may be extremely serious as far as the whole structural performance is disturbed.

Using a large database that was established for creep testing in different European research centers, a comparison is made between the experimental results and the creep formulas in the Eurocode 2; we observed that creep deformation is often underestimated.

In order to improve the theoretical estimation of creep, a Bayesian approach was performed to improve the proposed formulas in the codes of practice. Using the collected database, and after introducing many corrective parameters in the design model, a new formulation was proposed. The statistical uncertainty caused by inadequate relationships was also considered by introducing a coefficient of correction in order to account for the model error. The likelihood function is then developed and applied for available data.

The software BUMP "Bayesian Updating of Model Parameters" (developed in the University of California) is used to apply the Bayesian updating to the experimental results taken from the collected database.

The analysis is then conducted in order to decide how to take into account the prior distribution for the unknown parameters. At first, and since we have lack of previous knowledge, we have considered a non informative prior distribution. The coefficient of correction A is calculated for different ranges of compressive strength of concrete f_{cm}.

The different values of the coefficient of correction vary between 1.147 and 1.415 for values of compressive strength varying between 62 MPa and 18 MPa respectively.

In order to study the influence of the number of experimental tests used in each calculation, we have decided to increase the number of experiments used from 7 to 24 tests. Here we notice that in our case of a non informative prior distribution for θ, the increase in the number of tests gives almost the same results as in the previous part of the study (smaller number of experimental tests). This leads to the conclusion that in our case, the number of considered tests doesn't have a big influence on the obtained results.

Then, the influence of the choice of the prior distribution on the coefficient of correction A has been studied. That's why an informative prior distribution is considered here. In our case, since we have lack of previous knowledge, we have considered at the beginning a normal distribution and then a uniform one. Many calculations were performed using different cases for the mean values and standard deviation regarding the normal distributions. Also, many calculations were performed using different intervals for the uniform distributions.

All different coefficients of correction are collected and a comparison is finally conducted in order to see the influence of the choice of the prior distribution on the obtained results. As a conclusion, we could say that the choice of a prior distribution for the unknown parameters doesn't have a big influence on the obtained results.

Using the experimental database, this study demonstrates the importance of the Bayesian model assessment. It also shows that the incorporation of model uncertainty and measurement errors has a strong influence on statistical information to be used in the reliability context. The adoption of such a design approach would improve the long-term serviceability of structures subjected to time-dependent strains.

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Failure analysis of FRP-strengthened concrete beams

M. Ali-Ahmad, K. Subramaniam & M. Ghosn
The City College of New York/CUNY, USA

ABSTRACT: Several previous studies have demonstrated the merits of using Carbon Reinforced Plastic (FRP) composites for strengthening existing reinforced and prestressed concrete bridges. These studies have also shown that the success of this strengthening technique is highly dependent on the effectiveness of the bond between the concrete and the FRP. Yet, current available or proposed design specifications do not provide standard experimental methods to characterize the properties of the bond nor do they provide simple and accurate methods to analyze the bond's effectiveness when FRP sheets are attached to beams subjected to transverse and bending loads. In this paper, the authors propose an experimental set up to perform a laboratory investigation of the behavior of the interfacial bond between concrete and FRP sheets. This procedure leads to the characterization of the nonlinear behavior of the bond through the development of a material law for the FRP-concrete interface appropriate for the corresponding adherents material properties. The nonlinear material law of the FRP-concrete interface, which considers the shear-debonding mode of failure, can then be incorporated into a finite element numerical model for predicting the load response of FRP-strengthened beams while accounting for the initiation of debonding and the subsequent propagation of the bond's fracture along the interface between the FRP and the concrete substrate as well as other possible failure modes such as concrete crushing and FRP rupture. The validity of the numerical model is verified by comparisons to experimental results of externally strengthened reinforced concrete structures.

1 Test setup

The experimental procedure consists of a direct shear test that applies a pushing force against a concrete block while pulling on and attached FRP sheet. The set up is meant to simulate a mode II shearing crack failure in the bond between the concrete and the FRP sheet. During the test, the strains at the surface of the FRP composite and the concrete are obtained using the digital image correlation (DIC) technique that provides spatially continuous measurements of the strain field within an accuracy of 50 $\mu\varepsilon$.

2 Determination of the bond's material law

The measured FRP strains are used to calculate the interfacial shear stress at any location using the theory of fracture mechanics from the following equation:

$$\tau(y) = t\,E\,\frac{d\varepsilon(y)}{dy} \tag{1}$$

where $\tau(y)$ is the shear stress distribution along the interface of FRP and concrete at any location y; E is the elastic modulus of the FRP composite; t is the thickness of the FRP composite; and $\varepsilon(y)$ is the axial strain distribution in the FRP. Equation 1 indicates that the magnitude of the shear stress depends on the gradient of the axial strain in the FRP composite and not on the strain as implied in current specifications.

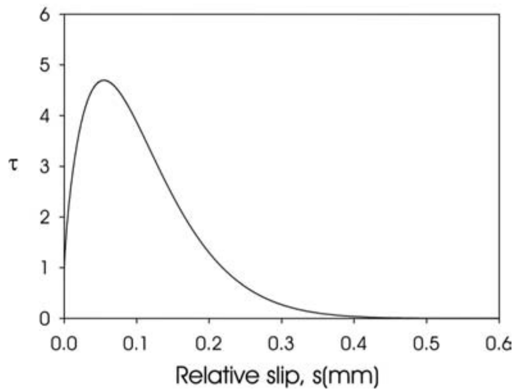

Figure 1. Typical material law for bond at concrete/FRP interface.

The relative slip between the FRP and the concrete along the length of the FRP sheet, s(y), at a given location, y, can be obtained by integrating the axial strain in the FRP up to that point. The material law for the FRP/concrete interface can be obtained by comparing the shear stress and relative slip at different locations. The interfacial nonlinear material law obtained in this manner will have the shape of the curve shown in Figure 1.

3 Finite Element analysis of a strengthened concrete beam

A finite element analysis of an experimentally tested plain concrete beam externally strengthened using bonded FRP sheets is performed to illustrate how the material law developed from the direct shear test can be applied to predict the load response of strengthened beams. In this case, the concrete is modeled as a nonlinear 3-D solid element, while the FRP is modeled by tension elements. The bond in the interface is modeled by two-noded-three-dimensional nonlinear spring elements with spring constants derived from the material law accounting for the tributary area of the bond represented by each spring. A comparison of the response of a beam thus modeled to the experimentally recorded load response showed good agreement confirming the validity of the proposed methodology for obtaining the material law of the bond and its application in finite element analysis of externally reinforced concrete structures.

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Designing with HSC for safety: Effect of age specification for characteristic strengths

S.M.C. Diniz
Federal University of Minas Gerais, Belo Horizonte, MG, Brazil

The need for stronger, stiffer, more durable and cost effective materials for bridges have led to an increasing interest in high-strength concrete (HSC) in the last decades. The use of HSC, in its turn, has introduced a number of changes: (*i*) due to economic considerations, the traditional 28-day specified compressive strength has been replaced by strengths specified at later ages such as 56 or 90 days; (*ii*) production of HSC depends on low water/cementitious ratios, stronger aggregates, higher cement contents, chemical and mineral admixtures, and strict quality control; (*iii*) testing procedures have demanded stiffer machines and smaller test specimens; and (*iv*) design codes have been revised to accommodate the mechanical properties of this material and the resulting structural behavior. While most of the aforementioned issues have been thoroughly scrutinized, the use of specified compressive strengths for design based on 56 or 90 days has not yet received enough consideration.

In reinforced concrete (RC) design, meaningful comparisons of reliability levels are possible if, and only if, the unaccounted normalized strength gains from 28 days up to the maximum strength remain the same. Experimental results indicate a higher rate of strength gain for higher strength concrete at early ages; however, beyond 28 days the differences are negligible (Carrasquillo et al. 1981). As a result, the requirement of similar normalized strength gains is violated by prescribing later age strengths for HSC.

From the aforementioned facts it can be concluded that meaningful comparisons of reliability levels implicit in a set of RC design recommendations are possible if, and only if, concrete compressive strengths are taken at the same reference age (28 days or later) in the resistance modeling. It can be expected that short, axially-loaded columns are the structural elements whose ultimate capacities will be mostly affected by changes in concrete compressive strengths. As such, the investigation reported herein addresses the effect of concrete age specification on the reliability of short, axially-loaded HSC columns. The reliability levels resulting from HSC columns, designed for 28-, 56-, and 90-day specified strengths, but evaluated at the same reference age (28 days), are computed. It should be observed that the choice of 28 days is in line with well established design procedures.

Eighty-four short, axially-loaded square tied columns were selected for analysis. All columns have been designed according to the ACI 318-02 code. The analysis was intended to reveal the influence of the concrete age specification on the column reliability with respect to the ultimate strength. Additional variables, – specified concrete compressive strength, amount of longitudinal steel, and sustained load ratio – have also been included in the investigation. Three specified concrete compressive strengths – 34.5 (normal-strength), 62 (medium-strength), and 96.5 MPa (high-strength) were chosen. For the 34.5 MPa concrete it is assumed that this is the specified compressive strength at 28 days. For the medium- and high-strength concrete, the specified strengths are assumed to correspond to 28, 56 or 90 days. Two longitudinal steel ratios (0.01 and 0.03) and six load ratios (0, 0.2, 0.4, 0.6, 0.8, and 1.0) have been selected. Grade 60 steel was used for the longitudinal bars. Longitudinal steel ratios close to 0.01 and 0.03 have been selected.

In this study, Eq. (1) is used as the deterministic model for the computation the axial resistance P of the column:

$$P = m\left\{k_{st}\, f_{c,28}\left[\left(b+b_d\right)\left(h+h_d\right)-A_{st}\right]+f_y\, A_{st}\right\} \tag{1}$$

where m is the model error, k_{sl} is the sustained load factor, $f_{c,28}$ is the equivalent 28-day in-situ concrete compressive strength,b and $h$are the cross-section dimensions, A_{st} is the area of steel, and f_y is the yield strength of the longitudinal steel. In this study, the factors 1/1.10 and 1/1.15 are used to compute equivalent 28-day compressive strengths from 56- and 90-day strengths, respectively (Ahmad 1994).

Reliability indexes β and probabilities of failure P_f, have been computed for the columns analyzed. The following expression defines the limit state used in the reliability analysis:

$$g(P, D, L) = P - D - L = 0 \tag{2}$$

where P is the axial resistance of the column; D and L are, respectively, the dead and live load acting on the column. In this study, a hybrid method, using the limit state of Eq. (2) and the column resistance given by Eq. (1), has been used in the reliability analysis.

The following observations can be made from the results obtained. Higher sustained load ratios have a deleterious effect on the concrete strength; on the other hand, more dead load implies in smaller load variability. As a consequence, column reliability depends on the concrete compressive strength (and age specification) and the longitudinal steel ratio. For the 34.5- and 62 MPa (at 28 days) columns, β increases as the sustained load ratio increases. In these cases, in spite of the deleterious effects of the sustained loads, the smaller variability of dead loads as compared to live loads is the dominant factor. For higher compressive strengths, later age specifications, and minimum longitudinal steel ratio, β increases as the sustained load ratio increases up to 0.6, decreases at 0.8, and then increases again for the case of dead load only. This shows that, as the sustained load ratio increases, the sustained load effects are more influential than the smaller variability of dead loads. It should be noted that the increase in β for the sustained load range 0.8–1.0, is due to the change in the dead load factor from 1.2 (for the combination dead plus live load) to 1.4 (for dead load only). This change corrects previously observed distortions (Diniz 1999) with the use of a single factor for the dead load.

All other conditions remaining similar the higher the f_c' the smaller the column reliability. Although HSC presents smaller variability, it also presents a relatively smaller in situ strength as embodied by the k_3 factor (ratio in-situ to standard cylinder strength). Moreover, for the same specified strength, column reliability decreases as the age of concrete strength specification increases. The lowest reliability levels are found for the combination of higher concrete compressive strengths, later ages specification, and minimum longitudinal steel ratios.

The results obtained have shown that the problem of later age specification for HSC columns does deserve attention. It should be noted that the validity of the results reported herein are highly dependent on the assumptions made pertaining to strength gains with age. It should also be emphasized that strength gains are highly dependent on actual curing conditions and admixtures used. As a result, for some mixes, the assumptions made may be very conservative. However, this work aimed in providing a general guideline, – as required in a design code to cover a wide range of mixes – rather than specific mixes. More research is needed for the assessment of strength gains with age for actual curing conditions and specific mix properties.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Designing and controlling concrete quality in the field for a 100-year life cycle

P.-C. Aïtcin
Professor Emeritus, Université de Sherbrooke, Canada

R. Morin
City Engineer, Ville de Montréal, Canada

ABSTRACT: The use of high performance concrete (HPC) having a W/C or W/B ratio between 0.30 and 0.40 increases the service life of concrete structures because HPC are more durable than concretes having a W/C or W/B greater than 0.50. But, in order to reach a 100-year life cycle with HPC it is absolutely necessary that these concretes be placed and cure appropriately to avoid the development of a too high autogenous shrinkage. Some HPC structures recently built have been severely cracked very rapidly because they wee built as if they were built with a normal strength concrete having a W/C or W/B ratio greater than 0.50. Ignoring the negative effect of the volumetric contruction of the hydrated cement paste when this construction occurs in the absence of an external source of water resulted in an early cracking. This early cracking is due to the tensile stresses developed in the menisci that are appearing within the cement paste due to chemical contraction.

Autogenous shrinkage develops in any concrete that does not beneficiate from an external source of water during hydration whatever its W/C or W/B ratio is. But for concretes having a W/C or W/B ratio greater than 0.50 it is negligible when compared to drying shrinkage. On the contrary, for concretes having a W/C or W/B ratio between 0.30 and 0.40 it cannot be anymore neglected because it can be as important as drying shrinkage. It is damaging because this shrinkage occurs at a time when the hydrated cement paste and the concrete have not developed a very high tensile strength.

But when any cement paste hydrates in the presence of an external source of water, it does not present any autogenous shrinkage since no menisci appear within the cement paste. According to Powers theory a cement paste having a W/C ratio of 0.36 does not present any porosity when all its cement particles have hydrated in the presence of an external source of water. Therefore, to build structures that will have a very compact microstructure and that will not present any fissures it is necessary to use a 0.36 W/C or W/B concrete and to cure it in the presence of water.

There are several ways to water cure concrete: fogging, watering, partial replacement of normal weight aggregates by saturated lightweight aggregates, superabsorbant polymers, or to reduce autogenous shrinkage use the use of shrinkage reducing admixtures. Some of these techniques have been implemented with success in the field so that it is now possible to build HPC structure almost free of any fissures. It is easy to predict that these structures will last more than 100-year. In order to illustrate the importance of detailed specifications to insure the construction of almost crack-free concrete structure, some of the requirements requested when HPC is used by the city of Montreal are outlined.

The W/B ratio of the HPC used is 0.36. The maximum temperature of the concrete at the jobsite is 20°C in summer and 25°C in spring or fall. All the horizontal surfaces are fogged for 24 hours as soon as there have been finished, then, they are watered for 24 hours and then covered with a geotextile that is maintained humid during 5 days. The formwork of vertical walls are released after 16 hours and the wall waver cured for 6 days. Walls are cast in alternance by section of 1.8 m. As presently lightweight aggregates are not available in Montreal a shrinkage reducing admixture is used to reduce autogenous shrinkage.

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Cruz, Frangopol & Neves (eds)
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Estimation of the *in-situ* concrete characteristics from building control results

J.-L. Clément
Laboratoire Central des Ponts et Chaussées, Paris, France

ABSTRACT: The main objective of our study is to estimate the elastic modulus in every concrete elements of a structure, from mechanical parameters obtained on cores, in order to be able to use realistic value in a structural calculation.

When concrete cores are taken out of a real structure, compressive strength and Young modulus are measured. We want to use a mathematical relation between these two values and we want to apply this relation to the control compressive strengths obtained at 28 days during the construction of the structure.

$$E_{cm} = 15.35\left(\frac{f_{cm}}{10}\right)^{0,3} \text{ for 28 days EC2 calculated values on concrete cores}$$

Our experience is based on the treatment of control strengths of real structures : we present here only one example. The main result is that these compressive strength depends on the temperature during the first day of casting : when the weather is hot, the compressive strength is lower. Concrete realised in winter is harder.

The dependency between strength and external temperature is illustrated on our example.

From these values, the correlation between strength and temperature is written as:

$$f_{cm}(T_{ext}) = -0.56\, T_{ext} + 52.63 \quad \text{with } T_{ext} \text{ in °C and } f_{cm} \text{ in MPa}$$

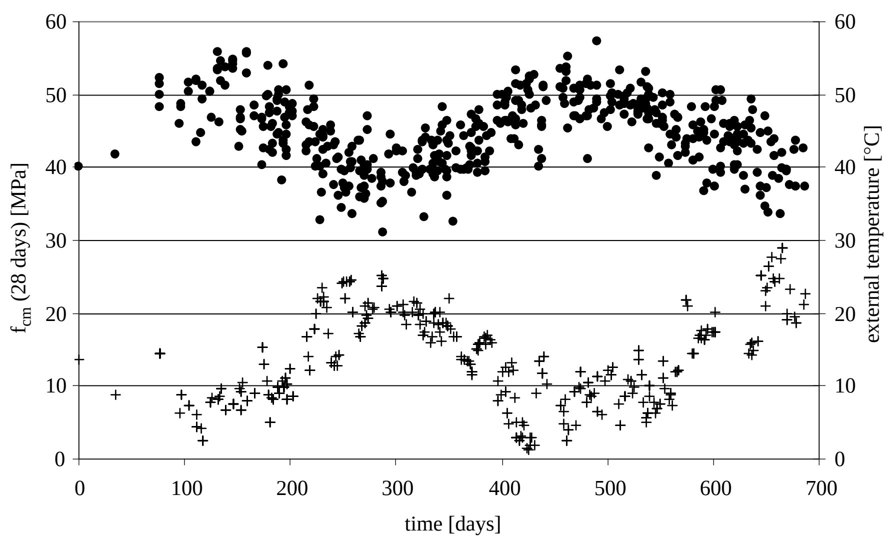

Figure 1. Control compressive strength at 28 days versus time.

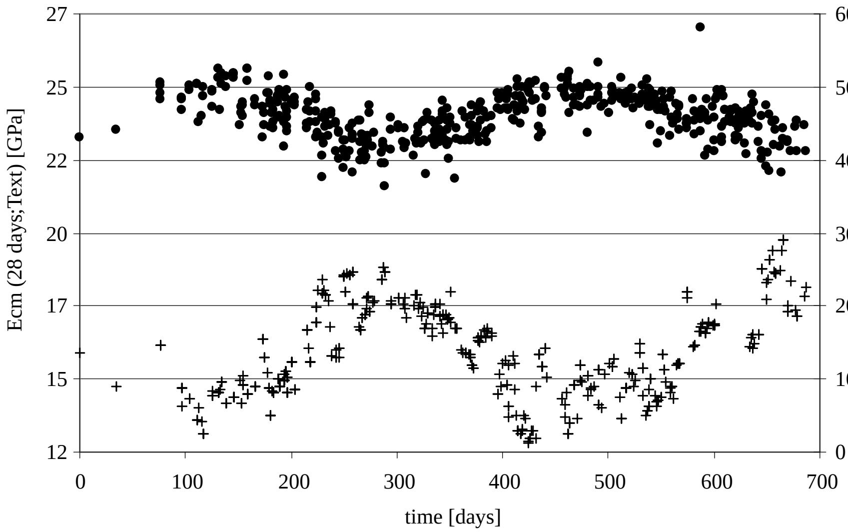

Figure 2. Estimation of the elastic modulus Ecm at 28 days.

The standard mean compressive strength is defined for a temperature of cure of 20°C. In our case, we obtain:

$$f_{cm}(20°C) = 41.4\ MPa$$

The previous relation between strength and Young modulus is then applied, in order to estimate the in situ Young modulus : mean value 23.5 GPa, standard deviation 0.8 GPa, coefficient of variation 3% (with effect of external temperature variations).

This method enables us to estimate the elastic modulus in every part of the structure, because it is easy to know when any part of the structure was built with what concrete. A re-calculation of the structure, especially the delayed deflection, can be performed using these new values of elastic modulus.

One of the limits of this approach is related to the differences between the cylinder strength and the concrete of massive elements. But it leads to a better estimation of Young modulus in the different elements of the structure, which can be used as data in order to re-calculate the whole structure.

This remark leads to propose some calculation, even during the design of the structure, using minimal, mean and maximal values of elastic modulus.

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Politics and perception in life-cycle decisions

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Selling life-cycle concepts within the political system

R.B. Corotis
Denver Business Challenge Professor, Department of Civil, Environmental and Architectural Engineering, University of Colorado, Boulder, CO, USA

ABSTRACT: The basis of all political systems is retention of power for those in office. This is true for democratic, representative, royal and autocratic systems. Public perception of service to the public at large, or at least those with influence, enhances this goal in the political system. Because people tend to value immediate returns, the concept of optimal design selection based on life cycle cost analysis is fundamentally at odds, therefore, with political systems. Instead, producing the most immediate benefit for the least cost is a natural inclination for those with political power when designing public infrastructure such as bridges, transportation and other networks. To some extent, the same is true in the private sector, where the customer desires the highest immediate return for investment. Recognizing this conflict and developing procedures to enhance the acceptability of life cycle costing will result in increased acceptance and use of this clearly optimal method for society.

In this paper several methods are suggested that will make life cycle costing more consistent with political rewards. These include public fora to educate people on the increased long-term value of life cycle costing, case studies of projects with high initial cost and low maintenance as well as those with lower initial cost and high maintenance, and a pubic infrastructure report that on a biennial basis informs a community of the capital, operating, maintenance and risk costs of all infrastructure. These approaches attempt to separate the capital cost budget from ongoing budgetary categories, and in so doing give each its deserved attention. Such a separation is common in private industry investment analysis.

By recognizing the goals and objectives of those making societal decisions, engineers will be able to tailor their presentations so as to be more successful in convincing political decision-makers to select designs that offer the most long-term benefit to a community, incorporating the issues of sustainability and risk.

One of the major issues is the incompatibility of structural infrastructure lifetimes and political cycles. Because of this incompatibility between long-term benefits of life-cycle analysis and immediate rewards consistent with political success, it is necessary to develop a mechanism by which "political capital" is more aligned with infrastructure lifetime value. Another issue is the disparity among funding sources and the recipients of the benefits of community infrastructure projects. A problem in real world application, especially for public infrastructure, is that the source of costs and recipient of benefits are not only different from each other, but various components within each differentially impact agencies and groups (Enarson and Morrow 1998). Some agencies have completely separate capital budgets, whereas others find it easier to increase their funding base in terms of taxation for new capital than for operation and maintenance of current facilities.

In order to address the political realities of these decisions, several steps are necessary on the part of those preparing issues for expenditure of public funds. One of these is to present case studies of past examples where consideration of long-term issues such as low probability, high consequence hazards and complete life-cycle maintenance and operation costs have made a significant impact on the selection of an alternative (Federal Highway Administration 2003; 2005; National Research Council 1991). Another is to involve the public in all stages of the decision process, including formulation of the basic options, evaluation of the alternatives and presentation to the larger community (*Understanding Risk: Informing Decisions in a Democratic Society* 1996).

One of the issues that is often neglected, but is implicit in infrastructure decisions, is that of uncertainty. The public in general does not deal well with consequences that differ from those that are expected. Even though the decision may have been the correct one in light of available information, the political reality for the decision-maker is that he or she will be held responsible for the actual outcome. In many cases it is possible to employ formal probabilistic analysis to convey the likelihood of a range of outcomes, although the acceptance of such ambiguity is still a political risk.

A final important issue in the various approaches to incorporating life-cycle decision analysis into the political process is the proper public accounting of the costs and benefits, and the conveyance of this information to the public. One of the key implicit issues is that of separating the capital cost budget from ongoing operational budgetary categories. Such a separation is common in private industry investment analysis. The credits and debits of existing and new infrastructure should reflect the current capital and operating values, including discounted future benefits and costs. Finally, there is an important additional aspect related to discounting. Psychologists note that there is a natural human preference to receive a benefit immediately rather than at some point in the future. This preference to receive benefits in the near term produces a discounting of future rewards, independently of investment and inflation (Corotis and Gransberg 2005).

The paper concludes that by recognizing the goals and objectives of those making societal decisions, engineers will be able to tailor their presentations so as to be more successful in convincing political decision-makers to select designs that offer the most long-term benefit to a community, incorporating the issues of sustainability and risk.

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User costs in life-cycle cost-benefit (LCCB) analysis of bridges

P. Thoft-Christensen
Aalborg University, Aalborg, Denmark

ABSTRACT: User costs are usually not included when optimal maintenance strategies and decisions are made, although authors often mention that user costs ought to be included. The life-cycle costs are minimized for the considered structure without considering the often significant costs for the users of the bridge and even without considering the long-term effects of the decision. Unfortunately, the maintenance decisions are often political decisions which are not easy to accept for the community. There is clearly a need to convince the decision-makers that user costs should be considered when major decisions are made.

Life-Cycle Cost (LCC) analysis is based only on the direct costs such as inspection and repair (preventive and essential). User costs are usually not included in an LCC analysis. Life-Cycle Cost-Benefit (LCCB) analysis is an extended LCC analysis where all kinds of indirect costs such as user costs are included. User costs are discussed in more detail later in this paper,

To illustrate the importance on including user costs in an LCCB bridge management system, a brief review of a few reports and other documents is presented in this paper, chapters 2 and 3. Notice, that in these (and most) documents, user costs are modelled deterministically although user costs are always very uncertain. Therefore, user costs must be modelled by stochastic variables or stochastic processes. However, a deterministic modelling based on statistic documentation is a good starting point for a stochastic modelling of user costs.

LCCB bridge management systems have a broad spectrum of application. They are very useful for groups of bridges, but also for individual bridges. In this chapter, the EU-supported LCCB bridge management system mentioned in chapter 2 is presented to illustrate how user costs may be included in decision-making for single bridges. LCC or LCCB systems may be used in designing a new bridge, but are also very useful in connection with decision problems regarding e.g. repair of a bridge after an inspection has taken place. As indicated in chapters 3 and 4, the total cost related to maintenance or replacement of deteriorated bridges will often be strongly dominated by the user costs.

The first major research on combining stochastic modeling, expert systems and optimal strategies for maintenance of reinforced concrete structures in an LCCB bridge management system was sponsored by EU from 1990 to 1993; see Thoft-Christensen (1995). The research project is entitled "Assessment of Performance and Optimal Strategies for Inspection and Maintenance of Concrete Structures using Reliability Based Expert Systems". The methodology used in the project is analytic using traditional numerical analysis and rather advanced stochastic modeling. In chapter 4, modelling of user costs is discussed on the basis of a simple bridge example analysis in this research project. Designing of LCCB bridge management system is briefly presented in chapter 5 for a single bridge, and in chapter 6 for a bridge stock.

In this chapter, modelling of user costs is discussed on the basis of a simple and straightforward implementation used in the above-mentioned research project. In the model, a number of issues are included such as closing down one or more lanes during maintenance. All relevant parameters are modelled as stochastic variables. The detour costs for a given bridge are estimated by considering the loss in the marginal benefits by having the bridge compared to no bridge, but only nearby routes for the traffic. This estimation is based on the average benefits for one vehicle passing the bridge by estimating the rental price of an average vehicle/km times the average detour length in km. Therefore, data on the traffic volume including increase in traffic volume per year are needed.

The model may easily be extended to include other kinds of user costs such as different categories of vehicles, cost of accidents, and loss of time due to the detour. Application of this LCCB system to a number of realistic examples shows that the benefits (negative user costs) play an important role. In Thoft-Christensen & Hansen (1993), a numerical example is shown where the user costs corresponding to different repair methods are much higher than the repair costs. Actually the repair costs are negligible compared with the user costs. In such cases the optimization should only primarily be concentrated on the user costs.

Including user cost in a probabilistic optimal bridge management system requires data for each year on the expected user costs for every single bridge, and an estimate of the probability that user costs will occur for every single bridge. It is absolutely not a simple matter to make reliable estimates of user costs. This fact is perhaps why most of the work on LCC bridge management systems completely neglects user costs or only mentions that user costs ought to be included. What is needed is a real comprehensive study of all aspects of indirect costs based on existing databases. However, most of these databases are not sufficiently detailed to be useful in that respect. User costs cannot be satisfactory modelled deterministic. The uncertainties are so high that a stochastic modeling is needed.

The importance of including user costs when the economic consequences of maintaining bridges are studied. It is argued that a cost-benefit analysis is needed when life-cycle analysis of maintenance (including inspection cost, repair cost, and user cost) of bridges is performed. This conclusion is based on an extensive study of documents on maintenance costs. From five of these documents, a limited number of excerpts are shown related to estimation of the importance of estimating user costs when repair of bridges are planned and when optimized strategies are formulated in the paper. Further, reference to three other documents is made. These excerpts clearly show that user costs in most cases completely dominate the total costs. In some cases, the user costs are even more than ten times higher than the repair costs.

The main conclusion of this paper is that an LCC based bridge management system in most cases is insufficient. User costs will in general dominate the cost of inspection and repair. Therefore, an LCCB analysis is more reasonable to use.

There is an enormous amount of work on user costs in bridge engineering in the literature. However, much more research is needed before an LCCB analysis in the bridge area can be made in a satisfactory way. Much of the work done until now is limited to narrow models without a wide area of application. A reliable life-cycle based tool must include direct as well as indirect cost. The bridge owners must learn to listen to the public when decisions regarding repair or replacement of structures are taken.

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Governing issues and alternate resolutions for a state department of transportations' transition to asset management

A.E. Aktan & F.L. Moon
Drexel University, Philadelphia, PA, USA

1 INTRODUCTION

Since 1991, the US Federal Government has required each state to implement pavement, bridge, safety, congestion, public transportation and intermodal management systems. While this represented an important improvement in the management of transportation infrastructure, the potential benefits were greatly limited by the disconnected and fragmented approach employed by the various transportation agencies. As a result, it is now well-recognized that infrastructures are complex intertwined systems, requiring a holistic "integrated total systems" approach for their effective engineering and management. According to Thompson (2004), the need for applying asset management principles to public infrastructures was highlighted as early as 1997 by the US General Accounting Office (GAO, 1997), and the Government Accounting Standards Board (GASB) established a set of accounting requirements in 1999 (GASB, 1999). The Federal Highway Administration (FHWA) established the Office of Asset Management in 1999 (http://www.fhwa.dot.gov/infrastructure/asstmgmt/), and the American Association of State Highway and Transportation Officials (AASHTO) published a process-oriented guideline in 2002 (Cambridge Systematics, 2002).

The highway transportation community agrees that the following fundamental building blocks would need to be in place to facilitate a successful transition from management of individual asset groups to a meaningful integrated asset management of the entire highway transportation system:

(1) System-level performance measures and objective measurable indicators should be defined.
(2) Performance measures and indicators for specific asset groups and the interactions between the performances of different asset groups should be defined.
(3) A system performance model incorporating the performances of each asset group and the interactions between these should be constructed, and the associated data needs and measurement standards to quantify and validate this model should be established. The measurement standards should recognize the aleatory (randomness) and epistemic (incomplete knowledge) classes of uncertainty associated with different data.
(4) Interoperable databases composed of both the legacy data available for various asset classes and measured objective data should be developed. These databases should also include an accurate and complete nuts-and-bolts inventory of all assets.
(5) Valuation and performance of each major asset group, and how individual asset groups influence each other's valuation and performance should be established based on the data and models in (3) and (4). The system-level valuation, performance modeling and simulations for scenario analyses should follow. In addition, these models should be repeatedly checked, validated and/or modified over time based on continued data collection.

2 OBJECTIVE AND SCOPE

It is clear from the Introduction that the challenges to making a rational transition to asset management of highway transportation are significant; however, the value and the responsibility for the stewardship of our transportation assets is a far more significant and formidable driver. To date, the concepts and fundamental building blocks of asset management, required for achieving this goal for complex infrastructure systems have been well-established. However, successful applications have been limited only to various private utilities, telecommunication and some transportation-related industries such as airlines. In contrast, some highway agencies have merely initiated the exploration of the concept, and are introducing adjustments and minor changes to the way they manage their assets. For example, some states have started implementing more comprehensive condition and performance models for bridges while others have been exploring the integration of corridor level pavement and bridge management. Many highway transportation agencies are now getting ready to take on the discussed challenges and start a transition to asset management. Naturally, given the complex social and technical systems and related mechanisms that govern the day-to-day operations and long-term programming of a major state Department of Transportation's (DOT) expenditures, making a meaningful transition to asset management requires an extensive effort for objective fact-finding, planning and consensus building.

The Pennsylvania Department of Transportation (PennDOT) is one agency that has been keenly interested in a transition to integrated asset management from managing individual asset groups separately. PennDOT's inventory of assets is one of the largest in the nation, distributed within urban, suburban, and rural Counties stretching from the Great Lakes to the Mid-Atlantic. Given the past, current and emerging federal and state policies and budgets, increasing congestion and other indicators of less than desirable performance, and new demands on resources by homeland security concerns, making a transition to integrated asset management is essential and inevitable. However, such a transition requires a rethinking of policy, strategy and planning, and modifications to current practice at many levels (asset groups, districts, states, regions, entire nation). The transition may occur along a time-horizon of several years to decades, based on whether marginal, considerable or significant and fundamental changes are accepted. For example, a DOT may start with an integration of only pavement and bridge maintenance management as an initial step. In any case, this is a major undertaking with many alternative strategies. One prudent strategy may be to first conduct an experiment by transitioning only a small sample but the entire set of the assets, including the corresponding organizational components.

Developing a road map to help make the transition – given the current status of the organization, the condition of assets under stewardship, and the changing demands, revenue and resources – is a complex, multi-objective, socio-technical, dynamic optimization problem. For success, it is necessary to evaluate and incorporate the current and future social, political, legal and other external constraints as well as the organizational (internal) constraints. Meanwhile, there is also a lack of knowledge on the engineering performance and behavior of the assets as a system. A meaningful road map that will offer the best chance for success requires a government-academe-industry partnership, integrating expertise from social sciences and various engineering fields. It is especially critical that the partnership is one integrating academe, government and industry, as all three entities have to work together for any meaningful infrastructure innovation.

In this paper, the writers are motivated to discuss how they have identified, framed and structured the issues that govern a transition by a major state DOT such as PennDOT to asset management. The issues will be classified as social, political, organizational and technical, and the interactions between them will be pointed out and mapped. Following a classification of the issues that govern such a complex sociotechnical system of systems, alternative resolutions for these issues will be formulated and discussed including, top-down, bottom-up, and mixed approaches.

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A budget management approach for societal infrastructure projects

K. Nishijima & M.H. Faber
Swiss Federal Institute of Technology, Zurich, Switzerland

1 INTRODUCTION

Life cycle costing analysis is broadly applied as a tool for decision support in regard to maintenance planning for civil engineering structures, whereby the expected total cost is advocated as the objective function to be minimized. The present paper takes the new perspective of considering the problem from a budgeting allocation problem where the aim is to optimize the allocation of budget for the purpose of maintaining the operation of the considered structures. Budgeting can be considered as a societal resource allocation which should be optimized from a broader organizational viewpoint. The amount of budget required for a facility or a portfolio of facilities may not be the same as the expected cost, since the lack of budget may cause consequences as a consequence of postponed maintenance. The justification for requiring more budget than the expected cost is given taking basis in the maximization of the net benefit. Whereas all the consequences associated to the project must be taken into account in the life cycle costing analysis, it is important to distinguish the financial costs which must be paid, e.g., repair cost or inspection cost, from the user costs which represent the follow-up consequences, i.e., opportunity losses caused by the loss of the expected functions of facilities or due to the effect of postponed maintenance. This is because only the costs to be paid are related to the budget. The present paper proposes an approach to determine the optimal amount of budget and the optimal maintenance decisions, considering these two types of cost. The maintenance planning of a RC structure portfolio illustrates how the proposed approach may be implemented in practice.

2 BUDGET MANAGEMENT APPROACH

Subject to significant uncertainties related to civil engineering projects, decision makers must decide on the amount of budget necessary and sufficient for successfully managing the projects. It has been widely accepted in life cycle costing analysis that the objective function to be minimized is the (discounted) expected total cost including follow-up consequences such as user costs. In this regard, it can be said that the optimization by the minimization of expected total cost implicitly assumes a "perfectly flexible budgeting", namely, a situation where the budget is always available when needed. For the purpose to assess the optimal amount of budget required for a project or projects, however, this may not be appropriate. A structure which has reduced availability due to failure or the need of repair works may not be rehabilitated due to insufficient budgets, which in turn may lead to additional user costs. The optimal budget allocation may not correspond to the expected total cost. In order to maximize the net benefit, the budget allocation, the financial costs and user the cost must be considered simultaneously.

In a broader sense, the objective function should be an aggregated utility, in which all the preferences of the decision maker are included, see e.g., Faber & Maes (2003). In practical situations, a decision maker may be precautious in a sense that he/she requests more budget than the expected cost in order to ensure a successful management of projects.

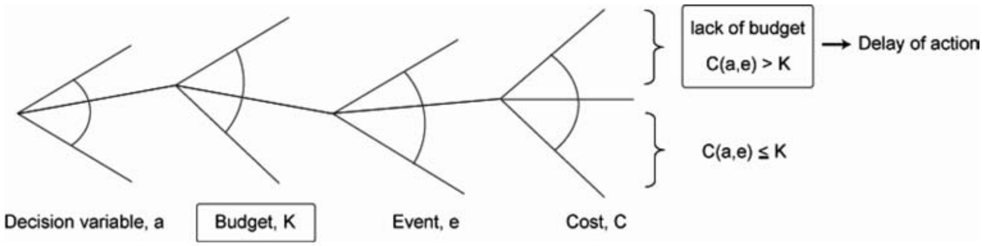

Figure 1. Decision event tree including Budget.

The net benefit is proposed to be the utility function for the representative of decision maker's preference. The net benefit induced by a project, *NB*, may be written as:

$$NB = \begin{cases} B - K - \Delta B(e) & (C(a,e) \leq K) \\ B - C(e,a) - \Delta B(e) - \Delta B(e,'C > K') & (C(a,e) > K) \end{cases} \quad (1)$$

where K is the allocated budget, $\Delta B(e)$ is a user cost corresponding to event e, $C(a,e)$ is the financial cost corresponding to (a,e), a is the decision variable and $\Delta B(e,' C > K')$ is a user cost induced by the possibly insufficient budget following the event e, see Figure 1. If the financial cost $C(a,e)$ does not exceed the budget K, the net benefit is the difference between the benefit and the sum of the budget and the user cost associated with the event e. Here it is assumed that the unused part of the budget within a budgeting period is not transferred to the next budgeting period which is a commonly known difficulty in the public sector. If the financial cost $C(a,e)$ exceeds the budget K, an extra budget must be asked for in order to reinstate the reduced availability, which will be provided at some later point in time, e.g. the subsequent budgeting period. Until the extra budget is obtained, the availability remains reduced, causing the additional user cost $\Delta B(e,' C > K')$.

As the amount of budget increases, the probability of budgeting failure $P(C > K)$ and the net benefit decreases, and vise versa. The optimal budget K^* and the optimal decision variable a^*, e.g. concerning inspection and maintenance activities are obtained by maximizing the expected net benefit $E[NB]$:

$$E[NB] = \int_E NB(a,e,K)dP(e;a) \quad (2)$$

where E is the set of possible events e and $P(e,a)$ is the probability of the occurrence of the event e given the decision variable a.

3 CONCLUSION

The proposed approach provides a rational framework for decision makers responsible for the budgeting and planning of maintenance activities for portfolios of facilities and leads to optimal budgets which are consistent with the adverse effects of possible insufficient budgets. For the purpose of illustrating the application of the proposed approach the problem of maintenance planning for a portfolio of RC structures subject to chloride-induced deterioration is considered. The example clearly shows that the optimal budgets differ from the commonly applied expected total costs and this also has an effect on the optimal choice of inspection plans.

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Societal aspects of bridge management and safety in The Netherlands

H.E. Klatter
Ministry of Transport, Public Works and Water Management, Civil Engineering Division, Utrecht, The Netherlands

A.C.W.M. Vrouwenvelder
Delft University of Technology, Delft, The Netherlands

J.M. van Noortwijk
HKV Consultants, Lelystad, The Netherlands, and Delft University of Technology, Delft, The Netherlands

1 INTRODUCTION

Bridges and other structures are designed for lifetimes of 50 to 100 years. A careful choice of requirements for a bridge is crucial in fulfilling societal needs, not only now, but also in the future. The cost of maintenance is substantial in the long run, because it depends on design choices made. For this kind of problems, a life-cycle costing (LCC) approach is an absolute necessity.

2 LIFE-CYCLE COSTING (LCC)

Generally, the following life-cycle phases can be identified: design, construction, use, and demolition. All costs should be considered, regardless of the funding source. Therefore, life-cycle costs should not only include the direct costs of construction, maintenance, and demolition, but also the indirect costs for the society and environment. Examples of life-cycle costing (LCC) are balancing the initial cost of investment against the future cost of maintenance, and balancing the costs of preventive maintenance against the costs of failure. Another example is the balancing between complete replacement of a coating and lifetime-extending maintenance (LEM) by means of spot repair. Complete replacement is expensive because a protective shield is required to seal off the maintenance activities from the local environment. A software tool developed by Rijkswaterstaat for this optimisation problem will be discussed (van Noortwijk and Frangopol, 2004).

3 DETERIORATION AND COST MODELS AND DATA

In order to perform the LCC calculation one needs models and data on the structural behaviour, the deterioration processes, characteristics and constraints of inspection and repair and–last but not least–costs. One option is to have a look at costs only. An example of such a model is given in Figure 1. Based on the data it may be concluded that for large bridges up to $t = 40$ year the yearly costs are almost constant, but that between 40 and 50 year a peak in maintenance is to be expected.

A step further in understanding is to include the effect of for instance a change of the geometrical dimensions of material properties. For that case we need models that describe the structural condition as a function of the ongoing deterioration. An example of this is given in Figure 2 where spalling of concrete bridges is modelled as function of the bridge age (Gaal, 2004).

Reality however is that reliable and mature physical models simply are not available for all relevant materials and environments for the time being. Probably, it is the best to combine physical models with statistical cost models. Such a mix could give the best possible guarantees to reach optimum maintenance schemes, including all data and knowledge that is available today.

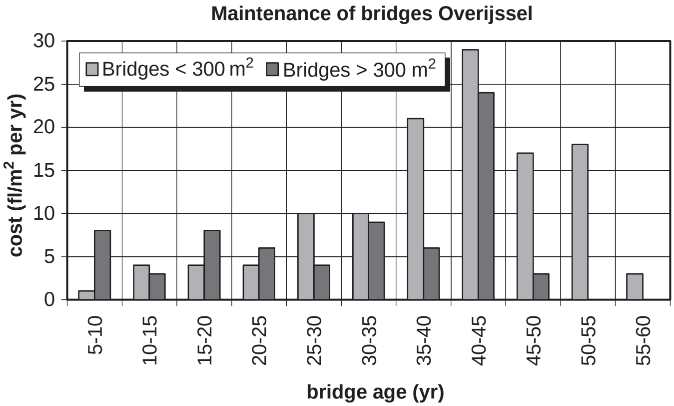

Figure 1. Maintenance costs versus age for bridges (less and larger than 300 m^2) in the Province of Overijssel (The Netherlands).

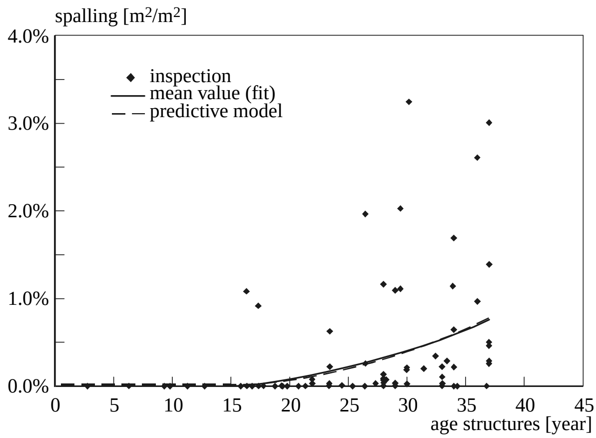

Figure 2. Comparison of theoretical prediction and inspection results for spalling (Gaal, 2004).

4 EVALUATION

A decision-theoretic approach for bridge management and safety has been defined in this paper with life-cycle approach as a key element. Although maintenance became more transparent, its effectiveness is yet to be proven. Especially, the effect of maintenance on the performance of infrastructures in socio-economic terms on a network level needs to be developed. In this respect, important items to be addressed on the engineers' agenda are the performance of infrastructure and a risk-based approach expressed in public understandable language.

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Safety of medium and long span bridge superstructures during the erection phases

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Importance of modal cross-correlation on wind loaded structures

V. Denoël, H. Degée
University of Liège, Department of Mechanics of Materials and Structures, Liège, Belgium

V. de Ville de Goyet
Bureau d'Etudes GREISCH, Angleur, Belgium

1 INTRODUCTION

In civil engineering applications, the dynamic analysis of large structures is often performed in a modal space. This method is known to offer an interesting decrease of the number of degrees of freedom as well as a decomposition of the structure's response into uncoupled components. Even if the response in each mode can generally be computed independently, the coherence of these modal responses must be accounted for in the determination of the structural response to stochastic loading. In this paper, we show that these modal cross-correlations can be significant even in the case of well-separated natural frequencies, opposite to what is sometimes thought. It is illustrated by an analysis of the Viaduct of Millau during an erection stage.

2 MODAL COUPLING IN STOCHASTIC ANALYSIS

The first objective of a stochastic analysis in a modal space is to determine the expression of the covariance matrix of the modal amplitudes. Diagonal elements of this matrix are the variance of the amplitude in each mode whereas the off-diagonal elements represent the correlation existing between amplitudes in different modes.

The amplitudes in the different modes have to be combined in order to obtain estimations of the displacements of the structure. This problem is not necessarily easy since all amplitudes are represented by positive values. However this problem has already been well debated, as it can be seen for instance in the difference between CQC and SRSS combination methods in seismic engineering. In a stochastic analysis, this operation is realized by pre- and post-multiplying the covariance matrix of the modal amplitudes by the matrix of mode shapes in order to obtain a covariance matrix of the displacements:

$$\left[\mathrm{cov}_x\right] = [\Phi]\left[\mathrm{cov}_\eta\right][\Phi]^T \tag{1}$$

which can also be written:

$$\begin{aligned}\sigma_{x_i}^2 &= \sum_m \sum_n \Phi_{im}\Phi_{in}\rho_{\eta_{mn}}\sigma_{\eta_m}\sigma_{\eta_n} \\ &= \sum_m \Phi_{im}^2\sigma_{\eta_m}^2 + \sum_m \sum_{n\neq m} \Phi_{im}\Phi_{in}\rho_{\eta_{mn}}\sigma_{\eta_m}\sigma_{\eta_n}\end{aligned} \tag{2}$$

and shows clearly that the variance of a displacement is composed of (i) a single summation referring to variances of modal amplitudes and (ii) a double summation referring to correlations between different modes. In order to accelerate the computations, this second term is often neglected.

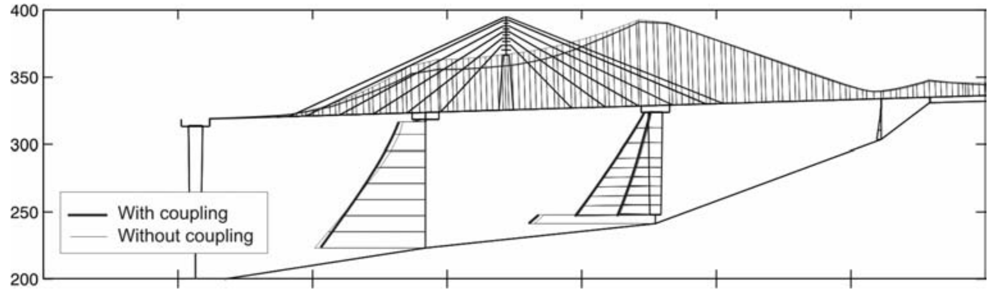

Figure 1. Standard deviation of the out-of-plane bending moments.

From Equation (2), it is obvious that this double summation can be neglected if the correlations between different modes ($\rho_{\eta_{mn}}$) are small. The present paper discusses the validity of this approximation in the context of wind-loaded structures. According to the assumptions that are generally admitted in the field, it can be demonstrated that the modal correlation is expressed by:

$$\rho_{\eta_{mn}} = \gamma_F \rho_{F^*_{mn}} + \gamma_D \alpha\left(\omega_m, \omega_n\right)\Gamma \tag{3}$$

where γ_F and γ_D are coefficients depending on the repartition of energy between the background quasi-static component and the resonant component. This relation shows that the correlation coefficient between the modal amplitudes can be approached by a weighted combination of (*i*) the correlation coefficient of the generalized forces ($\rho_{F^*_{mn}}$) and *(ii)* a dynamic correlation coefficient expressed as a function of the proximity of the natural frequencies [$\alpha\,(\omega_m, \omega_n)$] and of the coherence of the generalized forces (Γ). This simple formula shows that, contrary to what is often thought, the proximity of the natural frequencies is not the only parameter that can influence the modal correlation coefficients.

3 APPLICATION

The influence of the modal cross-correlations is illustrated on the famous Viaduct of Millau (France) during an erection stage. For this particular structure, and with conditions of loading described in the full paper, Figure 1 shows the standard deviations of the out-of-plane bending moments in the deck and in the piles. It can be seen that coupling terms affect these bending moments with a maximum effect of approximately 20% in the deck under the pylon.

4 CONCLUSIONS

It is sometimes thought that cross-correlation terms must be accounted for in the case of closely spaced natural frequencies only. In this paper we have used the classical decomposition of the modal response into a background and a resonant contribution to show that modal cross-correlation terms can also have a quasi-static origin.

In the most general case, the lack of modal coupling shouldn't be concluded before having inspected (i) the proximity of the natural frequencies (ii) the coherence of the generalized forces in the vicinity of the natural frequencies and (iii) the correlation coefficients of the generalized forces.

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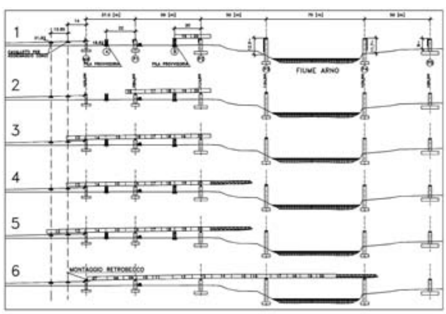
Steel bridges launching and safety against patch loading

E. Maiorana & A. Miazzon
OMBA Impianti & Engineering S.p.A., Vicenza, Italy

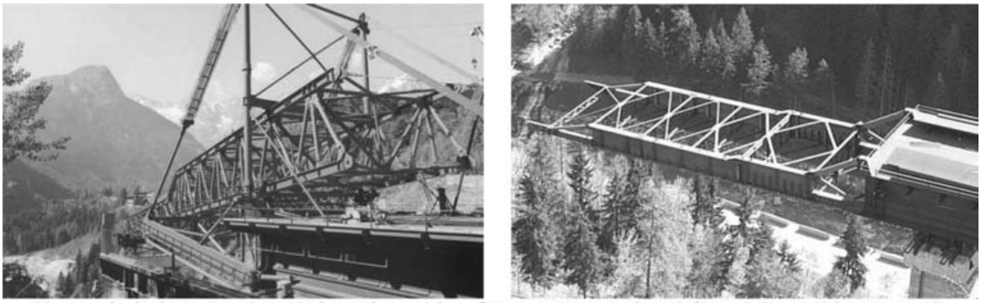
ABSTRACT: There are a large variety of construction methods used during bridges erection; one of the most interesting, particularly when bridges have long spans or have to overpass deep valley or rivers, is the launching of the bridge from an abutment. The launching phase, generally, is an important phase but last relatively a very short time compared to the total service life of the structure; consequently it is necessary that an accurate preliminary analysis of the technical approach of the erection method statements is adopted. It is also relevant to carry out precise, complete and consistent calculations to understand more exactly the mechanisms that the might structure develop, during the erection phases, in order to assign minimal safety coefficients required by the codes in force or in the technical specifications prepared for this temporary condition. Herein some specific investigations are proposed in order to solve satisfactorily the difficult problems met during the bridge erection: patch loading is one of the latter. Briefly some experiences gained in bridge launching is discussed, focusing on interesting solutions of some practical aspects and on erection techniques, like pull launching of the bridge in a wide curved or long length spans, using special but suitably designed equipments with due account taken of the cost-benefit aspect, when choosing a solution amongst those that are technically practicable.

The interaction between patch loading and bending moment in the elastic domain has been exhaustively analysed and described for design procedure. The proposed relationships are valid for

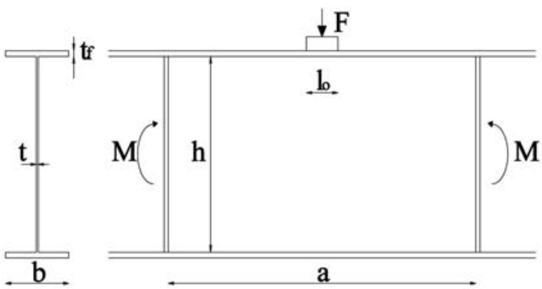


Arno bridge (2003). Launching phases

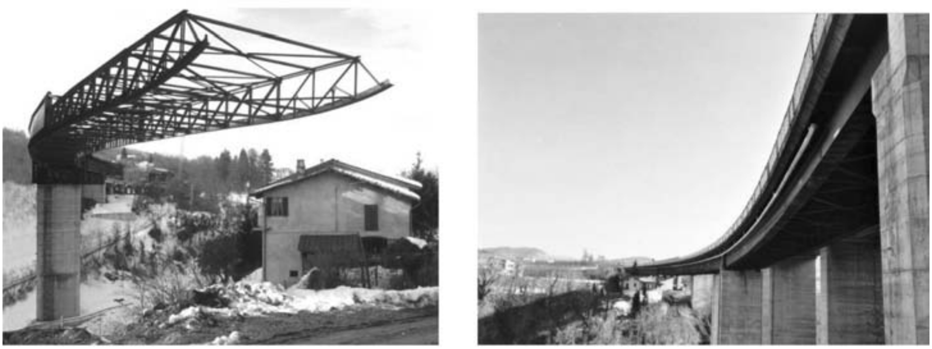
Verrand viaduct (2001). Connection of the two semi-bridges (during launching)



Verrand viaduct. Lattice girders (launching from Aosta yard and from Monte Bianco yard)



Longitudinal section of a typical girder web panel



Tanaro viaduct (2000). Lattice girder and overview

global stability of plates with $a/h > \sqrt{2}$ (class 4 of Eurocode 3-1-1), for which instability shapes are different in the two main directions of symmetry and also because those rectangular panels are commonly used in steel girders. The conclusions of the analysis are likely to be a contribution to the design of patch loaded web panels. The computational procedure described allows to evaluate in a simple way the reduction of elastic critical loads for patch loading due to the interaction with bending moments. It should be necessary to develop a comparison between more extensive numerical analyses and corresponding experimental results, in order to definitively validate the proposed method. On the other hand, the solution of a complex problem like the evaluation of the elastic critical load of web girders during bridge launching, subjected to combined patch loading and bending moment, is simple and accurate by using the proposed method.

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Safety of balanced cantilever and cable stayed bridges during construction

A.J. Reis
GRID Consulting Engineers, Lisbon, Portugal
IST-Technical University of Lisbon, Portugal

ABSTRACT: Bridge structures namely Balanced Cantilever (BC) and Cable Stayed (CS) Bridges are particularly sensitive to actions during erection stages.

Actions and structural safety criteria during construction are referred in bridge codes but not always specified. It is a designer responsibility to decide on the referred criteria. Simplified approaches are discussed, illustrated by design cases based on the author's design experience.

A basic issue at ULS during the erection stages is the definition of the construction loads and partial safety coefficients γ_g and γ_q for the fundamental load combinations, associated to permanent (G) and variable loads (Q) namely construction loads and wind loads.

During first erection stages (Fig. 1) of a BC bridge, the stability of the piers requires a geometrical and material nonlinear analysis or at least an approximate method to estimate the nonlinear effects. The approximate second order bending moment at the base section of the pier may be taken as:

$$M_{sd} = \gamma_g P w_{\max} + \frac{1}{2}\gamma_q \, q\ell^2 \tag{1}$$

where w_{max} is the maximum second order deflection at the top, P the total load at the top including 1/3 of the self weight (P^{ℓ}) of the pier and q the transverse load due to wind. For w_{max} one takes

$$w_{\max} = (\delta_o + \gamma_q \frac{q\ell^4}{8EI_{ef}}) \frac{1}{1 - \frac{\gamma_g P}{P_{cr,ef}}} \quad with\ P = \frac{p\ell}{3} + \sum_i G_i \tag{2}$$

Here δ_o is the amplitude of the geometrical imperfection at the top, and the critical load $P_{cr,ef}$ is determinate for the effective (cracked) bending stiffness EI_{ef} of the pier section, at the initial yielding of the reinforcement under bending and axial force.

During the balanced cantilever scheme of a curved box girder bridge (Fig. 1b), the dead load of the deck induces transverse bending moments in the piers, unless some provisional prestressing internal or external, is adopted. To get a full balance of the transverse bending moments in the pier shaft, it requires a prestressing force

$$\mathrm{P_o} = \frac{1}{\mathrm{Re}} \sum_{\mathrm{i}} \mathrm{G_{ni}}\ \mathrm{x_i^2} \tag{3}$$

where R is the radius of curvature and G_{ni} the nominal weights of the segments. The bending moments ΔM_y are due to deviations in G_{ni} and the bending moment M_x are due to the inplan curvature.

Balanced cantilever bridges, with tall piers, present low natural frequencies in bending and torsional modes of the pier (Fig. 1). It is usual to have longitudinal and transverse bending frequencies and torsional frequencies f_x,f_y and f_t respectively for tall piers (say between 60 and 100 m) and spans between 100 and 180 m in the range 0.1 Hz to 0.4 Hz. An example is shown in Fig. 1(a) where $f_x = 0.17$ Hz and $f_t = 0.29$ Hz.

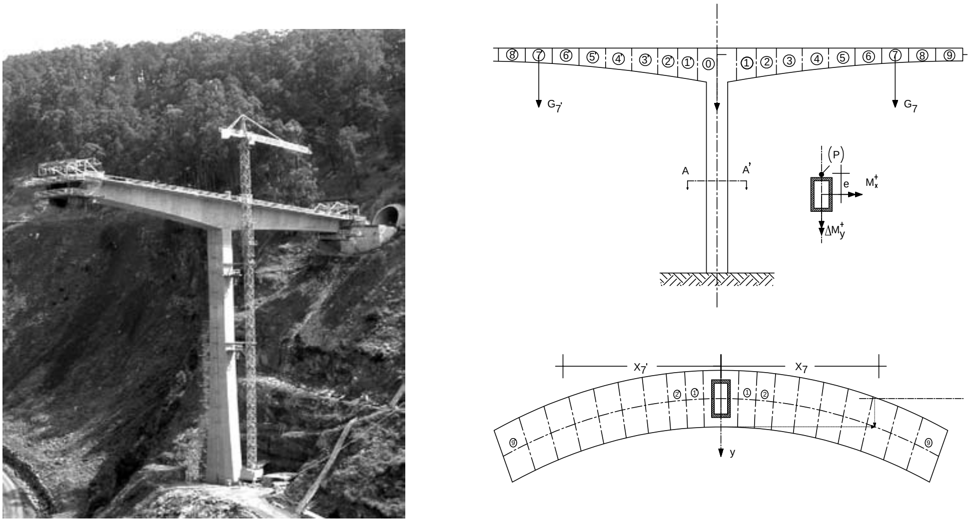

Figure 1. (a) The Ribeira Finda bridge, in Madeira Island. Main span 135 m; Fig. 1(b) Curved balanced cantilever bridge during construction.

Under random excitation and due to wind gusts, bending and torsional effects due to unbalanced wind buffeting shall be considered in design when checking the structural safety during construction stages.

The characteristic average wind speeds ($U_{k,50}$) are usually defined for time intervals of 10 minutes in a reference period of 50 years. For checking the structure during construction one adopts a shorter reference period T, say T = 10 years for a construction time not greater than 1 year. The associated characteristic averages wind speed may be estimated as

$$U_{k,T} = U_{k,50}\ (1 - 0071\ \ell n\ \frac{50}{T}) \tag{4}$$

yielding for T = 10 years $U_{k,T} = 0{,}88\ U_{k,50}$, i.e. a 23% reduction in the wind loads which are function of U^2.

In the case of a BC bridge during construction (Fig. 1) the symmetry of deck with respect to the pier yields a zero mean torsional moment due to the wind actions. If one takes into consideration the fluctuations of the wind speed along the overhangs, the wind speed does not reach the maximum value at each point x and (−x) simultaneous. One may keep the concept of a gust factor φ_t for the torsional moment T_k (characteristic value) if one takes the torsional moment T_h induced in the pier by the mean wind speed acting in one of the cantilevers only (half deck), i.e.$T_k = \varphi_T\ T_h$

The approaches to evaluate T_k are quite complex and in many cases limited by specific constraints of the design case as the topographic influence induced in the wind speeds in a bridge located in a valley. The piers may be to close to the slopes to disregard this effect (Fig. 1). To model this effect numerically may be very complex. Only model studies in wind tunnels may give sufficient reliable results for design practice. These wind tunnel tests may be based on rigid models assuming the geometry of the BC bridge is not susceptible to any aerodynamic instability effects. Reducing the analysis of the structural response to the first 3 modes, it is possible to show the generalized aerodynamic forces may be evaluated from the moments measured in the rigid model in the deck and in the pier.

From results based on the theorical approaches previously referred, one may say that for spans L up to 80 m, the approach based on the mean wind speed in one overhang only i.e $\varphi_T = 1$ is acceptable. For 80 < L < 180 m, values of φ_T reaching values up to 2,5 may be obtained. For BC

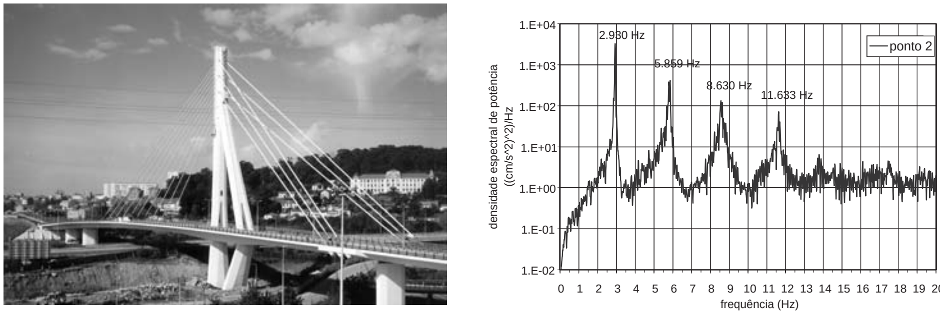

Figure 2. (a) The Viaduct over VCI in Oporto. A cable stayed bridge (central suspension) with a main span of 120 m; Fig. 2(b) Eigenfrequencies of the stays from the power spectrum.

bridges located in valleys, with large spans (say L > 120 m), either a wind tunnel test is made or some temporary cables anchoring the deck to the hill may be installed, as was the case of Fig.1(a).

Cable stayed bridges, with a prestressed concrete or a composite deck are usually built by the cantilever method. The deck may be erected by a balanced cantilever scheme from each mast or by an "unbalanced" asymmetric scheme if lateral spans have been previously erected as usually in single mast cables stayed bridges (Fig. 2). The adjustment of the forces in the stays is made in order to control the internal forces in the deck (force control) as well as to reach a previously defined camber plan (geometry control). Assuming a linear elastic response, the influence matrix of the forces of the stays, at each construction stage, can be determined. Hence a vector $\{x_j\}$ – adjustment of the forces in the stays, represents the design parameters at each construction stage.

The stay forces are evaluated with an equivalent modulus of elasticity E_{eq} given by Ernst formula, which approach E_o under increasing loading. The forces in the stays may be checked during construction by comparing the values measured in the stressing operations with the values determined from the measured natural vibration frequencies of an equivalent string. The main issue, is the definition of the "free" length L because, for short stays the influence of the anchorage length may be meaningful. Using the measurements from an accelerometer, the eigenfrequencies may be identifying from a power spectrum (Fig. 2b).

*Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Patch loading resistance of longitudinally stiffened plate girders

U. Kuhlmann & B. Braun
University of Stuttgart, Institute of Structural Design, Stuttgart, Germany

1 INTRODUCTION

For girders of steel and composite bridges being incrementally launched the resistance of the slender web plate subjected to patch loading is a matter of particular interest. During the launching process large support reactions are introduced as transverse forces into the girder web interacting with high bending moments. In almost every case the girder has to be strengthened for this temporary loading situation.

Strengthening may be undertaken e.g. by thicker webs and/or by longitudinal stiffeners. For bridges with large web heights where longitudinal stiffeners are rather commonly used to increase the bending and shear resistance it is beneficial to consider the influence of existing longitudinal stiffeners first instead of increasing the web thickness. However it is difficult to take advantage of longitudinal stiffeners as the majority of design standards does not provide appropriate rules for the conditions governing such a design situation e.g. long loading lengths.

2 EXPERIMENTAL AND THEORETICAL INVESTIGATIONS

Research on the ultimate resistance of longitudinally stiffened girders began in the 1970s and has been continued until today. A literature review shows that the majority of tests is limited to girders with short loading lengths, stiffener positions very close to the loaded flange and stiffeners with open cross-sections with a small torsional rigidity. Closed-section stiffeners have been studied only in very few tests with mainly short loading lengths. However in all studies a significant increase of ultimate loads could be observed due to longitudinal stiffening. The tests also show the tendency that closed-section stiffeners lead to a higher ultimate load than the use of open-section stiffeners.

Due to the limited number of tests with medium or long loading lengths and closed-section stiffeners a verification of the aforementioned tendencies required additional experimental investigations. Within this scope seven full-scale tests have been conducted at University of Stuttgart to study the influence of stiffener position, number of stiffeners and additional bending moment. It could be shown by the tests that the use of closed-section stiffeners substantially increases the ultimate load of patch loaded plate girders. In order to be able to examine the influence of further parameters not included in the experimental investigations a Finite-Element model has been developed and verified on the basis of the experimental results.

3 INFLUENCE OF STIFFENER POSITION AND LOADING LENGTH

Numerical investigations were conducted to study the influence of the stiffener position and the loading length on the elastic critical buckling loads of the subpanels and the patch loading resistance. The collapse loads were determined by nonlinear Finite-Element calculations. From the results it can be concluded that the ultimate load is not only dependent on the stiffener position h_1/h_w but also on the loading length c. Furthermore it is shown that the dependencies of collapse loads on the stiffener positions do not correlate with the corresponding first eigenmodi of the whole stiffened girder panel. As the subpanel failures show different post-critical reserves the determination of collapse loads of longitudinally stiffened girders have to take that into account.

4 PROPOSAL FOR AN IMPROVED DESIGN PROCEDURE

On the basis of the experimental and numerical investigations an improved design procedure (Seitz, 2005) has been developed which comprises a verification of the single subpanels and a verification of the stiffener. For the verification of the subpanels a modified approach based on prEN 1993-1-5 (2005) is used to allow for an appropriate determination of the collapse loads and its post buckling reserves compared to the critical buckling loads. To prevent the premature failure of the stiffener a second-order theory calculation considering newly derived geometrical stiffener imperfections is proposed. A comparison of resistances of the improved design procedure to prEN 1993-1-5 (2005) and current design proposals prove the applicability and the economic advantage of this approach for the prediction of the subpanel resistance.

5 INFLUENCE OF AN ADDITIONAL BENDING MOMENT

The girder properties used in the studies on stiffener position and loading length have been adopted to investigate the influence of an additional bending moment. A bending moment level of 70% and 90% of the moment resistance has been applied to the girders. An average decrease of ultimate loads of approximately minus 13% or 41% has been found for a preliminary study case. This is in contrast to the experimental results that showed no reduction of the patch loading resistance due to additional bending moments. So further investigations are necessary.

6 CONCLUSIONS

Experimental and numerical studies on plate girders strengthened by closed-section stiffeners prove the effectiveness of longitudinal stiffeners especially for large loading lengths. The results of the additional numerical studies show the applicability of the proposed design procedure but also the need for a modification of the established rules. Therefore a modification has been proposed which enables an appropriate determination of the collapse loads for each individual subpanel especially for long loading lengths. In order to complete the approach an additional criteria for longitudinal stiffeners has been developed on the basis of a second-order theory calculation. It was shown that the influence of an additional bending moment can have a decisive influence on the ultimate patch loading resistance. Further studies will investigate the applicability of the interaction curve of prEN1993-1-5 (2005) to the improved design procedure.

ACKNOWLEDGEMENT

This paper has been written with the support of Dr.-Ing. Martin Seitz, Stuttgart, Germany whose doctoral thesis (Seitz, 2005) forms the origin of most of the results presented here.

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Buckling of steel tied arches during erection

Ph. van Bogaert & A. Outtier
Civil Engineering Department, Ghent University, Ghent, Belgium

1 INTRODUCTION

According to a number of cases being considered, most arches seem sufficiently stable to give up the use of supporting towers. The first question in the transitory situation concerns the partial safety factors at ultimate limit state during erection, where the use of construction towers is not always necessary. The second one deals with the correct determination of arch imperfections.

2 PARTIAL SAFETY DURING ERECTION

The use of scaffolding towers may introduce reactions caused by wind. If the arch is not closed yet and as wind introduces a torque, the latter results in clamping moments at the arch springs, which are transferred to the edge crossbeams of the bridge deck. These clamping moments do not exceed the arch spring lateral bending moments in the final structure. The use of scaffolding towers radically changes the transmission of wind forces. The torque is resisted by vertical forces on the towers, transmitted to the bridge deck. Hence, the use of scaffolding towers may cause disturbance of the final effect of dead weight and critical conditions during erection.

Safety and reliability during erection phases are not considered separately in Eurocodes. In fact, these codes provide guidance to reduce wind effects during erection and to consider alternative loading on temporary bearings. In fact the only clause considering temporary phases states that at serviceability state, yield stresses should not be exceeded. Any reference concerning ultimate limit state lacks and this might be interpreted as if load combinations and factors should be identical as for final situations. This seems rather inconsistent with the fundamental reliability concept. Probabilities and risks cannot be identical for a very short-term condition compared to a final situation. During temporary phases it is more logical to introduce a safety margin, taking into account the risk and the total cost of alternative measures to increase reliability.

Based on this assumption, we find the optimum safety margin as:

$$g_{opt} = \sigma_g \cdot \frac{1}{\gamma} \cdot \ln\left(\frac{\gamma \cdot \alpha \cdot C_f}{B \cdot \sigma_g}\right) \tag{1}$$

This has been applied to the case of a 136.2 m wide steel tied arch bridge, launched across a railway site, for which the cost of rebuilding in case of total loss during erection would be 4-times as much as the initial construction cost. For this comparison, the value of σ_g is dominating the result of the safety index β_o. For 7% of standard variation, the partial safety factor equals $\gamma_s = 1.2$. This value certainly is much lower than the classical value at ultimate limit state for dead weight of 1.35, reflecting the fact that lower factors should be used for temporary situations.

3 BUCKLING OF STEEL TIED ARCHES

A recently built bridge, spanning 115 m, was used as a testing case for the calculations. This bridge is part of the high-speed railway connection between Antwerp and Amsterdam. During loading

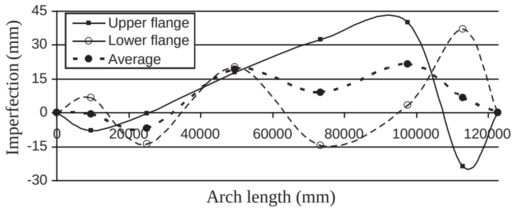

Figure 1. Imperfection in the upper and lower flange, and average imperfection.

tests with lorries, strain measurements were made. In order to verify the buckling resistance of the bridge, a refined FE-model of the bridge has been developed. Special attention was given to the bearings and the connection between the arch springs and the lower chord edge members, since a first calculation run of the model has shown that a torsional effect exists at the arch springs. On the one hand, it is used to re-calculate a given imperfection. Comparing a perfect arch and an arch having imperfections it clearly appears that changes in the stress field arise. On the other hand, the model enables to find the buckling curve of an arch with imperfections, using elastic-plastic analysis, which allows distinguishing the lateral displacement increase with increments of the load before reaching the bifurcation point.

The second focal point of this research is to formulate an analytical approximation of the problem. For various positions of the lorries, strains have been measured at 6 sections across the span, recordkeeping the live load effects only. The calculation scheme is as follows:

– First, the stresses caused by the normal forces in the arch section are calculated.
– Secondly, stresses derived from the in-plane bending of the arch are computed.
– The stresses derived from out-of-plane bending are derived. This effect is caused by the bending of the edge floor beam, which causes torsion of the arch section and the out-of-plane bending moments M_Z, which can be calculated using the slope deflection method as a continuous beam on five supports.
– In a further phase other secondary effects can be incorporated into this method.
– Based on the still unknown imperfections Δy, their influence on the stresses is calculated using the following formula:

$$\sigma_{imperfection} = \frac{N \cdot \Delta y \cdot y}{I_z} \qquad (2)$$

– By adding together all previous stresses and equalling them to the experimentally determined stresses, the unknown imperfections can be found.

The imperfection of the upper and lower flanges is shown in Figure 1. All imperfections do not exceed 40 mm, which is more or less the interval to be expected, especially if the average value is considered, which rises to about 20 mm for. It was important to take into account the effect of M_Z, because otherwise the imperfections at the arch springs would have been unacceptably high. Still, no more than 5% is assumed to be transmitted.

4 CONCLUSION

It is more logical to introduce a safety margin, taking into account the risk and the total cost of alternative measures during temporary phases to increase reliability. This line of thought shows that lower safety factors should be used for temporary situations.

An analytical method based on the calculation of all internal forces is proposed; which describes the imperfections rather well. Despite the fact that the out-of plane bending moment is relatively small, it seems to have an important contribution to the calculation of imperfections.

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Cruz, Frangopol & Neves (eds)

Buckling resistance of steel bridge web during launching

L. Davaine & J. Raoul
SETRA, Paris, France

ABSTRACT: In France, the incremental launching by sliding or rolling is the most common method for erecting steel bridges. Nowadays it is usual to launch steel I-girders of a twin girder bridge (span up to 120 m long) or box girders (170 m long span in Millau viaduct). The transverse load is applied through the lower flange over a long length, and the web buckling under patch loading may govern the design of the steel structure. Such long spans need steel webs up to 4 or 5 m deep that are usually longitudinally stiffened. In France, longitudinal stiffeners of twin-girder bridges are mainly discontinuous and made of single flat plates.

Such very deep girders are of course impossible to test in laboratories. An extensive review of the literature has led to retain about 150 tests of stiffened girders under patch loading, but only a few of them have a depth slightly greater than one metre and almost all of them have a very short loading length.

That is why a finite element method has been used to build an additional database covering the domain of large bridges.

After a very precise calibration of the software against experimental results (type of finite elements, size of the meshing. . .), an extensive study of the influence of the initial shape imperfection has been performed. Due to lack of space these important steps are not developed in the paper.

The numerical data base has then been built to cover the full range of stiffened bridge girders:

- Depths from 2 m to 5 m
- Web thickness from 14 mm to 30 mm
- Flanges from 900 × 40 to 1200 × 150
- Loading length from 500 mm to 3000 mm
- Panel length from 4 m to 8 m

More than 300 numerical tests have been carried out.

The design of stiffened webs under patch loading is covered by EN1993-1-5:2005. This very new code deals with this buckling phenomenon in the same way as for other instabilities by using a $\chi-\lambda$ approach.

A first very important conclusion of the paper is that this approach is safe against experimental tests, and also against numerical tests.

A second conclusion is that this approach may be improved, in particular because the calculation of the parameter F_{cr} used to compute λ is based on the first buckling mode, which does not correspond to the final buckling mode.

That is why a new χ–λ method is proposed, in which all the parameters of the existing EN1993-1-5 method have been recalibrated:

F_{cr} is now evaluated as an interaction between the first buckling mode of the whole stiffened panel and the buckling mode of the isolated subpanel located just under the patch load and assumed to have hinged boundary conditions,

F_y has been slightly modified to fit better with the FE results of slender plates, λ has been fully recalibrated (together with a partial factor γ_M) on the basis of both experimental and numerical results.

Altogether these improvements lead to a much less conservative approach for the design of large bridge stiffened girders under patch loading during launching.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Safety sensitivity for temporary bridge erection conditions

R.G. Sexsmith
Department of Civil Engineering, University of British Columbia, Canada

ABSTRACT: The required safety for bridge erection conditions is often not specified in codes and standards, but is chosen by the contractor's engineer to suit special conditions. This paper illustrates choice of safety levels by optimization, considering consequence costs and uncertainties, and the sensitivity of the optimum safety levels to the various parameters that have to be estimated. The required optimum safety levels are surprisingly high compared with normal code values due to the typical situations in which erection equipment and demands on the structure are often very inexpensive compared with the costs of consequences of failure of the permanent works.

Temporary structures for bridge erection are characterized by three very important distinctions from normal permanent structures: First, the cost of the temporary item is likely to be significantly less than the cost of the permanent works that are supported, hence the consequences of failure are disproportionately greater than the cost of the construction of the temporary item; second, the temporary item is likely to be exposed to load demand for a very short period of time (months or one or two years), hence the expected value of the maximum environmental loading is less than for permanent works; third, the present expected value of the consequences is made up of contributions from only a very short time in comparison with that from a permanent structure, which is exposed for many years.

The first distinction should lead to higher than normal safety requirements, while the second should lead to lower load demand, lower values of load factors, or some similar reduction in the design strength (but not, as is common, a reduction in safety!). The third distinction leads to a significant reduction in total present expected value of consequence costs, hence a justification for reduced safety.

Engineering a bridge erection project, whether it involves temporary forces on the permanent works or design of temporary falsework, requires many safety decisions, and in most situations there is little guidance, since most bridge design codes focus on the permanent structure. Often the contractor's engineer has to choose the design criteria without code constraints. This is both a blessing and a possible curse, as it places great responsibility on the engineering team, but should provide scope for innovation and cost savings where appropriate.

In this paper, the specific examples are hypothetical, but they are based on the author's experience with specific projects in which the proposed methodology could have been used, and would have been cost effective.

There are many typical ways to provide appropriate safety to a temporary situation. Design can be carried out to the same criteria as for the permanent works. This is common, especially for small projects where custom criteria is not worth the effort of development. When a project is large, however, most designers use some strategy to recognize the temporary nature of the work. Commonly they may use higher allowable stresses or lower load factors. This has been very common for wind and lateral loads, but has been used even for dead load or selfweight stresses. Another approach is to use a lower return value for lateral (usually wind) load. For example, the Canadian Bridge Code (CSA 2000) suggests the use of ten year return wind for falsework. Presumably this is accompanied by the normal load factor for wind.

Such approaches recognize the reduced value of the expected maximum load effect in the case of wind, but seem illogical in the case of selfweight, which is certainly acting at its full value no matter how short the exposure time. However, there seems to be little or no recognition of the high consequences of failure relative to the cost of the item under design.

A logical safety criteria is to seek to minimize the present expected value of cost. Construction cost is expended at the outset. The remaining cost is the cost of consequences, converted using concepts of utility (Howard 1968) into expected value in monetary units (dollars) and discounted to the present time (time of the design decision). The discounting is what drives the required safety down if the time of exposure to load is short, while the consequences of failure drive it up if the construction cost is relatively small compared with the consequences.

The approach is illustrated by some examples of the choice of safety level for temporary erection conditions, with an examination of the sensitivity of the results to the parameters that require estimation. In general, most temporary works are relatively inexpensive compared with the consequences of failure, since the permanent structure is supported by the temporary works. In such cases the optimization approach leads to safety requirements more severe than those usually found in codes and standards for normal construction.

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Status and findings of current BHM applications in the world

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A methodology and decision support system for scheduling inspections in a bridge network following a natural disaster

K. Kepaptsoglou, M.G. Karlaftis & T. Bitsikas
National Technical University of Athens, Greece

P. Panetsos, & S. Lambropoulos
Egnatia Odos SA, Greece

ABSTRACT: Transportation networks are the backbone of modern societies; commuting, freight transportation, leisure travel are mainly accommodated by highways. A natural disaster may disrupt highway operations and therefore multiple community functions, which have to be rapidly restored. In addition, it creates needs for immediate emergency response (relief services etc). It is a fact that transportation networks are "lifelines"; both emergency response and quick restoration of community functions rely heavily on the ability of transportation networks to handle traffic. Unfortunately, transportation infrastructure elements such as bridges and tunnels are highly prone to damages caused by natural disasters (for example earthquakes). Bridges are probably among the most expensive and complex constructions of a transportation network. On the other hand, bridges are vital links within a transportation network; their failure to operate may lead to long bypasses and inability to access communities.

Following a natural hazard, the condition of the transportation network elements must be assessed and damages have to be identified. Inspections are therefore necessary, immediately after the catastrophic event. Specialized crews must be dispatched and inspect critical elements of transportation infrastructure such as bridges. The objective of the current paper is the scheduling of bridge inspection crews following an earthquake. A model and a decision support system, parts of a bridge management system currently developed in Greece are presented, which are designed to aid local authorities in optimally assigning inspectors to the bridge network.

The procedure followed for scheduling inspections consists of two major steps:

- Identifying the area affected by the earthquake, using a special software package.
- Assigning inspection crews to the bridges included within that area, taking into account initial reports on probable network link failure (bridge collapses, pavement failures etc).

A brief description of the procedure has as follows: After establishing the area affected by the earthquake and the corresponding part of the transportation network, reports are gathered from the local authorities (police, fire department etc) on possible failures of transportation elements. These reports provide information on possible network links that cannot be crossed by inspection crews. According to that information a new plasmatic network is formed (having bridges as nodes) that takes into account possible access difficulties. Availability of crews and the point within the area where the inspection crews start from are also defined. The decision support system already incorporates estimates on travel times between bridges and bridge inspection times. Using a heuristic procedure (Clarke – Wright "savings" algorithm), crews are optimally dispatched to bridges.

The decision support system (DSS) is incorporated to a BMS under development for a Greek Motorway Authority (Egnatia Motorway Authority, www.egnatia.gr). All information on the original network (the road network structure and positions of bridges) is stored in the BMS database. Additional information stored is related to the inspection time estimates for each bridge. Other information pre-included concerns the possible starting positions of inspection crews. Data imported

directly by the agency include (a) average travel speeds, (b) the number of crews available, (c) the starting point, (d) the part of the network affected by the earthquake and (e) possible failures in network links through a graphical user interface. The DSS transforms the network and provides the necessary results using the "savings" heuristic. The results include the bridges that have to be visited by each inspection crew and the total time needed by each crew for inspecting its assigned set of bridges.

Inspection of bridges following an earthquake is vital for the quick restoration of the transportation network. The current study presents a methodology and a corresponding decision support system that can facilitate agencies in quickly scheduling emergency aftershock inspections. The tool is based upon accurate estimates on affected transportation regions and a widely used algorithm scheduling. The decision support system is currently in use by the Egnatia Motorway Authority in Greece.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Continuous monitoring of concrete bridges during construction and service as a tool for data-driven bridge health monitoring

D. Inaudi & B. Glisic
SMARTEC SA, Manno, Switzerland

ABSTRACT: Bridges are omnipresent in every society and they affect its human, social, ecological, economical, cultural and esthetic aspects. This is why a durable and safe usage of bridges is an imperative goal of structural management. Measurement and monitoring have an essential role in structural management. The benefits of the information obtained by monitoring are apparent in several domains. First, it helps to improve and enlarge the knowledge concerning structural behavior and makes accurate calibration of numerical models describing and predicting this possible behavior. Thus, project and construction can be optimized in structural and economical aspects. Second, permanent monitoring can give early indications of structural malfunction. In this way, safety measures can be considered in time, and intervention on the structure can be performed immediately and with minimal economic losses.

The lifespan of a bridge starts with construction. Followed by testing of the bridge and most importantly the service period. During service, the structure may be refurbished, strengthened or enlarged, according to necessities. Finally, at the end of usage, the bridge can be dismantled. Monitoring during each period of the bridge lifespan is important and can give rich information allowing a better understanding of structural behavior and consequently better planned and less expensive management. In this paper, a brief introduction to the monitoring process is firstly presented. Then, the importance of the monitoring of each period of a bridge's life is examined step by step, and illustrated with a relevant practical application examples.

1 INTRODUCTION

The monitoring (or auscultation) of structures involves recording of time dependent parameters during certain periods. These parameters are related to the construction material (concrete, steel, timber, etc.) and to the structure itself. In both cases they can be physical, mechanical or chemical.

The life of a concrete bridge starts with construction – pouring of concrete. Follow curing of concrete, testing of the bridge and most importantly the service period. During service, the structure may be refurbished, strengthened or enlarged, according to necessities. Finally, at the end of exploitation, the bridge can be dismantled. Monitoring during each period of the bridge lifespan is important and can give rich information allowing a better understanding of structural behavior and consequently better planned and less expensive management.

In the next paragraphs we will explain generally and through examples, importance and benefits of monitoring performed during each phase of the structure life.

2 MONITORING

Monitoring is usually carried out in order to achieve one or several goals: structural and asset management, increase of safety, knowledge improvement.

There are different approaches to assess the structure and we can classify them in three basic categories: static monitoring, dynamic monitoring and system identification and modal analysis, and these categories can be combined. Each category is characterized by advantages and challenges and which one (or ones) will be used depends mainly on structural behavior and goals of monitoring.

Each category can be performed during short and long periods, permanently (continuously) or periodically. The schedule and pace of monitoring depends on how fast the monitored parameter changes in time. For some applications, periodical monitoring gives satisfactory results, but information not registered between two inspections is lost forever. Only continuous monitoring during the whole lifespan of the structure can register its history, help to understand its real behavior and fully exploit monitoring. The investment in the maintenance of the structure, using periodical inspections as a mean of control, can exceed the cost of a new structure.

The importance and benefits of continuous monitoring span the different phase in the lifetime of a bridge: during construction of a new bridge, after refurbishing, strengthening or enlargement of bridge, during testing of bridge, during service of bridge, during dismantling of bridge.

3 EXAMPLES OF CONTINUOUS MONITORING

The paper presents two application examples of long-term monitoring for a cluster of high-rise buildings in Singapore and for a bridge in New Mexico, USA.

Continuous monitoring allowed the registering of relevant structural parameters including all phases of the structure life. The benefits of whole lifespan monitoring of bridges are presented in this paper. They reflect through better planned and less costly structural management, increase of safety and improvement of knowledge concerning real structural behavior. The whole lifespan monitoring calls for sophisticated monitoring systems, which performances satisfy safety, technological, economical and esthetical aspects, being easy to use, fast to install, durable, reliable, stable, independent from human intervention and insensitive to external influences.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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The current status of SHMBM engineering

S. Sumitro
Smart Structures, IL, USA

M. Tominaga
Tommy ACE, Yokohama, Japan

Y. Kato & T. Okamoto
Keisoku Research Consultant, Hiroshima, Japan

ABSTRACT: Most of the infrastructures which were constructed in the 20th century such as bridges, dams and tunnels, are facing crisis life-span attainment due to their structural and functional degradation. Especially, this tendency is strong to the infrastructures which were built from concrete and steel material. It can be considered as an engineer's dedicated task and in the same time also as a social request to continue performing proper maintenance and finding way to extend the life span of such deteriorated infrastructures.

There is a close resemblance between aging human society and deteriorating of existing infrastructures, and both of them may be categorized as the most serious public problems in human-beings civilization. Although Structural Health Monitoring Based Maintenance (SHMBM) Engineering has been proposed previously, but various new technology including hardware and software has intruded into this engineering field. In this paper, a practical innovative monitoring technology so called elastomagnetic sensory technology is reappraised and arranged systematically with a structural health monitoring system to make them more practical use in future utilization of SHMBM engineering. Furthermore, some application examples of the innovative new monitoring technology are reported as well.

Distributed Bridge Health Monitoring System consists of a main server, substations, sensors, and high-speed data transmission network as shown in Figure 1. Sensors installed on the bridge are grouped locally and connected to a related substation at the local area through optical fiber, copper cables or wireless network. After collecting and pre-processing data from each sensor, the substation sends the data to main server through the network.

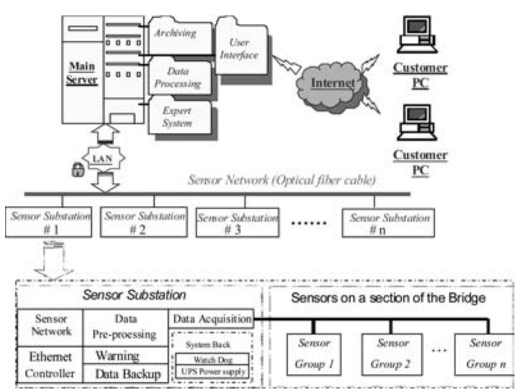

Figure 1. Schematic diagram of bridge health monitoring System.

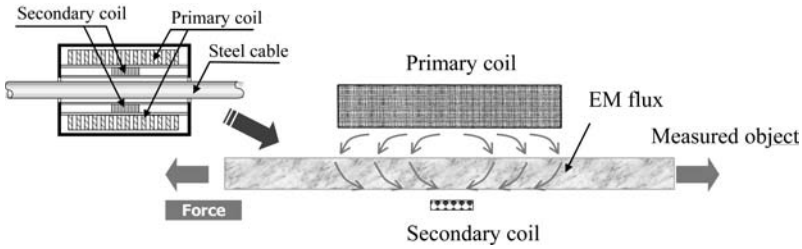

Figure 2. Monitoring mechanism.

The EM sensor enables easily to measure changes of permeability with regard to the known load state, in the most cases the zero stress state. The primary winding magnetizes the cable and the change of the magnetic flux induces voltage in the secondary coil as shown in the Figure 2. The real sensor is the steel structure itself. The high tensile low carbon steel is very suitable for EM method application (Sumitro 2005). By observing numerous field measurement results, it is confirmed that EM sensor is a non-destructive, no-contact, easy to operate measurement system to measure actual stress of steel wires, bars and cables. From the long-term field-application experience, it is confirmed that the EM sensory technology is reliable, accurate and generally applicable to many structural monitoring situations, even when other methods are inapplicable. Therefore, EM measurement technology is apparently suitable for developing a long-term health monitoring system for grasping actual stress of any steel-made infrastructures.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

GNSS for bridge deformation: Limitations and solutions

X. Meng, G.W. Roberts & A.H. Dodson
Institute of Engineering Surveying and Space Geodesy, The University of Nottingham, University Park, Nottingham NG, UK

C.J. Brown
School of Engineering and Design, Brunel University, Uxbridge, UB, UK

ABSTRACT: Satellite Systems R&D at the Institute of Engineering Surveying and Space Geodesy (IESSG) dates back to the 1970s. The use of Global Navigation Satellite Systems (GNSS), particularly the Global Positioning System (GPS) for bridge deformation monitoring started in the early 1990s and still forms a major theme of research activities at the IESSG. In recent years, joint studies with Brunel University in using GNSS for monitoring the deflection of the Humber Bridge and the London Millennium Bridge, and computational tools such as Finite Element (FE) models for the predictions of structural dynamics have been carried out, aiming at a better understanding of structural performance under a variety of active loadings such as wind, traffic, temperature and tidal current. A recent project, "*A Remote Bridge Health Monitoring System Using Computational Simulation and GPS Sensor Data*" has been conducted in collaboration with Cranfield University and industrial partners such as Leica Geosystems Ltd, W S Atkins, Pell Frischmann and Railtrack (now Network Rail). The overall objective of the project was the creation of a system employing advanced computational tools such as FE model coupled with GPS, triaxial accelerometers, pseudo-satellites (pseudolites) and other sensors to remotely monitor the health condition of operational bridges without the need for on-site inspection. This paper addresses the limitations of GPS based bridge deformation monitoring systems according to the knowledge acquired by the authors in their practice of more than 15 years. Relevant solutions are recommended to help different users fully understand the advantages and disadvantages of this modern positioning technology. The paper is organized as follows. The first section is a brief introduction and is followed by a discussion section of the major limitations of GPS for bridge deformation monitoring. The section Solutions for Tackling Problems discusses the potential improvements of the GPS based monitoring system performance through different approaches such as filtering techniques and hybrid sensor systems. The subsequent sections are some suggestions for future work and the paper conclusions.

In summary, the main points of this paper include following contents. To enable a GPS receiver on an operational bridge to conduct carrier phase based positioning, clear lines of sight to a minimum of five well distributed satellites and one (or more) reference receiver(s) at a nearby location, with precisely known coordinates, are required. However, this observation condition cannot always be met in the context of bridge deformation monitoring. Signal obstruction and reflection caused by any superstructure, surrounding topography, or even passing vehicles make the direct extraction of structural dynamics from the noisy output (normally in the form of continuous 3D coordinate and/or velocity time series) very difficult. Different de-noising procedures or algorithms have been developed over the years by the authors and high positioning accuracy has been achieved. Residual GPS signal delay caused by the atmosphere, and also the height differences between bridge and reference stations, also limit positioning accuracy. Appropriate mitigation models such as adaptive filtering have been studied. To increase the integrity and reliability of the whole monitoring system multiple, networked, GPS reference receivers, as well as hybrid sensor integration approaches have been extensively employed. Streaming positioning solutions from each monitoring node to a control centre for further updating of an FE model or diagnosis of the bridge performance is

a very important aspect of the monitoring system. The current design of RTK GPS hardware only allows one-directional communication. An open Internet based data streaming technique has been proposed and a prototype monitoring system has been developed during the project. For the successful use of GPS for bridge monitoring and evaluation of structural condition, the authors believe that mutual understanding and seamless interfacing between the surveyors and structural engineers is of utmost importance. Otherwise, this already proven modern technology will not be exploited to its full potential for bridge deformation monitoring.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Development of a bridge management system for a freeway authority in Greece

M.G. Karlaftis, T. Bitsikas & K. Kepaptsoglou
National Technical University of Athens, Greece

S. Lambropoulos & P. Panetsos
Egnatia Freeway Authority, Greece

ABSTRACT: The Egnatia Freeway is a major transportation corridor, crossing northern Greece from the Adriatic Sea to the Turkish Border and connecting Europe with Asia Minor. With a length of 700 km and more than 650 bridges it is considered the largest transportation project ever undertaken in Greece. The Egnatia Freeway authority, responsible for supervising the construction of the project, and managing and maintaining the freeway, has initiated a project for developing a bridge management system (E-BMS) with the objectives of:

- Monitoring the freeway's bridges and evaluating the condition of their elements,
- Allocating funds for bridge MR&R activities,
- Protecting bridges against potential seismic events and
- Handling aftershock actions after a seismic event.

The E-BMS structure is shown in Figure 1:

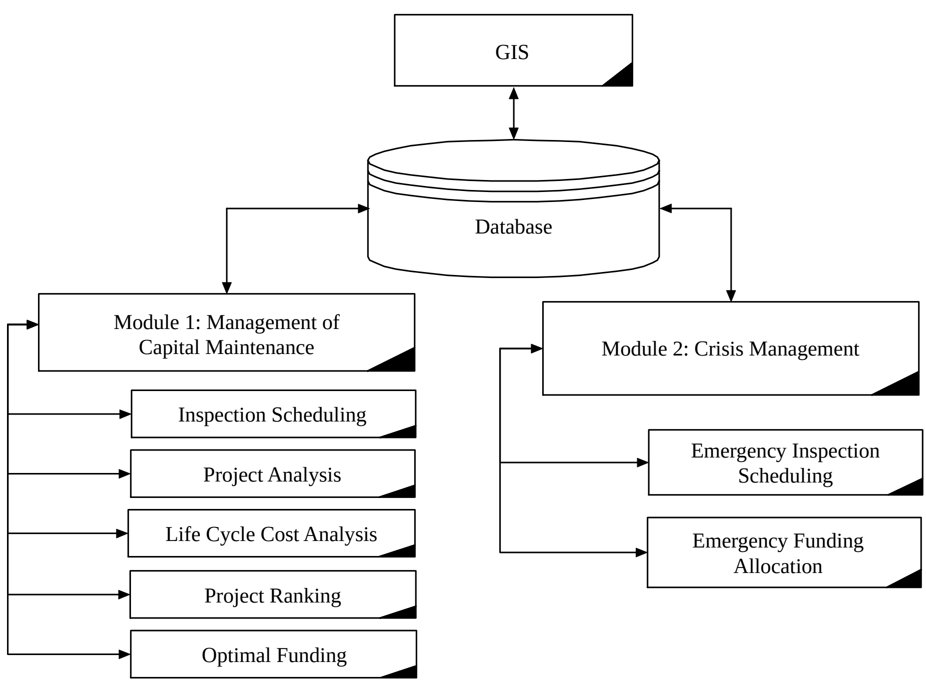

Figure 1. E-BMS Structure.

The core of E-BMS is an extensive ORACLE™ based database, containing information for all 1700 bridges and culverts. The database is interconnected to a GIS system, allowing user-friendly extraction of the database information. Various information fields are included in the database such as information on bridge position, ownership, road classes over the bridge, geometric characteristics of every bridge, detailed structural characteristics of every bridge, traffic and operational characteristics, condition ratings, inspection history, inspection, capital maintenance history, photos and as-built plans.

The E-BMS consists of two primary analysis tools: (a) a pro-active tool designed to handle capital maintenance management of the bridge network and (b) a re-active tool used to manage the bridge network after unexpected events. Both pro-active and re-active modules have incorporated seismic considerations.

The capital maintenance management module is used to support capital repair needs identification and optimal allocation of resources. It follows a bottom-up approach (project to network level), similar to that of the Indiana Bridge Management System – IBMS. The module can (a) propose repair actions in a time-horizon, (b) perform bridge life-cycle analysis, (c) rank repair projects according to several criteria and (d) optimally allocate funds for capital maintenance. The final outcome is a program of repair needs and corresponding cost estimations for a given horizon, with the objective of optimizing total authority and user welfare.

Of particular important was the development of a module that would aid decision makers in their actions towards making the freeway operational, after a severe unexpected event such as an earthquake. It was decided that these actions would include (a) dispatching inspectors to the bridges affected by the event for assessing after-event bridge condition and (b) optimally allocating funds for emergency repairs.

While E-BMS is based upon well-established and tested knowledge, it contains several novelties such as:

- A hi-tech, extensive database, linked to a GIS, dedicated to the bridge network.
- Both a proactive and reactive approach in bridge management decision making.
- Earthquake considerations in capital maintenance management and more specifically to:
  - Project Decision Making
  - Life-cycle analysis
  - Project Ranking
- A module to assist decision making after unexpected events.
- Analysis and optimization models based on metaheuristics (genetic algorithms).
- Special considerations in project decisions related to expansion joints and bearings.

The Egnatia Freeway authority is probably the largest highway authority in Greece and the Balkans. Its extensive bridge network led to the need for developing a customized bridge management system, adapted to the special conditions met in the freeway. While the BMS is based upon existing knowledge and practices, innovations have been applied in an effort to capture special conditions met in the Egnatia Freeway authority. The system is currently under testing and in the following years is expected to be used as the primary decision making tool of the authority on capital bridge maintenance.

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Monitoring performance of the Tamar suspension bridge

D.I. List & R. Cole
Tamar Bridge and Torpoint Ferry Joint Committee

T. Wood
Cornwall County Council, UK

J.M.W. Brownjohn
University of Sheffield, UK

ABSTRACT: The Tamar Bridge, a truss deck suspension bridge with main span of 335 metres was, when opened in the UK in 1961 the longest in the UK. Structural assessment in 1994 indicated that the deck had to be replaced and the stiffening trusses strengthened.

The final design incorporated a lightweight steel main deck, with additional cantilever decks at each side, while other significant changes included additional stay cables, extra longitudinal girders and rearrangement to the deck bearings.

1 INTRODUCTION

The Tamar Bridge forms a vital transport link over the River Tamar between the county of Cornwall and the city of Plymouth. The bridge is owned, operated and maintained by the two local authorities, and has relied solely on toll income to cover all capital and operating costs.

The original bridge (Figure 1) was designed by Mott Hay and Anderson as a conventional suspension bridge with symmetrical geometry, having a main span of 335 metres and side spans of 114 metres, and with anchorage and approach spans the overall length is 642 metres. Unusually for a suspension bridge of this era, the towers were constructed from reinforced concrete, and have a height of 73 metres with the deck suspended at half this height. The towers sit on caisson foundations founded on rock.

Main suspension cables are 350 mm in diameter and each consists of 31 locked coil wire ropes, and carry vertical locked coil hangers at 9.1 metres centres.

Figure 1. Tamar Bridge under construction.

Figure 2. Bridge section at midspan before and after strengthening and widening.

The main cables are splayed at anchorages and anchored some 17 metres into rock. The stiffening truss is 5.5 metres deep and composed of welded hollow boxes. The original 3-lane deck, spanning between cross trusses, was of composite construction with a 150 mm deep reinforced concrete slab on five universal beams surfaced with hand-laid mastic asphalt 40 mm thick.

When opened in 1961, Tamar was the longest suspension bridge in the UK and the first to be built since World War 2. It was initially carrying approximately 4000 vehicles a day with a maximum gross weight of 24 tons, but in the late 1990's, after nearly four decades of use, it was found that the Tamar Bridge would not be able to meet a new European Union Directive that bridges should be capable of carrying lorries up to 40 tonnes. Since restricting use by such vehicles would damage the local economy, the bridge needed to be strengthened or replaced.

The appointed consultant (Acer, Now Hyder) proposed replacement of the main deck with a lightweight orthotropic steel deck, and having investigated a range of traffic diversion options, proposed construction of temporary relief lanes cantilevered off the bridge truss, to act as a supplementary diversion route while the main deck was being replaced. As the design developed, it soon became apparent that the permanent addition of cantilever lanes offered a cost-effective improvement in terms of both capacity and safety, as well as satisfying the temporary diversion requirement, although such permanent widening would require primary legislation.

This paper first describes the bridge structure and the upgrade modifications, then describes the monitoring implemented during the project for tracking the bridge performance and evaluating the safety of the completed structure. Finally some recent experiences and future developments in performance monitoring are described.

2 MONITORING

The Structural Monitoring System (SMS) installed by Fugro Structural Monitoring has been used to monitor cable loads, structure and environment temperatures and wind speed and profile. The purpose of the SMS was and is to provide engineering information on the performance and condition of the bridge during and after the widening and strengthening gather data. In particular it was used track deck profile and cable loads during the strengthening works.

3 PRESENT PERFORMANCE

Present performance is satisfactory, but in addition to the periodic level surveys, the operators (Tamar Bridge and Torpoint Ferry Joint Committee) are interested to know about absolute tower and deck movements. In addition, a number of the additional cables have exhibited oscillations during strong winds, sometimes with rain, and the nature of these oscillations is of interest. The effect of semi-permanent water-butt dampers needs to be checked to see if they offer a permanent mitigation strategy. Hence some additional sensors will be added to form a parallel real-time monitoring system in early 2006.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Lesson learned from monitoring of long-span cable-supported bridges

Y. Fujino & D.M. Siringoringo
Bridge & Structure Laboratory University of Tokyo, Tokyo, Japan

ABSTRACT: Many large bridges in Tokyo metropolitan area were designed and constructed before the 1995 Hyogo-ken Nanbu (Kobe) Earthquake. The near-field ground motions measured during the 1995 Kobe Earthquake were extremely severe and much larger than that specified in the bridge design codes at that time. The bridge seismic design code was later revised to prepare the ground motions experienced in Kobe, and accordingly seismic retrofit program of bridges has also started.

In a context of retrofitting, evaluation of performance of existing bridges plays an important role. It is largely understood that while dynamic characteristics of a bridge may be modeled analytically, the real behavior can only be validated from measured data. Fortunately, in the last decade many large and important bridges in Japan were instrumented with permanent sensors. Hence, valuable data about bridge response to various types of motion such as ambient, traffic and strong motion were recorded and analyzing records from the past events allows us to capture the real performance of a bridge. Using a systematic analysis procedure, global behavior of a bridge during various types of loading can be investigated. In addition, with a dense-array sensor arrangement the performance of local components can also be studied.

Two strategies of bridge monitoring using vibration response are presented in this paper, namely: ambient vibration measurement and strong motion measurement. Ambient measurement has gained popularity recently due to its convenience in measuring vibration response while the structure is under service loading. This is viewed as an effective method to frequently assess structure's performance during its service life. In ambient vibration measurement, loading could be from wind, waves, vehicular or pedestrian traffic or any combination of service loading. This offers economical advantage since it eliminates the need for loading testing devices and also does not require the suspension of structure services during measurement. In addition to ambient excitation, strong motion data acquired during seismic events that frequently hit Japan, also provides excellent opportunities to gain insight into behavior of bridges and performance of their components. In fact, due to its large excitation level, some phenomena that do not appear during ambient measurement might appear during seismic excitations.

This paper presents a review on cases study of monitoring of instrumented bridges using these two excitations. The focus of case studies is to show that monitoring can be a useful tool for structural evaluation that goes beyond the mere of modal identification or sensor deployment issues. The work described in this paper involves: (1) case study of structural identification using inverse analysis retrieved from ambient measurement of long span suspension bridge, (2) instrumentation and identification of three long span bridges in Tokyo Metropolitan area using strong motion records, and (3) performance evaluation of link-bearing connection of Yokohama-Bay Bridge based on identification results.

It is shown that with a systematic identification technique, monitoring data can be utilized as tools to understand the real mechanisms and also to discover any deviation or unwanted mechanism that might hinder the structure from fully functioning. These are important features for future structural health monitoring. The lesson learned from each case study is hoped to give a better understanding that might lead to a promising future application for bridge assessment and monitoring.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Cable hanger plate replacement; a case study on Bosporus Bridge

A. Turer, A. Caner & C. Yilmaz
Middle East Technical University, Civil Engineering Dept., Ankara, Turkey

ABSTRACT: Fatigue at steel bridge structures is one of the major problems that often times do not reveal itself until later stages of damage. Design of fatigue prone members or connections is vital especially for non-redundant fracture critical elements. Cyclic traffic and/or wind loads on bridges can further aggravate stress variations at connections. Bridge members or connections that reach their fatigue life or get damaged due to other reasons may become difficult to replace or rehabilitate especially for suspension bridges. This paper addresses cable hanger plate fatigue problem of a suspension bridge, Bosporus-Istanbul, and also discusses alternatives for hanger plate replacement procedure that will not result in disruption of the traffic. Transfer of cable tensile load to the bridge through alternative paths is necessary for replacing the cable connection plate. Four alternatives are generated for load transfer: a) bypass by supplementary 'V' cables connected to the original cable, b) in-plane cable transfer, c) direct connection to the main suspension cable, and d) inverted 'U' shape bypass frame for improved stability. Cable strain measurements that will be taken during the load transfer process would allow measurement of axial load existing on cables. The strain based measurement of cable force can be compared against vibration based measurement to calibrate for the unknowns such as i) cable boundary conditions, ii) cable bending rigidity, and iii) cable inclination. Furthermore, measurement of cable strains during cable plate replacement enables comparison of axial load on the cable prior to and after the connection plate replacement. Permanent instrumentation of cables would allow health monitoring of cables and determination of traffic-temperature loading versus cable force relations.

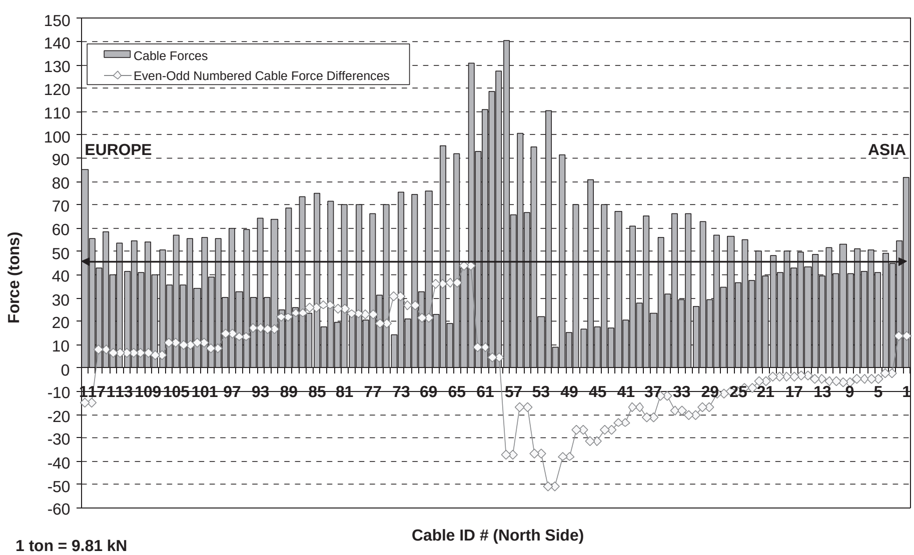

Figure 1. North side cable forces and cable force differences.

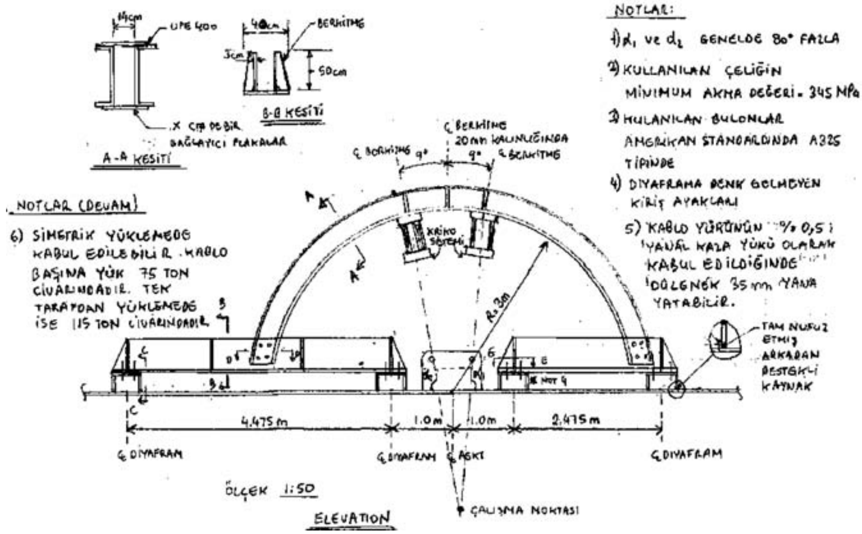

Figure 2. Hand sketch of Mobile Arc Framing for Cable Grabbing Device.

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Structural identification of constructed systems and the impact of epistemic uncertainty

F.L. Moon & A.E Aktan
Drexel University, Philadelphia, PA, USA

1 INTRODUCTION

The ASCE recognizes that civil engineering practice is falling short of delivering products and services to satisfy the societal needs for transportation, water, power and energy delivery, and sustainable constructed systems (Committee Report, 2005). Paradigms such as performance-based civil engineering, health-monitoring and integrated asset management offer great improvements in the engineering and managing the constructed environment. However, there is one major knowledge barrier standing in the way of effectively leveraging such innovative paradigms, this is a lack of our ability to reliably predict the mechanical characteristics, loading environment and behavior of constructed systems at various critical limit states of performance. The writers' experience and a recently conducted in-depth literature survey has reiterated what many civil engineers have learned through anecdote, i.e. the epistemic uncertainty that governs constructed systems and their interactions with nature and human systems makes it very difficult to develop complete scenarios involving their performance and to predict their reliability. Epistemic uncertainty introduces a significant risk in any decision related to the constructed environment that most engineers and managers are often unaware of, especially when we try to design for sustainability and project conditions and performance of a system into the distant future. Unfortunately, no modern design, operation or maintenance code for constructed systems, while many of these are explicitly framed by the structural reliability theory, recognizes that epistemic uncertainty may render any stipulation of (time-based) structural system reliability greatly suspect. In the case of performance in the operational, serviceability, durability and security limit-states, epistemic uncertainty especially stands as a major barrier to meaningful management.

The writers contend that the only way civil engineers can reduce epistemic uncertainty related to the performance of constructed systems is by system identification studies on existing constructed systems and infrastructures that have varying ages, and that have demonstrated both good and poor performance under various limit-states of operation, serviceability and durability, safety, and performance under extremely rate hazards. Structural identification, a subset of system identification, aims to characterize the mechanical characteristics, load distribution paths and the kinematics of complete soil-foundation-structural systems reliably (Aktan and Yao, 199?). During (1997–2000), an ASCE Committee explored the state-of-the-art in structural identification, producing a draft report. This Committee is revived in 2005 with the writers taking the lead, and now aims to collect data and information on structural identification applications in the field, and interpreting this data and information for a better understanding and therefore mitigating the risk due to epistemic uncertainty prevailing in constructed system behaviors.

2 STRUCTURAL IDENTIFICATION

According to Doebling et al. (2000) structural identification (St-ID) can be defined as "*the parametric correlation of structural response characteristics predicted by a mathematical model with analogous quantities derived from experimental measurements*". In this paper we will adopt this definition and aim to discuss how various uncertainties motivate and challenge the St-ID of constructed systems. First, it is important to recognize two classes of uncertainty: aleatory (due to natural randomness) and epistemic (due to our inability to correctly understand, model or predict reality). Given that epistemic uncertainty in essence represents a lack of knowledge (i.e. known-unknowns and unknown-unknowns), in general, it is not realistic to expect to bound it within a quantifiable confidence interval such as is commonly employed to address aleatory uncertainty. However, unlike aleatory uncertainty, it is possible to reduce epistemic uncertainties by the generation of knowledge that informs and reduces unknowns. In general, St-ID is motivated by the need to quantify aleatory uncertainty and reduce the epistemic uncertainty associated with the behavior of constructed systems.

In the case of aleatory uncertainty, its non-deterministic nature allows random variable representation, and thus traditional statistical and probabilistic approaches are well-suited and have been exploited to address this challenge. In the case of epistemic uncertainty however, the resulting errors may be deterministic in nature (Ang and De Leon 2005) and thus a similar approach is not appropriate. For example, consider that even under service conditions, constructed systems have many nonlinear (e.g. cracking, local yielding, visco-elastic bearings, soil, etc.), non-stationary (e.g. slippage, intermittent contacts and friction in connections, movement systems and boundaries, temperature and humidity effects, etc.), and non-observable (e.g. soil-foundation interaction, intrinsic forces, etc.) behaviors, some of which violate the fundamental assumptions of all current St-ID techniques. The severity with which such assumptions are violated affects the resulting level of epistemic uncertainty associated with the identified model. In the best situations, the effect of this uncertainty will be minor; however, it should still be approximated and made transparent, particularly to decision-makers. In the worst situations, this uncertainty may be so severe that an identified model may bear little resemblance to the real structure.

The authors believe that the significance of this challenge has resulted from a lack of appreciation for the inherent distinctions between constructed systems and their manufactured counterparts. Overcoming this challenge will require that the St-ID approaches developed for manufactured systems be adapted to explicitly recognize and address the unique attributes of constructed systems. An example of this type of development is the recent advances associated with operational (or output-only) modal analysis, which recognizes the cost and difficultly of performing forced vibration tests on large constructed systems (Brownjohn et al. 1992, Fujino et al. 1999, Wenzel and Pichler 2005, among others). In addition, several researchers have developed St-ID approaches that explicitly address aleatory uncertainty (due to natural randomness), which can be significant for constructed systems (Bucher et al. 2003, Yuen and Katafygiotis 2002, Beck and Katafygiotis 1998, Beck 1990). However, while these advances are relevant, the distinctions they address are far from exhaustive, and thus a wider, sustained effort is necessary. As a first step, civil engineers must leverage the centuries of accumulated heuristic knowledge to identify and communicate the principal distinctions and unique challenges related to the St-ID of constructed systems to the wider community.

The objective of this paper is to, through the use of selected past research investigations, illustrate how various uncertainties both motivate and challenge the St-ID of constructed systems. The paper will discuss common sources of uncertainty related to constructed systems as well as a brief overview of the state-of-the-art in St-ID. This will be followed by descriptions of selected studies that illustrate the potential of St-ID to both reduce epistemic uncertainty and quantify aleatory uncertainty related to constructed systems. In addition, various sources of uncertainty that continue to hinder the reliable implementation of current St-ID approaches will be discussed and illustrated through the use of past studies. The paper will conclude with a discussion of the authors' vision for future research on this topic.

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Service life prediction based on permanent output only monitoring

H. Wenzel
VCE Holding GmbH, Hadikgasse, Vienna, Austria

ABSTRACT: Several permanent monitoring systems in bridges have been installed over the past years. The analyses of this data allow service life prediction after the compensation of environmental influences and several data fusion processes. A breakthrough has been achieved in both the frequency analysis as well as time domain analyses.

The paper explains how the periodic report is automatically generated by the system and is distributed to the operator of the bridge and the lifecycle management of the system. A practical example concentrating on the Europabrücke in Austria, a 40 year old record breaking steel bridge will be provided.

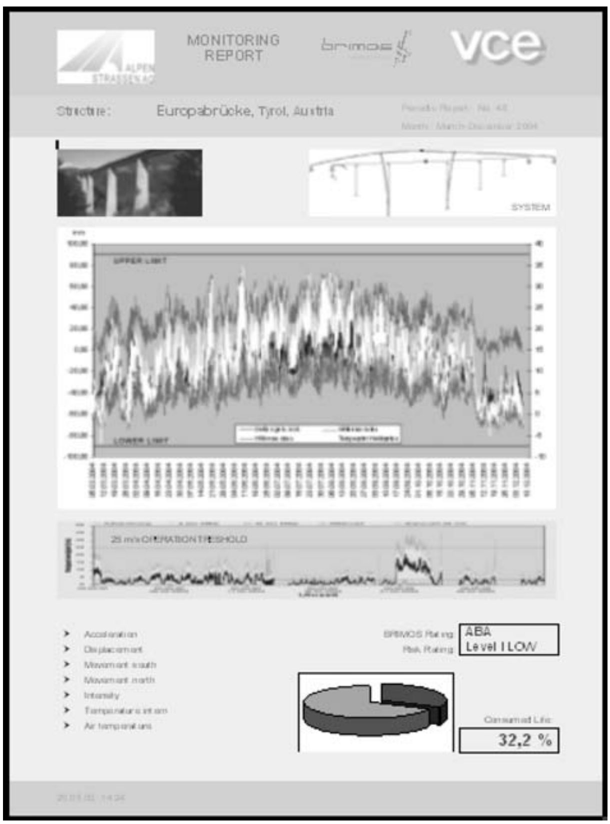

Figure 1. Periodic Report of Europabrücke.

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Steel stringer bridge load rating based on field calibrated grid models

A. Turer
Middle East Technical University, Ankara, Turkey

ABSTRACT: Bridge load rating is commonly performed to assess the level of load carrying capacity of a bridge under standard loads. Load rating process involves a certain level of analysis of the bridge structure commonly using nominal sectional properties. However, on site conditions are generally different than those listed in the blue-prints and variations in structural properties such as support conditions, section or composite action losses, deterioration due to aging, salting, corrosion etc should also be considered for load rating of a bridge. One of the best ways to conduct a load rating that would consider current condition of a bridge involves non-destructive testing and structural identification based approach. In this paper, a two dimensional grid model updating approach based on field measured data and related rating studies are explained. The proposed grid model has advantages of better simulating bridge geometry and loading conditions as compared to the 1D beam models. Model updating or calibration process is not only based on collected dynamic data (mode shapes, modal frequencies, and order of modes) but also utilizes static deformed shape information (BGCI). Usage of deformed shape and dynamic data together imposes advantages on structural identification. Optimization routines for calibration are also improved by grouping variables at different stages of calibration improving the chance and speed of finding the global minimum of objective function. Usage of 2D grid based modeling, calibration, and rating program developed under MATLAB language is demonstrated using a concrete-deck-on-steel-stringer example bridge.

Grid models carry the advantages of modeling the transverse direction especially for skewed bridges. Short analysis duration with small number of optimization parameters which is suitable for iterative-automated optimization programming and powerful numerical capability for calculation of bridge load rating from member rating coefficients are additional advantages of 2D grid based bridge rating. The bridge load rating using field calibrated 2D grid models is a better alternative to 1D simplistic load rating tools since the bridge geometry and properties are defined better, and field measurements are used. Automation of calibration and load rating is an advantage over 3D-FE models which are more difficult to construct, manipulate, and calibrate.

The example application using a sample bridge shows that the bridge rating factors have a tendency to decrease when the calibrated grid model is used for rating. A lower rating factor obtained for the calibrated model may be an indication of deterioration during the 37 years of the sample bridge's life span or improperly calculated and initially assigned grid member structural properties due to the discontinuous geometry introduced by the grid model.

Inspection and prediction of structural performance

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Bridge condition and health measures for needs analysis

P.S. McCarten
Opus International Consultants, Napier, New Zealand

ABSTRACT: Road Controlling Authorities (RCA) have recognised the need to have transparent and robust information supporting requests for maintenance and improvement project funding. Bridge managers recognise decisions for allocating funds for bridge maintenance and improvement projects require different emphasis and hence different bridge management information and performance measures. This paper focuses on the funding needs for maintenance and rehabilitation.

Accurate financial forecasting for bridge maintenance/rehabilitation needs is dependent upon the experience and knowledge of the bridge management team, the Bridge Inspector and the Bridge Structural Engineer. The reporting of these needs in a robust manner to convince funding authorities is difficult when the inputs are subjective. Hence it is desirable to have condition data that shows the change in condition that can occur as a result of different funding investment levels. It is also fundamental to report to all 'asset stock holders' that necessary structure reliability is achieved throughout the structure life.

Ideal asset management monitors changing condition trends with time to ensure cost effective maintenance management intervention is applied. If asset managers can use the condition monitoring data to reliably predict the bridge condition response to varying bridge maintenance and rehabilitation funding investment then they can more confidently determine the funding needs. The monitoring techniques also offer the RCA a mechanism to report their achievements. With the multitude of demands and expectations Bridge Engineers are becoming reliant on bridge management systems (BMSs) for analysis of the data generated and the associated monitoring and reporting. With an underlying concern for the robustness and reliability of condition data collected in a bridge inspection programme analysis of trends is desirable. This paper presents the results of a study into condition inspection data collected over a nine year period for a sample of 40 bridges in British Columbia, Canada, under the Ministry of Transportation (BCMoT) control with the aim to identify trends in condition vs maintenance/rehabilitation funding investment.

Typical bridge design ensures there is reserve structural capacity available immediately following construction allowing some deterioration to take place before the structure transitions into the 'critical deterioration' phase. The management of the structure through the initial phase and to identify when the critical deterioration phase has begun is dependent upon the bridge inspection programme implemented. It is generally accepted that once a structure has deteriorated into the 'critical deterioration' phase close monitoring using Non Destructive Testing and Evaluation (NDE) procedures is necessary to ensure the funding needs are known with more certainty.

Having well trained and qualified bridge management and inspection personnel is fundamental to effective bridge management. The focus of the bridge management team in the initial period of deterioration is to develop an understanding of the as-built structure, identify the high risk elements (indicators of structure criticality) in it and monitor the high risk components to ascertain when the RCA expected level of reliability is likely to be compromised. Having an early understanding of the structure reliability raises the opportunity to apply proactive preventive maintenance strategies.

While RCA's have been considering several types of performance measures this paper focuses on the BCMoT Bridge Condition Index (BCI) and Stock Condition Index (SCI) as applied to the systems method for best bridge maintenance management practice. The SCI is the numerical average of the BCI for each age group of bridges after each inspection cycle. Caltrans has recently developed the Bridge Health Index (BHI) and this is briefly reviewed in the paper. While the field

data collected for the BHI is similar to that for the BCI the index analysis is quite different, with the differences being in the number of condition states and the application of the weights, where:

- BCI uses a ‘fixed’ number of condition states, condition state factors and component weights, with the weights based on rational risk considerations
- BHI uses a number of different condition states depending on the type of component, applying different condition state factors depending on the type of component and using ‘variable’ component weightings depending upon the economic value of the respective components in the bridge

Compared to the BHI the BCI is considered more ‘pure’ in representing bridge condition, it views the bridge as a structural system and offers more opportunities for statistical analysis and trend reporting. These advantages are explored in the paper through a case study.

The case study involves 40 bridges in a coastal environment under the management of BCMoT. This sample size is considered statistically significant and the analysis is expected to have a precision better than 15% at a 95% Confidence Interval. Nine years of inspection records have been used. While there were changes to the bridge management team the bridge inspector remained unchanged through this period. The bridges in the study group are of mixed materials, with precast concrete or structural steel beams and a cast insitu concrete deck predominating, the group includes three steel trusses, a RC arch and a pedestrian overbridge. The bridges have mixed ages and through the period of study no bridge age exceeded 60 years.

Throughout the nine year study period the bridges were subject to maintenance and rehabilitation and hence there were positive and negative changes in element, component, bridge and stock condition. The focus of this study was to investigate trends between the Funding Investment and Bridge Condition by retrospectively analyzing the condition data received from BCMoT.

From the analysis undertaken the slope of the SCI trend line was identified as the most appropriate condition indicator and while it has no intrinsic value it reflects the rate of structure deterioration. Due to limitations in available maintenance and repair cost data rehabilitation cost information was sourced from the bridge management team and normalized to the replacement value (RV) for the 40 bridges to determine the Bridge Rehabilitation Investment (BRI).

When the BRI and the Condition Indicator data were compared there was good correlation throughout the nine year period investigated. The study suggests BRI funding should ideally approximate 0.5% RV, and be at least 0.3% RV. This level of BRI funding correlates well with that used for international ‘best practice’ bridge maintenance/rehabilitation. The study group SCI trends also correlated well with expected prototype performance, the SCI Targets used by BCMoT and the actual BCMoT rehabilitation programme.

While the study is limited in its size the results are meaningful and give confidence to using information gathered from a well set up inspection program. The method uses the SCI as set out in the systems method for best bridge maintenance management practice. The method is best suited to the ‘zone of tolerable deterioration’, specific bridge ‘needs’ are required when critical deterioration has occurred and when changes in levels of service are required the ‘needs’ must be evaluated with other network priorities. The importance of using experienced Bridge Engineers to forecast funding ‘needs’ cannot be overstated.

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Prediction and analysis of deterioration of Moscow Bridges

G. Brodski
AGA Group, Inc., Chestnut Hill, MA, USA

Yu.A. Ponomarev & Yu.A. Yenutyin
GUP "Gidromost", Moscow, Russia

ABSTRACT: The main objective of a bridge management system (BMS) is to ensure optimal planning of repair and rehabilitation activities and to establish a budget for it. Within most BMS, this objective is attained on the basis of deterioration models, which describe wear of structures and their elements in the course of time in quantitative terms, using both probabilistic prediction methods, considering only one of them for all structural elements.

The analysis suggested below attempts to verify the adequacy of deterioration models, previously adopted on the basis of experts' judgments. The analysis is based on the experimental data obtained since 2002, when the BMS of the City of Moscow was implemented. During this time 1,059 facilities (bridges, tunnels, pedestrian bridges, embankments, etc.) containing in total over 17 millions of 215 types of standard elements (SE) has been inspected.

An analysis of inspection results has indicated that there is no significant correlation between the age of a given facility and its technical condition index. This absence of correlation usually attributed to the validity of presentation of a structures deterioration in the form of Markovian process. We believe, however, that the results obtained should be linked to the fact that selective repair activities compensate for different rates of bridge wear, which depends, to a substantial degree, on the following three main factors: quality of manufacture of an element/bridge; various mutual effects of SE on each other; degree of protection of a structure against external impacts.

The law of condition changes has been determined as a trendline for a set of points on the graph of Condition state versus time. It has been assumed that the baseline point is the year of construction or rehabilitation of a bridges/SE, if any. The actual values of wear intensities have been determined in agreement with a procedure for adaptation of baseline data, by computing an actual residual service life to failure and a corresponding rating coefficient. The projected wear of every type of SE is analyzed by the methods of mathematical statistics, with special analysis of the cases where the wear parameters went beyond classified intervals.

The results obtained suggest a conclusion that strength deterioration (fatigue) of most bearing or frame structures does not become apparent even during periods compatible with or exceeding the service life prescribed by regulatory deterioration models. In an overwhelming majority of cases, deterioration of bearing and frame structures has been caused by wear of material due to corrosion of reinforced concrete and steel, which led to decrease in their working cross-sectional area. The reported scatter of condition categories is attributable mainly to variations in the properties of material and its protection. Another cause of the scatter can be traced to the occasional, infrequent and unpredictable emergency situations, such as collision of vehicles with bridge structures.

For SE referring to construction materials, scatters of condition states are significantly more pronounced. This can be explained by variations of parameters of particular materials (for example, different grades of strength, frost-resistance and impermeability). These initial properties dependent on both the manufacturing conditions and the quality of secondary protection under a given operational environment.

For elements with a limited service life, e.g., for expansion joints, the regulatory exponential deterioration models are very close to reality.

Based on the results obtained, it is suggested to subdivide all SE into 4 groups:

a. SE, the deterioration models of which can be assigned on the basis of mean values obtained in the process of bridge operation without taking into consideration specific design features or specific conditions of a region, like SE "Filled movement joint". This model can be determined as "Exponential";
b. SE, the deterioration models of which can be assigned on the basis of mean values obtained in the process of bridge operation but should be adjusted taking into account specific design features of a bridge and/or specific conditions of a given region, as well as operating conditions, for example, SE "Reinforced concrete ledge" and most of the materials like "Steel" or "Reinforced concrete". Deterioration model for these SE had been determined as "Fan-shape" one because of corresponding look on the graph;
c. SE, deterioration of which is caused only by accidental situations, and repairs of which should be predicted based on the probability of accidental failures, such as SE "Steel beam". The model can be determined as "Zero", because BMS would not predict any regular deterioration for such SE;
d. SE, deterioration of which depends on wide array of factors, which cannot be specified based on current knowledge. For these elements, for example, "Reinforced concrete column", stochastic deterioration model, characterized by transition probabilities would be the best.

According to the experimental data, the largest group is the second one. Thus, in order to obtain a relatively reliable prediction, one should assign the parameters of fan-shaped deterioration model individually for each SE, considering its quality, as well as the mutual influence of adjacent elements. This can be resolved by applying to each SE a specific "quality" and "influence" coefficients. The quality coefficient may be used not only for considering specific properties of a given SE or objectivity of a given inspection but also as a criterion for prescribing certain repair actions, e.g., provision of protective coating or replacement of an element with a new one of better quality.

It is also important that experts' judgment referring to "rated durability" for most SE was excessively pessimistic and corresponded to minimal values within the given life span obtained on the basis of inspection findings. Real longevity of SE on the Moscow bridges exceed the judgment almost twice (87% on average). An assessment of the durability using the "lower limit" criterion significantly affected the objectivity of the prediction in the process of the BMS operation.

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Bridge deck deterioration: A parametric hazard-based duration modeling approach

P.M. Christofas & M.G. Karlaftis
National Technical University of Athens, Athens, Greece

ABSTRACT: Infrastructure Maintenance and Rehabilitation (M&R) decision making, either at the network level or at the project level, is based on current (measured) and future (predicted) infrastructure facility conditions. Therefore, accurate predictions of future infrastructure conditions are essential for effective M&R decision-making. Infrastructure condition is often represented by discrete ratings. For example, for bridge decks, the FHWA bridge-rating system is used most commonly. Bridge inspectors employ ratings of 0 to 9, with 9 representing near-perfect condition (FHWA 1979). The use of discrete representation of facility condition makes it necessary to develop discrete models of deterioration (Mauch and Madanat 2001).

The deterioration of a concrete bridge deck is a stochastic process that varies widely with several factors, many of which are generally not captured by the available data. Therefore, probabilistic models are used to predict the deterioration of infrastructure facilities such as bridge decks. In this paper, we present duration models that predict the time between changes in condition-state for reinforced concrete bridge decks. To accomplish this, we used hazard based duration models that incorporate parametric methods. Hazard based duration models predict *changes* in condition over time as a function of a set of explanatory variables and are used to compute infrastructure transition probabilities. The remainder of the paper is organized as follows. Section 2 reviews the available data set and the basic specification for the hazard based duration model. Section 3, examines model specification issues and the estimation results for both the Log-Logistic and the Weibull hazard based duration models. The final Section summarizes the findings of this research.

The data set used in this paper is part of the Indiana Bridge Inventory data base. It consists of approximately 5,700 state owned bridges from Indiana, and is a subset of the National Bridge Inventory (NBI) data base. The data set contains inspection records from 1978 through 1988. The condition evaluation rates the condition of the major bridge components, e.g. deck, superstructure, substructure, and so on, on a scale from 0 to nine, where bridges with a 0 rating are in the poorest condition and those with a rating of 9 in the best condition. The dependent variable of interest in our models is the time spent by a bridge deck in a given condition-state (variable name is 'time-in-state' (TIS)), extracted from consecutive inspection reports (earliest inspection year in the database was 1974 and most recent 1996).

The class of parametric failure time models estimated in this paper are also known as the *accelerated failure time* (AFT) models. What is actually estimated, in practice, is a model rather similar to an ordinary linear regression model. In a linear regression model it is typical to assume that the error term (ε_i) has a normal distribution with a mean and variance that are constant over i, and that the εs are independent across observations. One member of the AFT class, the *log-normal* model, has exactly these assumptions. Other AFT models allow distributions for ε other than the Normal (such as the extreme value, log-gamma, and logistic), but retain the assumptions of constant mean and variance, as well as the independence across observations. If there were no censored data, the parametric survival models could be estimated using Ordinary Least Squares (OLS). But, survival data typically have at least some censored observations, and these are difficult to handle with OLS. As a result, Maximum Likelihood is used to estimate the parameters.

The basic thinking behind duration modeling is to examine whether the longer a bridge deck remains in the same condition state the *more* likely it is that it will drop one or more condition states within a specified time period.

As is well known, the sign of coefficient estimates indicates the ‘direction’ of the relationship between the independent and dependent variables. The negative coefficient for AGE indicates that aging bridges drop to a lower condition state at higher rates than do newer bridges, as was expected. Interestingly, the magnitudes for the coefficients are, per se, informative as reported; but, with a simple transformation, these estimates lead to some interesting and intuitive interpretations. For a 0–1 variable such as WEARSURF, taking e^{β} (in the Log-Normal model) yields the estimated ratio of the expected (mean) survival times for the two groups; in the model presented above, for example, $e^{-0.544} = 0.52$. Therefore, controlling for other covariates, the expected time in a state for bridges with no protective systems is 52% lower than bridges with protective systems. For a quantitative variable such as AGE, the transformation $100(e^{\beta} - 1)$ is used, giving the percent decrease in the expected survival time for each one-unit increase in the variable. Thus, using this transformation for age, each additional year of age for an interstate and primary road bridge is associated with a 51,74% decrease in expected time in a state, holding other covariates constant. It is interesting to note that, for a number of variables such as *REGION* and *AVGADT*, the statistical analysis performed in this paper did not indicate a statistically significant relationship between these variables and TIS for a bridge deck. This result, although surprising at first, clearly indicates that the deterioration phenomenon is complex, with a number of interrelations affecting statistical analyses; this suggests that further refinement of the models is required and that non-parametric approaches that generally do not suffer from multicollinearity or make restricting assumptions regarding the parametric form of the underlying deterioration mechanism may be a promising avenue for future research.

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Bridge inspections, a case for trained bridge inspectors

B. Kamya
Jacobs UK Ltd, Aberdeen, United Kingdom

ABSTRACT: The inspection of bridges on site depends a lot on individual inspector's judgement. This dependence on the interpretation of the site defects by the individual bridge inspector is the basis of the review of this practice in this paper. This site stage belies the potential to create variable results where the level of training and qualifications of the individual bridge inspectors lack consistency.

The inspection practice in United Kingdom UK is reviewed. The review considers how the inspection of highway and railway bridges is managed. Highway bridge inspections are managed directly by the relevant local authorities or contracted out to management agents. The railway bridge inspections are contracted out to consultants, Dean (2005). The inspection of bridges approach in the United States of America USA, Canada, France and South Africa is briefly reviewed. The review compares the similarities and differences in the approaches.

In the UK the development of the inspection techniques from the initial publication of the Department of Transport DOT's Bridge Inspection Guide in 1983, Allison & Woodowrd (2005) to the more recent Highways Agency Design Manual for Roads and Bridges (DMRB) design and advisory standards are considered, Highways Agency (1993, 1994a, 1994b, 1997, 2001). Other UK standards considered are the Network Rail Standards of competence for examination of structures Network Rail (2001). The UK inspection technique as implemented through the manuals and standards is compared with techniques from the other countries. In the USA the comparison is made with the requirements of the American Association of State Highway and Transportation Officials (AASHTO) and Federation of Highway Administration (FHWA), Manual for Maintenance Inspection of Bridges and National Bridge Inspection Standards (NBIS). The comparison with the practice in Canada is based on details in CN Standard Practice Circular (SPC) 4000, Inspection of Steel, Timber and Concrete Bridges. In France and South Africa the comparison is based on procedures by Service d'Etudes Techniques des Routes et Autoroutes (SETRA) and South Africa Roads Infrastructure respectively.

The UK, USA, Canada, France and South Africa all have a formal approach to bridge inspections. There are variations in the approaches which depend on the type of inspection and interval between each type of inspection. The similarities are that in the UK a General inspection is similar to routine inspection in the USA, a visual inspection in Canada and continuous surveillance in France. The Principal inspection is similar to an In-depth inspection in the USA, a detailed inspection in Canada and organised surveillance in France. There were also similarities with the UK's Special Inspections and Accidental inspection. With the exception of the UK, other countries had formal qualification and training requirements as par of their procedural requirements for all their bridge inspectors. The UK Network Rail has a similar approach of using approved structure examiners.

It is proposed that this curious lack of a formal qualification and training requirement for bridge inspectors in the UK, except for Network Rail, compared to the USA, Canada, France and South should be reviewed. In support of this proposal are examples are given of the benefits to be gained from using qualified and certified inspectors, Duffy (2005), Nordengen, Welthagen & Fleuriot (2005), Kruger & Ronny (2005) and the importance of hiring qualified bridge inspectors in the effective operation and management of the inspection program, Wisconsin Department of Transport (2001).

As materials and methods of construction evolve, bridge inspectors inevitably face a growing challenge to keep up to date with the essential skills to perform the inspection tasks. This paper also proposes that in view of the benefits of training bridge inspectors, the current practise of depending on peer review, in comparison with the USA, Canada, France and South Africa practice of training and certifying bridge inspectors' merits consideration. This paper does not set out to prove that countries employing certified bridge inspectors save more time and money in their bridge management. What the paper sets out to demonstrate is that the USA, Canada, France and South Africa have a practice with benefits and it could contribute to good bridge management in the UK. For that reason further research is recommended to look at effects of using qualified and trained bridge inspectors on the costs in bridge management systems.

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Correlation between reduction in load capacity and structural condition of highway bridges

D. Gurenich & W. Robert
Cambridge Systematics, Inc., Cambridge, MA, USA

ABSTRACT: The evaluation of whether a bridge can carry the loads which it is intended to carry is a fundamental consideration in determining an agency's bridge needs. Systems such as the Federal Highway Administration (FHWA) National Bridge Investment Analysis System (NBIAS) and the American Association of Station Highway and Transportation Officials (AASHTO) Pontis Bridge Management System (BMS) consider bridge load carrying capacity when evaluating overall bridge investment needs. However, a major limitation of the approach used by NBIAS and Pontis is that the load carrying capacity of the bridge is assumed to be static; in simulating future conditions no adjustment is made to the load rating as a result of predicted deterioration.

It is widely acknowledged the load carrying capacity of a bridge is reduced as a bridge ages and deteriorates. A significant body of research exists on the impact of truck weight and traffic on bridge load capacity, including a recent report and accompanying software on the effect of truck weight on bridge costs (Fu, et al. 2003). Given that the bridge inspections compiled through U.S. National Bridge Inventory (NBI) provide a wealth of data with respect to physical conditions measured using NBI condition ratings, as well as of the age of existing bridges, it would be desirable to relate predicted future loss of load capacity (measured in terms of the load rating of the bridge) to bridge deterioration in order to improve the predictive ability of existing bridge management systems. However, there exists no consensus on the quantitative relationship between load ratings and bridge conditions or age.

The research described in this paper was performed to attempt to quantify the relationship between load rating and either condition ratings or age in order to develop approaches for improving the modeling approach in NBIAS. The research was performed using NBI data for U.S. bridges obtained in 2004, representing bridge conditions in the U.S. states, District of Columbia and Puerto Rico as of 2003. These data were collected as detailed in the FHWA NBI Coding Guide (FHWA 1995). An analysis was performed to evaluate the statistical relationship between load capacity (measured using the inventory rating, NBI Item 66) and five independent variables: age of the bridge (measured in years since the bridge was constructed or reconstructed using NBI Items 27 and 106); design load (NBI Item 31); and NBI condition ratings for the bridge deck (NBI Item 58), superstructure (NBI Item 59) and substructure (NBI Item 60). The regression analysis was conducted on a set of over 468,000 bridges, including all bridges in the NBI but excluding culverts.

The primary causal variable for inventory rating was found to be the design load. The correlation coefficient r between inventory rating and design load was greater than 0.55, and was considered to be significant. Thus, the approach used for the remainder of the analysis was to perform a series of linear regressions between the remaining independent variables and the dependent variable load rating. The regressions were performed for each design load category, for the entire bridge population as a whole, ignoring the design load parameter.

The results show that inventory rating is correlated positively with the deck, superstructure and substructure condition ratings, and negatively with bridge age. For each of the independent variables, the correlation with inventory rating is weak. In all cases the correlation coefficient r is always greater when the regression is performed for all design load categories than for an individual category. In all cases the correlation coefficient was low.

Taken in the order of their increasing explanatory power represented by the correlation coefficient, predictive variables of the estimated regressions range as follows: deck rating ($r = 0.27$), age ($r = 0.28$), superstructure rating ($r = 0.37$), substructure rating ($r = 0.38$). It should be observed that these values were obtained for the models that were estimated across all design load categories, correlation coefficients of the regressions estimated for the individual categories are one to two orders of magnitude lower.

The research described here indicates that load carrying capacity is poorly correlated with NBI condition ratings and bridge age. Though one might expect these to be correlated, confounding between the independent variables and bridge design load, as well as potential issues that may result in increased variability in load rating and condition ratings, largely mask any such relationship in the NBI data analyzed. Further research should be considered to determine the link between bridge conditions and load capacity, and to develop approaches for modeling the potential loss of load carrying capacity in bridge management systems.

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Bridge inspection and monitoring

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Durability in B.O.T. bridge projects

F. Branco, J. Ferreira & M.M. Branco
Instituto Superior Técnico, Technical Superior Técnico, Technical University of Lisbon, Portugal

ABSTRACT: Governments, all over the world, are using the B.O.T. (Build, Operate and Transfer) solution to finance large infrastructure projects, when public budgets are not available. As, in this system, the technical aspects of design, construction, operation and maintenance (during the concession period) are developed by the concession company, after the signature of the concession contract, the quality of the final infrastructure needs to be very well defined during the initial tender phase. In fact, afterwards, all the missing aspects will definitely lead to the most economical solution.

In this paper, illustrated with the B.O.T. tender process of the Vasco da Gama bridge (one of the longest in Europe), the main technical aspects that must be considered in the tender specifications are presented. These include the discussion of the service life, the implications on safety and durability, the definition of the construction quality procedures, the monitoring of the operation, the implementation of inspection and maintenance plans and the reception conditions at the end of the concession.

1 INTRODUCTION

Due to the permanent shortage of public funds, governments, all over the world, are using the B.O.T. (Build, Operate and Transfer) solution (or other similar public-private systems) to finance all types of large infrastructure projects. In this solution, with the initial financing of private banks, a concession company designs, builds and explores the infrastructure. The private investment is recovered during the concession period with the exploration of the infrastructure. At the end of the concession the infrastructure is transferred back to the Government authority. Examples of this financial idea can be found in history since many centuries ago but it began to be currently used in the last decade of the XX century to finance the construction of roads, bridges, airports, hospital, power plants, etc., due to the significant profits that can be obtained from these public infrastructures.

As the technical aspects of design, construction, operation and maintenance (during the concession period) are developed by the concession company, after the signature of the B.O.T. concession contract, the quality of the final infrastructure needs to be very well defined during the initial tender phase. In fact, afterwards, all the missing aspects will definitely lead to the most economical solution and the final quality of the infrastructure will be reduced.

In this paper, illustrated with the B.O.T. tender process of the Vasco da Gama bridge (one of the longest in Europe), the main technical aspects that must be considered in the tender specifications are presented. These include the discussion of the service life, its implications on safety and durability, the definition of the construction quality procedures, the monitoring of the operation, the implementation of inspection and maintenance plans and the reception conditions at the end of the concession.

These aspects are related to activities of the authorities during the project life, beginning with the organization of the tender, follow-up of the design, control of the construction quality and control of the operation and maintenance phase, leading to a quality system in BOT projects.

2 TENDER AND TECHNICAL SPECIFICATIONS

To prepare a B.O.T. tender the initial studies must clearly define the financial model and this is usually associated to:

a) Definition of the parameters of operation
b) Definition of the concession time by volume or time

After the definition of the financial model the technical specifications for the infrastructure must be developed for the tender (Branco 2004). The main problem to prepare the tender of a B.O.T. project is that the technical specifications must be written defining the quality to be achieved during design, construction and service life of an unknown design of the future infrastructure.

Usually this is implemented by imposing only the application of existing technical codes during design and construction. Unfortunately this is not enough in this type of projects, because codes are mainly prepared for design phases controlled by the owner, where it is easy to impose alterations. In B.O.T. projects the following technical aspects need to be additionally considered in the tender specifications: i) Related to the design phase; ii) Related to the construction phase; iii) Related to service life.

3 CASE STUDY – THE VASCO DA GAMA BRIDGE

The Vasco da Gama Bridge consists of a 12 km crossing of the Tagus River in Lisbon, being one of the longest bridges in the world. In this paper the bridge is taken as a case study and the following aspects were considered: i) Traffic and deck width; ii) Structural service life; iii) Design structural actions; iv) Construction procedures; v) Operation and maintenance.

4 CONCLUSIONS

In B.O.T. projects, the activities of the authorities during the project life must consider the organization of the tender, the follow-up of the design, the control of the construction quality and the control of the operation and maintenance phase.

In this paper, the technical main aspects that must be considered in the tender specifications to achieve quality were presented, illustrated with the case study of the Vasco da Gama Bridge. These aspects included the discussion of the service life, its implications on safety and durability, the definition of the construction quality procedures, the monitoring of the operation, the implementation of the maintenance plan and the reception conditions at the end of the concession.

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Post-mounted corrosion sensors, experiences and interpretation of data for use in service life models

R. Sørensen, T. Frølund & M. Sloth
COWI A/S, Kongens, Lyngby, Denmark

Corrosion monitoring systems is still relatively new. The first steps were taken in the late 80ties where build-in sensors for new reinforced concrete structures were developed.

In the late 90ties the principle from the build-in sensors were transferred to post-mounted corrosion sensors for existing structures. With these sensors the ingress of the "corrosion front" into the concrete cover is measured. This is especially useful for large marine bridges, where the operation and maintenance costs can be reduced significantly by applying a preventive repair strategy before deterioration commences. In contrary to conventional visual inspection, a corrosion monitoring system will provide the owner with detailed information about the current state of deterioration inside the structure. The corrosion monitoring technique will ensure detection of the critical initial stages of deterioration and unacceptable rates of deterioration can be detected at an early stage, allowing the owner to make cost-optimal maintenance decisions.

A standard corrosion sensor setup consists of: A piece of black steel (steel sensor), a piece of a noble metal (counter electrode) and a reference electrode placed in the cover to the reinforcement within a concrete structure and in direct contact with the concrete matrix. With these three electrodes among other macro-cell current, corrosion rate, half-cell potentials and electric resistance can be measured. When the black steel electrode is passive the macro-cell current will be close to zero due to the negligible difference in the electrode potentials, while a significant macro-cell current will be generated, if the black steel electrode is active corroding due to the difference in electrode potentials.

With a number of black steel sensors located in the concrete cover at varying depths the ingress of the "corrosion front" can be measured. When a trigger value at the outermost steel sensor is exceeded corrosion has been initiated. When there is a signal from the next outermost steel sensor this sensor has been activated and corrosion has been initiated in this depth, etc. see figure 1.

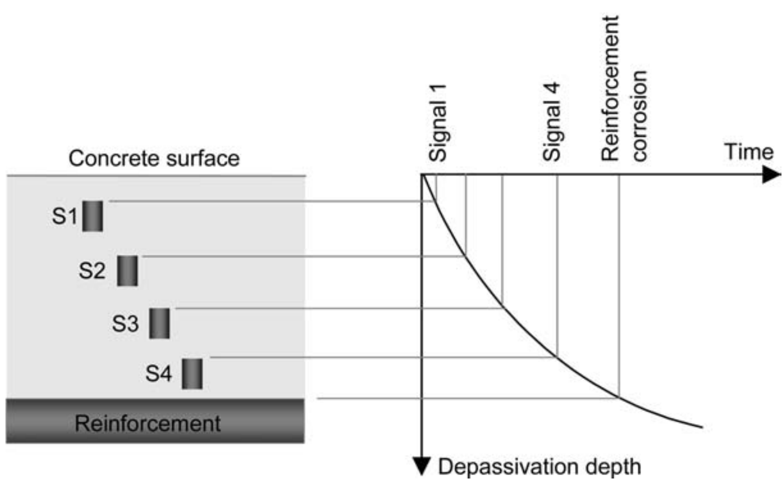

Figure 1. Principle of detecting the ingress of the "corrosion front" into the cover to the reinforcement.

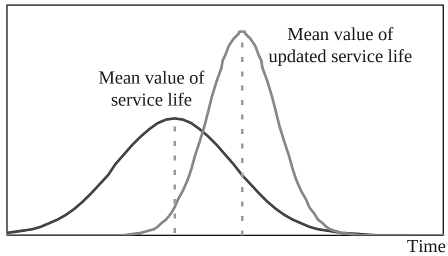

Figure 2. Density function for corrosion initiation time with or without information's from a corrosion monitoring system. The effect of updating with the result from the post-mounted sensors is that the expected average service life can be changed and furthermore that the uncertainty (seen as a more narrow density function) on the service life is reduced.

A methodology for estimating the residual service life of our existing structures is an essential parameter in rational operation and maintenance. Service life models for chloride ingress and carbonation are available. Modern probability-based models for evaluating the residual service life of structures need factual data of the key parameters of the ongoing chloride ingress. Data from post-mounted corrosion sensors can be included to update the ingress models (see figure 2) and to estimate more accurately the chloride threshold value for onset of corrosion. The observation of corrosion initiation at a given depth at a certain time can be used for Bayesian updating. It is also possible to include the complementary information that corrosion has not been detected at a certain depth at a certain time.

It is recommended that the service life models are based on a probabilistic model as just described, where the results from the corrosion monitoring system can be included on a rational basis via Bayesian updating taking into account the inherited uncertainty. Further information can be seen in Sloth et al. (2006). It is concluded that in order to reach a reliable service life prediction, data from post-mounted corrosion sensors have to be included. In fact the results from the corrosion sensors are contributing in two ways: It gives an indication of the *present* status of the "corrosion front" and it can be used as information for updating the service life model and thereby the prediction of the *future* condition state. The experience from using these sensors is still limited, but the results so far seem very promising.

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Bridge decks with GFRP – concrete composite sections

J.R. Correia, J.G. Ferreira & F. Branco
Instituto Superior Técnico, Technical University of Lisbon, Portugal

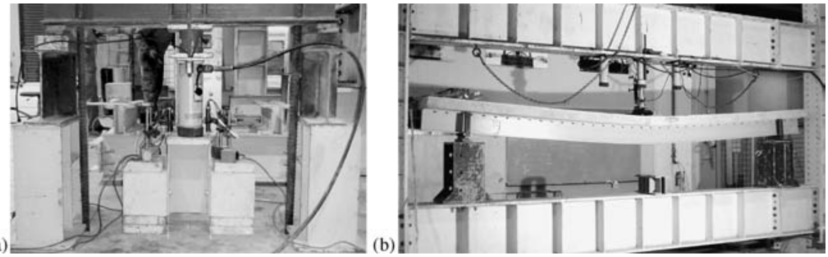
ABSTRACT: The development of bridges with improved durability and higher speed of construction led, in recent years, to the study of new structural solutions, where GFRP pultruded profiles are increasingly taking place, due to their strength, lower self weight and better durability. However, GFRP profiles are considerably flexible and susceptible to instability phenomena. To overcome these drawbacks, their use in GFRP-concrete composite sections of bridge decks may have a great potential, substituting the classical steel-concrete composite decks with lower maintenance and increased speed of construction. This paper presents the experimental study of GFRP-concrete composite beams under static behaviour, where two different shear connection systems were investigated: (i) stainless steel bolts; and (ii) a continuous epoxy adhesive layer. Shear connection tests were conducted on GFRP I-profiles connected to concrete, using both systems. The results of those tests were used to design and test simply supported beams with GFRP-concrete composite cross-section, showing the viability of this solution for bridge decks.

1 INTRODUCTION

In recent years, the maintenance/repair costs of bridges built with traditional materials increased very significantly, leading to the development of new structural solutions with improved durability and higher speeds of construction. A growing number of new bridges are being constructed as all-FRP structures, using glass fibre reinforced polymer (GFRP) pultruded profiles, replacing the more traditional steel and concrete materials, due to several advantageous properties, such as lightness, high specific strength, design flexibility, low thermal/electrical conductivity, ease of installation, low maintenance requirements and high corrosion resistance. The use of GFRP profiles in GFRP-concrete composite (or hybrid) structural elements presents a very interesting potential for bridge construction, substituting the classical steel-composite decks, with lower maintenance requirements and higher speeds of construction, specially if precast concrete elements are used. This paper presents results of an experimental research concerning the flexural behaviour of a new GFRP-concrete composite solution for bridge decks. Shear connection tests were conducted on GFRP I-profiles connected to concrete, using two different systems: (i) stainless steel bolts; and (ii) a continuous epoxy adhesive layer. The results of those tests were then used to design and test simply supported beams with GFRP-concrete composite cross-section, using both shear connection systems, showing the viability of this solution for bridge decks.

2 ANALYSIS OF GFRP-CONCRETE COMPOSITE SECTIONS

Based on the classical hypothesis for the analysis of steel-concrete composite elements, a set of equations for the design of GFRP – concrete composite beams, for both service and ultimate behaviour, was developed. For serviceability analysis, both partial and complete shear interaction situations were considered to compute deflections. For failure analysis, both partial and full shear connection situations were considered to compute ultimate strength.



(a) (b)

Figure 1. Experimental investigations: (a) Shear connection tests; (b) Flexural tests on a composite beam.

3 SHEAR CONNECTION TESTS

Three shear connection tests (SCS1 to SCS3) were performed to analyze the behaviour of the connection between a GFRP profile and concrete elements with different compositions, using steel bolts with different diameters (figure 1a) and one shear connection test (SCS4) was performed using an epoxy adhesive layer. Shear connection tests allowed for the definition of the steel bolts diameter and the longitudinal spacing of those shear connectors when placed on the composite beams. Shear connection tests also allowed determining the connections' flexibility and the ultimate strength of the composite beams.

4 FLEXURAL TESTS ON GFRP-CONCRETE COMPOSITE BEAMS

Three composite beams were produced and tested in bending (figure 1b): composite beam HB1 (using SCS1), was tested in a 4.00 m span under one point load at midspan; and composite beams HB2 (with SCS3) and HB3 (with SCS4), were tested in a 1.80 m span under two point loads 0.64 m apart and centred in the midspan. Based on the results of the flexural tests, the following subjects were analysed: deformability; ultimate strength; shear distribution; effectiveness of the concrete slab width; composite action and interconnection slip; effect of the slippage on the deformability and strength; accuracy of the developed analytical models.

5 CONCLUSIONS

Aiming to obtain an alternative solution to steel-concrete composite bridge decks, with lower maintenance and higher speed of construction, this paper presents results of the experimental study of a structural solution for bridge decks, combining GFRP pultruded profiles and concrete elements, where two alternative shear connection systems were used: (i) steel bolts and a (ii) continuous epoxy adhesive layer. The following main conclusions can be addressed:

1. GFRP-concrete composite beams, using both shear connection systems, are a viable structural solution that can be used for bridge construction, presenting reasonable stiffness, high strength, low self-weight and improved durability.
2. Comparing with the behaviour of simple GFRP profiles, GFRP-concrete composite beams show a considerable stiffness and strength increase, with a better use of the profiles' properties.
3. The ultimate strength and deflections of GFRP-concrete composite beams can be predicted with a good precision, using methods of analysis developed by the authors, where shear deformation and interconnecting slip are considered.
4. Comparison of the two alternative shear connection systems shows that a continuous epoxy layer provides a significantly higher stiffness than that obtained using steel bolts. Consequently, the use of the first shear connection system in composite beams provides lower deformations and higher ultimate bending moments.
5. When designing this type of composite structural elements, premature failures can be prevented using adequate constructive detailing, such as that constituted by the web compressive stiffeners placed at the end supports sections.

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Sensors in civil engineering infrastructures

J.G. Ferreira, F. Branco & M.M. Branco
Instituto Superior Técnico, Technical University of Lisbon, Portugal

ABSTRACT: Civil Engineering Structures had a significant evolution in the last decades, related to the construction of very large infrastructures (long bridges, tunnels, high rising buildings, etc.) where a safe and functional behavior, during a durable service life, are the principal targets to be achieved. Based on the experience acquired in the monitoring of large infrastructures it is here presented the current lay-out of sensors for bridge monitoring within a bridge management system. This lay-out is analyzed and ten ideas are presented for new sensors to be developed for civil engineering, towards a future intelligent structure.

1 INTRODUCTION

In Civil Engineering Structures had a significant evolution in the last decades, related to the construction of very large infrastructures (long bridges, tunnels, high rising buildings, etc.) where a safe, durable and functional behavior, during a long service life, are the principal targets.

In fact, the construction of large infrastructures, to reach lives over 100 years, imposes the implementation of follow up systems to detect any arising anomaly and to act with the associated correction procedure. These systems are presently being defined from the structure design phase (integrated with the structural and durability analysis) and aim to cover the structure life period.

These follow up systems evaluate the safety, durability and functional conditions of the infrastructure using on line monitoring and also visual/in situ testing inspection procedures, leading to information that is integrated in a global management system of the infrastructure, during construction and the service life. This information will allow the definition of the maintenance/repair actions and the optimization of the associated costs (Branco 2004).

The use of sensors in infrastructure monitoring has the advantage of allowing on line assessment, quick evaluation in accident situations and reduction of inspection costs. Besides the monitoring applications, sensors are also presently being used in structural solutions incorporating passive and active control.

In current follow up systems there is still a significant weight of the visual/in situ testing procedures, as there is a lack of sensors to perform the corresponding activities. In fact, several types of sensors to perform on line monitoring still need to be invented, namely to get a better analysis of the structural behavior or to analyze the durability problems.

Based on the experience acquired in the monitoring of large infrastructures it is here presented the current lay-out of sensors for bridge monitoring within a bridge management system. This lay-out is analyzed with a discussion of the evolutions needed, in terms of new sensors (ten ideas for new sensors) and management of results, towards an intelligent structure.

2 MONITORING DURING CONSTRUCTION

In this paper new types of sensors are foreseen regarding the following aspects of the construction phase: i) Evolution of the Material Properties (concrete strength); ii) Evolution of Structural

Characteristics (strain sensors with high durability and sensors for displacement over water); iii) The Reception Test; iv) Materials Durability (sensors for durability parameters).

3 MONITORING THE STRUCTURAL BEHAVIOUR

Regarding the structural behaviour evaluation two new types of sensors are presented, namely for bearing and for scour monitoring.

4 ACTIVE AND PASSIVE CONTROL

New types of sensors are also considered for active and passive control aiming to increase the reliability of these systems that are supposed to minimize damage in the structures during severe actions like earthquakes.

5 MONITORING THE DURABILITY

New sensors to control the material degradation like concrete carbonation or chloride penetration are presented aiming to monitor the durability.

6 MONITORING THE OPERATION

New techniques for operation control (traffic control, weight assessment, etc) are predicted in this paper towards more rational exploration procedures and control.

7 CONCLUSIONS

Monitoring is becoming part of the current quality control of construction and service life behaviour of main infrastructures. These follow up systems evaluate the safety, durability and functional conditions of the infrastructure using on line monitoring, leading to information that is integrated in a global management system of the infrastructure. This information will allow the quality control, the definition of the maintenance/repair procedures and the optimization of the associated costs.

Nevertheless, due to its limitation, monitoring is still complemented with visual/in situ testing procedures to evaluate the structure conditions. To reduce these "manual" procedures in a path to implement intelligent structures, ten new ideas for sensors to be developed were presented in this paper.

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Strain Checker: Stethoscope for bridge engineers

T. Ojio & K. Yamada
Nagoya University, Nagoya, Japan

Y. Saito & S. Shiina
Tokyo Sokki Kenkyujo Co., Tokyo, Japan

1 INTRODUCTION

Strain information is the one of the most important keys to understand the real behavior of existing structures. Thus, strain gauges are widely used for strain measurement. Strain gauges require bonding on the surface to be measured. The bonding and wiring procedure take times and cost of field works. Structural engineers have to decide their point of interest at their desk in advance. However, in cases of structural damage evaluation, it is quite difficult to decide where to measure because there are usually many uncertainties.

Frictional type strain gauge (See Figure 1) is pressed against the surface to measure the strain by friction produced at the interface. The authors reevaluated the advantages of the frictional type strain gauge when it was used in on-site structural diagnosis. The advantages are that structural engineers can measure strain in short time and diagnoses it on site. The authors enhanced mobility and usability by designing magnet attachment for frictional type strain gauge. It is called Strain Checker (See Figure 2).

2 APPLICATION EXAMPLES

The authors and the colleagues carried out case studies using Strain Checker, this paper describes the brief report of following studies.

– Search of neutral axis of composite girder: Kojima Viaduct
– Investigation of cracks over bearings of a steel box girder bridge: Shin-bashi bridge
– Durability evaluation of a steel arch bridge: Kirchenfeld Bridge, Switzerland

2.1 *Measurement of neutral axis of a composite girder bridge*

Kojima Viaduct is located 20 km to the east of Nagoya. The end span of the viaduct is a simple-supported composite steel plate girder bridge of 35m span. The authors searched neutral axis of

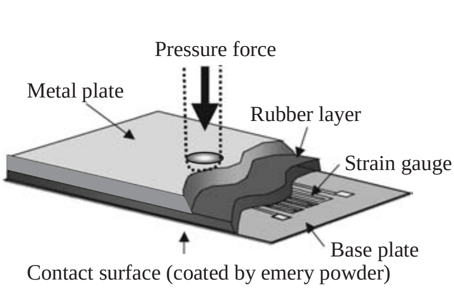

Figure 1. Frictional type strain gauge.

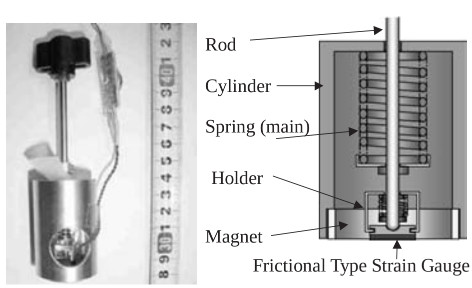

Figure 2. Strain Checker.

Figure 3. Search of neutral axis by Strain Checker.

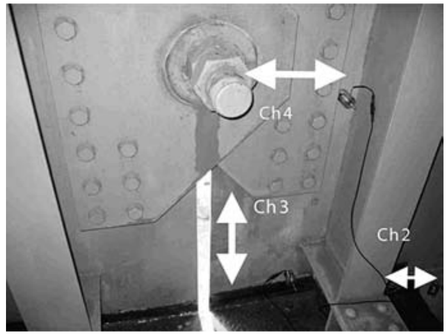

Figure 4. A fatigue crack investigation by Strain Checker.

Figure 5. Installation of Strain Checker by rock climbers.

Figure 6. Strain measurement in concrete deck (A steel plate was used to obtain average strain).

a side main girder in the end span by Strain Checker. Dynamic strain responses at four points in eleven sections were measured for five-minutes. The neutral axes were estimated by the strain distributions at maximum response of the lower flange. The estimated neutral axes were located on higher position than calculated ones by design plan.

2.2 *Investigation of fatigue crack on a steel box girder bridge*

In November 2005, fatigue cracks were found in ten bearings of Shin-bashi bridge. Strain measurement using Strain Checker was carried out on a side span as a preliminary investigation into the cause of the cracks. Measuring points were decided to check the structural behavior of main girder, the bearing, and connection plates. The dynamic response for a test vehicle with the gross vehicle weight of 250kN were measured by Strain Checker. The results show uplift force can be applied to the bearing by upstream traffic load.

2.3 *Durability evaluation of a steel arch bridge*

Kirchenfeld Bridge is located in Bern, Switzerland. It was built in 1883, and has two traffic lanes and two Tram lines. Strain measurement using Strain Checker was carried out for the verification of strain due to bus and tram. Rock climbers set Strain Checkers without scaffolding as shown in Figure 5. Strain Checkers were installed in steel members and concrete deck (Figure 6). The measured strain records were compared with computed ones to verify the assumptions needed for the analysis.

3 CONCLUSIONS

The Strain Checker can measure strain on painted steel surface or concrete surface without adhesive and paint removing. It does not require special skill for setting, and saves time on site. Engineers can decide where to measure strain and diagnose on site in short time. Three case studies show that Strain Checker is a stethoscope for bridge engineers.

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Health monitoring of structures using cement-based piezoelectric composites

L. Qin & Z. Li
Department of Civil Engineering, Hong Kong University of Science and Technology

ABSTRACT: Among the techniques used in sensors, piezoelectricity has been proved to be one of the most efficient mechanisms for most applications in smart structure. It is noted that the sensors suitable for application in other engineering fields, such as mechanical engineering, may not be applicable in civil engineering due to the differences in the properties between smart materials and the host structures. To meet the requirement of civil engineering structures, a 0–3 cement based piezoelectric ceramics composite had been developed. This composite was made by fine PZT powder and white cement.

The composite fabrication process included the following steps: PZT powder generating, composites forming, electroding, and polarizing. This cement-based 0–3 piezoelectric composite should be encapsulated before it could be embedded into concrete like an aggregate. Two methods, direct casting and sandwiched bonding methods were developed to make the sensors.

The current research focused on the electromechanical properties and mechanical properties of the piezoelectric composite at relatively low frequencies due to the special requirements of civil engineering structures.

The magnitude response was to show the relationship between loading magnitude change and electrical voltage output under certain frequency. The results indicated that mechanic-electrical dynamic response of cement-based piezoelectric composites matched a linear relationship very well.

The frequency response was the relationship between electrical output and variety of input frequency. There was a plateau among certain frequency range in the frequency–magnitude part. For phase angle, corresponding to the above plateau part, it changed linearly with the frequency approximately.

Several kinds of typical complex waveforms: square wave, complex sine wave and random wave, were applied to cement-based piezoelectric ceramic composite, and the mechanic electrical response signals were measured to examine the input/output consistency of composite. We could observe that the electrical signals were quite close to the mechanical signal, except some hyperfine oscillation.

After calibration, the sensor could be used in concrete structure. To measure the performance of such kind of sensor at structure level, test were carried out combining the cement-based piezoelectric ceramic sensors into concrete structures, such as beam and frame. We could compare the input signal (kobe, maximum acceleration is 0.34 g) and the output. The maximum stress, measured and theoretically predicated using the sensitivity of calibration, was compared. The deviation of the maximum stress was reasonably small.

From the results of the experiments, the following points could be concluded. From the magnitude response measurement, this kind of cement-based piezoelectric ceramic composite showed good linearity, which was important for exact measurement of stress change. According to the frequency response, the working frequency range could be determined.

Measurement with multiple function inputs indicated that cement-based piezoelectric ceramic composite held an excellent performance to dynamic signals in frequency range for general civil engineering applications.

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Fatigue analysis

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Serviceability and fatigue issues related to vibration of the cables of the Alamillo cable-stayed bridge in Sevilla (Spain)

J.R. Casas & A.C. Aparicio
Technical University of Catalunya, Barcelona, Spain

ABSTRACT: Rain combined with wind action provoking vibration in stays is a phenomenon that has occurred in several cable-stayed bridges. In the last years such response has been also observed several times during stormy weather in the longest stays of the Alamillo cable-stayed bridge built in Sevilla (Spain) in 1992. The maximum displacements observed are in the order of magnitude of 0.5 m and have caused discomfort problems to the pedestrian circulation across the bridge, and may derive in important fatigue problems in the stays themselves in the future. The paper shows the magnitude of the problem, the analytical and experimental studies carried out to analyse the possible solutions and also the first steps developed to stop the vibration problem. The solution supposes the installation of dampers in the cables most prone to vibrate. The damping devices should be in accordance with the relevant aesthetic constraints of the structure.

1 INTRODUCTION

The Alamillo cable-stayed bridge (figure 1) was open to traffic in 1992 during the EXPO-92 world exhibition in Seville (Spain). The span length is 200 m and the tower height 134 m. Deck and tower are connected by 13 pairs of parallel stays. After the construction of the bridge, a dynamic test was carried out measuring the acceleration of tower and deck during the passage of two trucks over the undisturbed and disturbed (by obstacle) pavement. This allowed to derive the natural frequencies, damping and mode shapes of pylon and deck (Casas 1995, Aparicio and Casas 1992). A dynamic test was also carried out in the stays to derive the final force in the cables from the natural frequencies obtained in their free-damped vibration (Casas 1994). Although at that time it was already known the vibration phenomena in cables due to the combined action of wind and rain, during the dynamic test performed in 1992, the damping level in the stays was not measured because definitive criteria to avoid the problem based on damping requirements was not yet available.

2 DESCRIPTION OF THE PROBLEM AND PROPOSED SOLUTION

After some years in operation, the rain-induced vibration of the longest cables (cables 9 to 13) appeared for moderate wind speeds, in the order of 15 to 20 m/s. The maximum transversal deflections are in the order of 0.5 m for cable lengths higher than 200 m. This large amplitude refrains pedestrians to cross the bridge when vibration occurs and can lead in the next future to fatigue problems (FIB 2003, EN-1993). Because the wind-rain induced vibration phenomena is highly dependent on the damping level of the cables, a dynamic test of all stays was carried out in 2004 to measure the level of critical damping in each stay as the first step to analyse and solve the problem. The free-damped portion of the acceleration records were used to obtain the natural frequencies, via Fast Fourier Transform (FFT) and the damping ratios as presented in table 1.

The results in table 1 show that the longest cables of the bridge, in fact, do no match the recommendations today available to avoid rain-induced vibration (FIB 2003, EN-1993, PTI 2001). According to the recommendations from PTI 2001, it was concluded that and additional damping

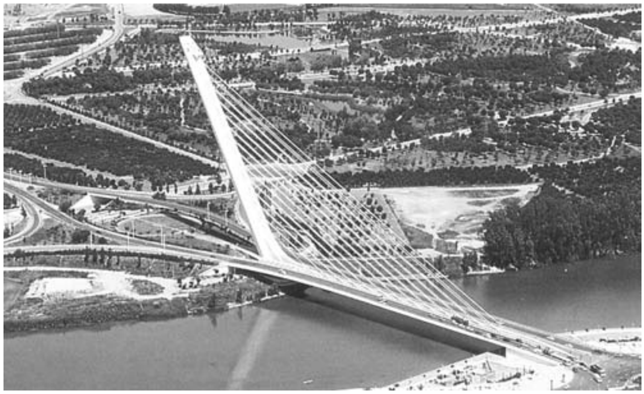

Figure 1. View of the Alamillo bridge.

Table 1. Values of the logarithmic decrement deduced from the total and decomposed signals (R = right, L = left).

	Total record	Decomposed record			
Cable	δ	δ_1	δ_2	δ_3	δ_4
1R	0,150	0,063	0,080	–	–
2L	0,078	0,074	0,037	–	–
5L	0,030	–	0,027	–	–
7L	0,024	–	0,012	0,012	0,011
8R	0,036	–	0,031	0,011	–
9L	0,022	–	0,020	0,010	–
10L	0,019	–	–	0,013	0,011
13R	0,015	–	0,010	0,003	0,003
13L	0,020	–	–	–	–

in the range of 0.03 to 0.05 (logarithmic decrement) in the longest cables (5 to 13) is necessary to stop the rain-induced vibrations. This can be solved by providing damping devices.

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Fatigue cracks of welds and their repair in steel spans of railroad bridge

Z. Manko
Institute of Civil Engineering, Wroclaw University of Technology, Wroclaw, Poland

ABSTRACT: This paper shows several examples of such cracks. An attempt is made to explain their nature and genesis. Also several ways of repairing effectively this kind of dangerous damage to bridge structures are demonstrated. A description of the weld cracks, discovered during expertise and field test work, in structural components of the wind bracings in a steel three-span railroad bridge with simply-supported spans and an open deck situated on the Wroclaw – Poznan railroad line is presented.

1 INTRODUCTION

In July 1994, the Lower-Silesian District Directorate (Management) of the Polish State Railways (DDOKP) in Wroclaw commissioned expertise-research works aimed at the assessment of the service suitability for 225 kN axle load vehicles moving at the speed of 140–160 km/h of all the engineering structures such as bridges, viaducts and culverts situated between the 8.241 km and the 61.032 km of the Wroclaw – Poznan railroad line. In all, 83 engineering bridge structures have been inspected and their strength assessed and 5 of the structures were subjected to field test loads both in the static and dynamic ranges.

In this paper it is described where the cracks in the welds were situated in the gussets of the nodal wind braces and a tentative explanation of their causes as well as methods of repairing them – which may become useful for the engineering practice – are given.

2 DESCRIPTION OF SPAN STRUCTURE

The effective span of each of the spans is 3×11.30 m and their respective total length – 11.90 m. The space between the spans is 180 mm and so the overall length of the steel structure is 36.06 m. The construction height of the spans deck is 945 mm and the axial spacing of the main girders – 3.30 m. The solid-walled main girders consist of a 1120×12 mm web, 300×30 mm top and bottom flanges reinforced at midspan with 6300 mm long 240×15 mm cover plates. The crossbeams in the form of welded plate girders consist of 550×11 mm web and top and bottom flanges in the form of 180×15 mm steel sheets. They are situated perpendicular to the longitudinal axis of the bridge span and spaced at every 2.05 m, except the support crossbeams laid on the skew at an angle of 76°. Two stringers are made of 300 mm rolled I-sections.

3 DESCRIPTION AND LOCATION OF CRACKS

Cracks in welds were noticed during a field inspection of the bridge. Those were butt welds joining horizontal gusset plates (to which the angles of the wind braces were fixed) with the lower flanges of the solid-walled main girders. The joints were executed by fixing with 1/2 V butt welds a $220 \times 120 \times 10$ mm gusset plate to the bottom flange of the plate girders and to the bottom flange of the crossbeams. A wind brace angle ($80 \times 80 \times 8$ mm) is welded with a 5 mm thick fillet weld to the top surface of the gusset plate. The other end of this brace angle is welded to a 500 × 120 × 10 mm gusset plate fixed with a butt weld to the middle part of the lower flange of the adjacent crossbeam.

At the place where the same gusset plates with angles were joined with crossbeams neither cracks nor any other distressful signs of processes, which could affect adversely the load bearing capacity or the strength of the components or the spans were found.

4 ANALYSIS OF CRACK CAUSES

Weld cracks of this kind (which seemed to have fatigue character) can be caused by different factors related to the characteristics of the material from which the components are made and to their geometry as well as to the kind and range of variable loads. Also the way in which the welds were laid and even the quality of the electrodes used can have serious consequences. Material and geometric characteristics include the mechanical properties of the material and the way the component has been formed (the cross-sectional shape, the kind of the joint, the type of the weld, the manner of machining of the metal surface or the grain weld, and so on). The latter effects are labeled as a structural notch and such notches often constitute centers at which fatigue cracks are initiated. In the investigated bridge, 10 mm thick gusset plates were welded (by applying a butt weld) to the 30 mm thick bottom flanges of the plate girders transversely to the action of the force in the flange. As a result there was a severity notch in the form of a sudden change in the thickness of the elements and consequently, a difference in the rigidity at the joint. This is one of the most common notches that often lead to cracking. The problem probably was compounded by the shape of the gusset plate, which is rectangular, and without any fillets (which are routinely used today) at the place where the weld was laid.

5 PROPOSAL OF REPAIR

Since cracks in welds are only in the wind braces, which are considered secondary components in relation to the main girders (rather low of web height) and joined rigidly to crossbeams, they do not necessitate an immediate repair. Still if they were left unattended, they might propagate further in the welds and then spread in the gusset plates (which was found to have already happened in two cases) and even extend to the bottom flanges of the plate girders.

To repair the damage and to remove the supposed causes of the fatigue cracking, holes should be drilled at the ends of cracks; the damaged areas of the welds should be excised and refilled. Whereas the sheets in which cracks appeared should be strengthened with welded cover plates. This was a relatively simple and inexpensive solution.

6 CONCLUSIONS

At the moment the extent of the cracks and the condition of the welds in the span structures are checked periodically. It will be possible to draw definite conclusions concerning the described fatigue cracking and its causes only after more comprehensive strength tests are carried out on the material, the quality of the welds is radiographically examined and strains in the structural components and the welds under a known load are measured.

On the basis of the obtained results it will be possible to carry out a proper theoretical analysis supported by strength calculations. Also it should be emphasized that the effectiveness of a repair method which will be used can be determined best by conducting service tests **under real trains traffic** that will verify the assumptions made and give an accurate picture of the behavior of these components in the span structures. Such tests are being conducted by the author with the approval of the Railroad Road Management Unit of the Lower-Silesian District Directorate (Management) of the Polish State Railways (DDOKP) in Wroclaw to explore the problem in depth and validate the analyses made.

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Fatigue behaviour of riveted steel lap joints

M.A.V. Figueiredo
Faculty of Engineering, University of Porto, Porto, Portugal

A.M.P. Jesus, A.S. Ribeiro
UTAD/ Engineering Department/Mechanical Engineering Division, Vila Real, Portugal

P.M.S.T. de Castro, A.A. Fernandes
Faculty of Engineering, University of Porto, Porto, Portugal

ABSTRACT: The maintenance and safety of existing bridges is a major concern of governmental agencies. In particular, the safety of old riveted highway bridges fabricated and placed into service at the end of the 19th/ beginning of 20th centuries deserve a particular attention, since they were designed taking into account traffic conditions, both in terms of vehicle gross weight and frequency, completely different from those observed currently. In order to assure high safety levels in old riveted steel bridges, highway authorities have to invest heavily in their maintenance and retrofitting. In this context, knowledge of the fatigue behaviour of riveted joints is of paramount importance.

The present paper reports research work carried out to characterize the fatigue behaviour of the Portuguese Pinhão riveted highway bridge, designed by Eiffel at the end of 19th century and built between 1903 and 1906. The Pinhão bridge crosses the Douro river and links Pinhão to São João da Pesqueira and Peso da Régua. The bridge has three spans of 68.8 meters each and one span of 10 meters; there is only one deck with 6 meters width, divided in one traffic lane with 4.60 meters width and two sidewalks with 0.675 meters width each. The goal of the paper is to characterize the residual fatigue strength of the bridge. Both traditional S-N approaches and Fracture Mechanics approaches were used. The study is supported by an experimental program for the evaluation of material properties such as tensile strength, toughness and crack growth properties. Also, fatigue tests of riveted joints were carried out. The material and riveted joints were extracted from bridge members. The members removed from the bridge were a diagonal and a bracing from the first span, from Pinhão side. While the bracing is composed by two equal-leg angles, riveted to each other, the diagonal has a rectangular cross section. Several types of specimens were prepared using the material samples removed from the bridge. These specimens were used in chemical and metallographic analyses, hardness measurements, tensile tests, notch toughness tests, fatigue crack propagation tests and fatigue tests of riveted lap joints.

The chemical and metallographic analyses revealed that the material of the bridge is a carbon steel with small content of Mn, Si and C. The microstructure is composed, essentially, by ferrite with a small content of perlite.

The tensile tests demonstrated a high ductility of the materials with almost null strain hardening. Elongations of 70%, associated to reductions in area of 33%, were observed. The measured mean yield stress and the ultimate tensile strength were 306 and 361 MPa, respectively.

The hardness measurements revealed a small scatter, which confirms a good microstructure homogenization. Mean hardnesses of 108 and 116 HV40 were found for the diagonal and bracing materials, respectively.

Two types of notch toughness tests were carried out. The Charpy V-notch and the COD tests. Both tests revealed that the materials exhibit very acceptable toughness properties even for the current design requirements, which allows a high tolerance to the presence of defects.

Crack growth tests using Compact Tension (CT) specimens were also carried out to characterize the crack propagation law for the materials of the bridge. Two stress ratios were investigated,

namely, $R = 0.01$ and $R = 0.5$. It was observed that Paris law gives a good description of the crack propagation data (Paris & Erdogan 1973). The global mean crack growth law derived was:

$$da/dN = 3.1961\times10^{-15} \cdot \Delta K^{3.6117} \tag{1}$$

where da/dN = crack propagation rate expressed in mm/cycle; and ΔK = stress intensity factor range expressed in $N.mm^{-1.5}$. The influence of the stress ratio is small and the materials from the two members present very similar fatigue crack propagation behaviours.

Finally, fatigue tests of riveted lap joints were performed which allowed the evaluation of a S-N curve with the following form:

$$\log\Delta\sigma = 3.3108 - 0.2226\log N \tag{2}$$

where $\Delta\sigma$ = stress range in MPa; and N = failure cycles. The observation of the facture surfaces revealed, for some specimens, the existence of initial cracks that nucleated and propagated during the last century. The comparison of the experimental data with the AASHTO (1995) class D S-N curve revealed that the later is conservative.

The residual life of the bridge was evaluated using a S-N approach, based on the class D S-N curve, proposed in the AASHTO procedures for riveted joints. Considering a standard vehicle with a gross weight of 300 kN and three axles (RSAEP, 1983) and using information about the daily average traffic flow supplied by the Portuguese highway authorities, the stress spectra at each member was evaluated and the critical one was identified. This analysis demonstrated the safety of the bridge, against fatigue, for a period of 30 years after its rehabilitation. Subsequent analyses, with more precise stress spectra, based on the actual weight density distribution of trucks, confirmed again the safety of the bridge. In this case, the damage accumulation rule proposed by Miner (1945) was applied. Finally, Fracture Mechanics was applied to assess the number of cycles required to propagate an initial crack until critical dimensions, leading to unstable propagation. An initial semielliptic crack was assumed and its propagation was modelled using the Paris law identified for the materials with the experimental work. The stress intensity factor range was evaluated using formulations available in the literature (Cheng & Li 2003). The results from the application of the Fracture Mechanics confirmed the results of the simulations based on the S-N approach.

The main conclusions of this study can be summarized as follows:

- The material of the Pinhão bridge presents mechanical strength properties similar to values obtained with materials of other European bridges built at same time.
- The toughness values of the material are much higher than values demanded by current design codes of practice, which allows a high tolerance to the presence of cracks. Cracks hided by the riveted heads are not critical unless if they become visible. Inspection routines for crack detection are required.
- The fatigue resistances obtained with the fatigue tests of the riveted joints are compatible with the recommendations of actual international codes of practice such as the AASHTO.
- The present study demonstrates the bridge safety against fatigue, after rehabilitation, for a period of 30 years. This analysis was supported by important assumptions related with the stress spectra at the critical locations. More accurate analysis can be adopted if the stresses/loads are experimentally monitored, during a representative period of time.

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Accurate fatigue stress determination in concrete railway bridges considering rail track – structure interaction

A. Herwig & E. Brühwiler
Swiss Federal Institute of Technology (EPFL), Lausanne, Switzerland

ABSTRACT: So-called "non-load bearing" parts of (railway) bridges increase the stiffness and decrease the stresses in short span bridge structures. The rail track, consisting of ballast, sleepers and rails, has a non-linear behaviour and its rigidity depends on the vertical load. By means of analytical investigations, the influence of parameters such as the reinforcement content, the load configuration and the load amplitude on the fatigue stress in the steel reinforcement of the concrete bridge structure is obtained. The results show a 5–8% reduction of the fatigue stress when considering this interaction as compared to conventional structural analyses not considering the load bearing contribution of the rail track. The benefits of this methodology is in view of examining concrete railway bridges for heavier future traffic loads. Since shear failure is unlikely to occur in common bridge elements such as decks and girders, emphasis is given on the determination of stresses due to bending moments.

1 MODELLING OF RAIL TRACK – STRUCTURE INTERACTION

The effect of the passage of two axles, subsequently called "axle configuration 2Q", and of four axles, called "axle configuration 4Q" on stresses in steel reinforcement of a simply supported 10 m-span slab bridge is investigated.

The horizontally rigid and continuous rails with the stiff ballast (the ballast follows a non-linear force-displacement relation in the horizontal direction which depends on the vertical load (Frýba 1996)) create partial clamping of the bridge structure near the abutments which in turn reduces the bending solicitation of the deck slab at mid span where the axle loads are placed. Then the loads are increased by increments of 50 kN with a constant medium value for the ballast stiffness at each step. The values of the slope is approximated by constant values in 10 discrete elements to model the structure. With the calculated relative horizontal displacement between the deck's upper side and the rail grid the value of the horizontal force q is obtained by iterations using the stiffness relation for the ballast which depends itself on the value of q.

In order to calculate internal forces, a fictive normal force F_r in the rail is assumed contributing to the bending resistance of the bridge beam. At mid span, the force F_r is the sum of the contribution q of each element in one half of the span L.

By stating equilibrium of internal forces, the stress range in the reinforced concrete is obtained. In order to determine the reduction of the stress range, the corresponding stress range is calculated for both with and without rail track – bridge interaction effect.

2 RESULTS OF THE PARAMETRIC STUDY

Subsequently the results of some parametric studies are shown. There is a strong dependency on the reinforcement content (Fig. 1a). In particular for low reinforcement contents, the reduction of fatigue stresses varies between 5–7% (axle configuration 2Q) and 6–8 % (axle configuration 4Q,

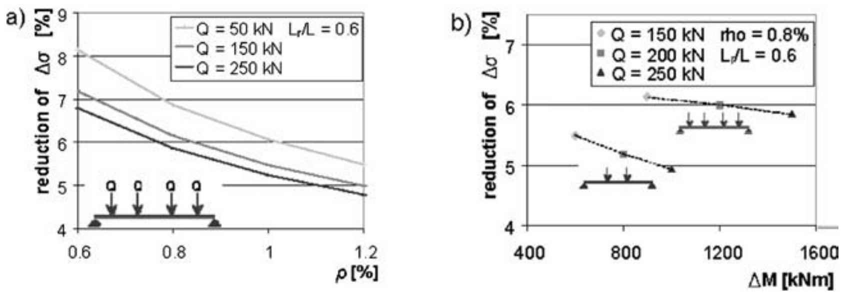

Figure 1. a) Influence of the reinforcement content ρ on the reduction of the stress range in the reinforcement for load configuration 4Q and b) Influence of the axle configuration on the reduction of the stress range in the steel reinforcement with height of rail axis over the upper surface of the bridge deck $h_r = 600$ mm.

Fig. 1a). For higher reinforcement contents, the reduction in stress range is less pronounced but fatigue problems in the steel reinforcement are also less likely to occur.

Load configuration 4Q leads to higher bending solicitation (Fig. 1b), but the reduction of stress range in the reinforcement is more important. There is also a strong dependency of the reduction in stress range on height of the rail axis h_r over the upper surface of the bridge deck. Increasing thickness of the ballast thickness provides an increase in stiffness of the rail track – bridge deck system. The effect of moving axles applying different load positions are analyzed in terms of the stiffness of the system rail track-bridge.

The modelling with stationary axle loads represents well the reality, as moving axle loads create also maximal stresses at mid span and the stiffness of the system rail track – bridge than is the same in both cases.

3 CONCLUSIONS

This study shows that considering rail track – structure interaction leads to more accurate fatigue stress determination in short span concrete railway bridges.

The parametric study has shown that the axle load configurations (f.ex. 2 or 4 single loads) and the reinforcement content are the most important parameters involved. Especially slender deck bridges with low reinforcement content (provided the structural safety is fulfilled) benefit, due to their low bending stiffness, from the rail track – bridge structure composite effect.

The results of this study also provide a methodology and an instrument that allow creating charts or tables for more accurate determination of fatigue stresses.

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Application of post-weld treatment methods to improve the fatigue strength of high strength steels in bridges

U. Kuhlmann, A. Dürr & H.-P. Günther
Institute of Structural Design, University of Stuttgart, Germany

1 INTRODUCTION

According to the present state of the art the fatigue strength of welded high strength steel connections is assumed to be the same as for welded standard steel connections. For an effective application of high strength steels in constructions subjected to repeated loading additional efforts are therefore necessary in order to improve the fatigue strength. This can be done for example by the application of post-weld treatment methods. However, up to the present it is not possible to apply the positive effects of such methods within the fatigue design standards for steel structures. Within this paper an experimental test program studying the effectiveness of the two post-weld treatment methods TIG-dressing and Ultrasonic Impact Treatment (UIT) is presented. The test program comprises small scale fatigue tests on standard and high strength steel grades as well as full size girder fatigue tests.

2 APPLICATION OF HSS

The application of HSS in composite road bridges was analyzed in a parameter study on single and double span bridges within the range of span between 20 m and 70 m for the steel grades S355, S460 and S690 for an open plate girder cross section consisting of two welded I-girders for two-lane roads. The results of the parameter study show that the importance of the fatigue verification increases with the application of HSS. The critical construction detail for the fatigue verification is the transverse stiffener in the sagging moment region with the detail category of 80. In order to achieve an economic application of HSS an improvement of fatigue strength is necessary.

3 POST-WELD TREATMENT METHODS

Very effective methods to enhance the fatigue strength are post-weld treatment methods. However up to the present it is not possible to apply their positive effects in the design of structures. The most popular methods with a large amount of available test data and application recommendations are grinding, TIG-dressing and needle and hammer peening.

A very new and promising post-weld treatment technique is the so called "Ultrasonic Impact Treatment" (UIT). UIT is a mechanical hammering technique at a frequency of around 200 Hz superposed by a ultrasonic treatment at a frequency at 27 kHz. These combined impacts result in plastic deformation at the treated surface leading to beneficial compressive stresses in the surface layer and a smoothen of the weld toe transition. Several investigations showed a substantial enhancement in the fatigue performance for different welded construction details by UIT.

4 RESEARCH PROGRAM

Based on the existing and successful studies on UIT an extensive testing and research program was initiated at the universities of Stuttgart and Weimar (Kuhlmann et al. 2006). The purpose of the

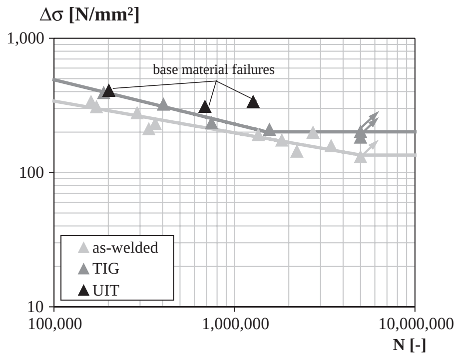

Figure 1. S-N diagram (S690, R = 0.1).

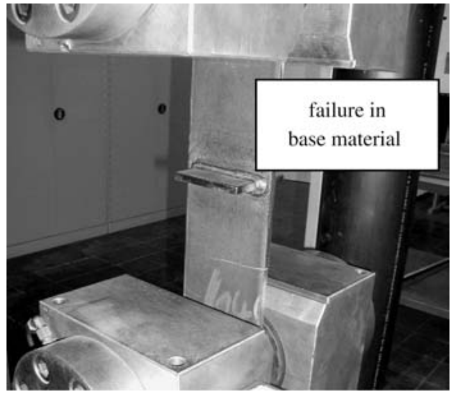

Figure 2. Fatigue failure in the base metal.

research program was to investigate the fatigue strength improvement on HSS by the application of different post-weld treatment methods (TIG-dressing and UIT) on the detail of the transverse stiffener. The influence of yield strength, plate thickness, stress ratio and specimen size were investigated on small scale tests and girder tests for the steel grades S355, S460 and S690. On the results of the small scale specimens it could be observed that for all steel grades due to the application of post-weld treatment methods the fatigue strength could be improved considerably, especially in the region of high-cycle fatigue. The best results were achieved by the post-weld treatment method UIT with an improvement of fatigue strength $\Delta\sigma_C$ related to 2 million cycles of around 80–90% for the steel grades S355 and S460. The effectiveness for the UIT-application increased even with the application of HSS, see Figure 1. For the grade S690 the improvement of fatigue strength was so high that the failure of the specimens did not occur anymore in the region of the weld but moved in the base material, see Figure 2.

The results of the fatigue girder tests have shown also the effectiveness of UIT in components even with the high stress ratio of R = 0.5. The enhancement of fatigue strength related to 2 million cycles was due to the application of UIT of around 35% compared to the as – welded state.

5 CONCLUSION

Based on the work presented in this paper the conclusion can be drawn, that when applying HSS in composite road bridges the fatigue verification can become the decisive design criteria. An effective use of HSS can be achieved with the locally limited application of post-weld treatment methods on the critical construction details. Especially the post-weld treatment method UIT seems to be very promising.

ACKNOWLEDGEMENT

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Fatigue strength of web-gusset welded joint pasted with glass fiber reinforced polymer

H. Suzuki
Meisei University, Hino, Tokyo, Japan

ABSTRACT: In this study, fatigue tests were done using three kinds of web-gusset welded joint specimen; a specimen without strengthening, a specimen with a glass fiber reinforced polymer (GFRP) pasted with an adhesive on the weld zone, and a specimen with the weld zone coated with only the adhesive. The effect of the GFRP and the adhesive and of only the adhesive is shown.

1 INTRODUCTION

The effect of GFRP on the fatigue strength of a welded joint is described in this study. The target welded joint in this study is a web-gusset welded joint. Such joints are used in plate girder bridges: gusset plates are welded to a web plate of main girders to connect lateral bracings. It has been reported that many fatigue cracks occur in the round weld toe of the end of the gusset plates in web-gusset welded joints.

This paper describes fatigue tests of web-gusset welded joints strengthened with GFRP and an adhesive, and with only an adhesive. The effect of the GFRP and the adhesive and of only the adhesive on the fatigue strength of the web-gusset welded joints is discussed on the basis of S-N curves, relation between crack length and number of cycles, and photographs showing the crack propagation.

2 EXPERIMENTAL PROCEDURE

The specimen configuration is shown in Figure 1. Figure 1(a) is a non-strengthened specimen: NS. Figure 1(b) is a specimen pasted with the GFRP using an adhesive: GA, or coated with only the adhesive: AD. In GA, four layers of GFRP were pasted to the weld zone using the adhesive, and total thickness of the GFRP and the adhesive was 2.8 mm on the base metal. In AD, the thickness of the adhesive on the base metal was 0.7 mm. The thickness of the strengthening materials on the weld zone was thicker than that on the base metal. The GFRP in this study was a resin sheet hardened with ultraviolet rays and unidirectional fabrics. The adhesive was an epoxy acrylate adhesive requiring the mixture of two liquids.

3 RESULTS AND DISCUSSION

Figure 2 shows regression curves of NS, GA, and AD. Comparing the regression curves of NS with GA, both curves come close in the high stress range. However, the fatigue life of GA is improved more than that of NS at lower stress range. The strengthening effect by pasting the GFRP is not expected in the high stress range, but a high strengthening effect can be expected in the low stress range. Next, comparing GA with AD, it is known that the strengthening effect of AD is greater than

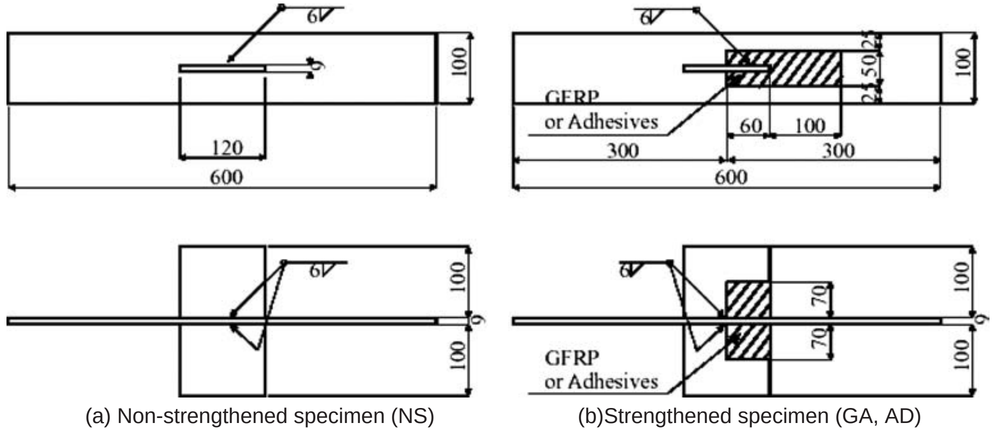

Figure 1. Specimen configuration.

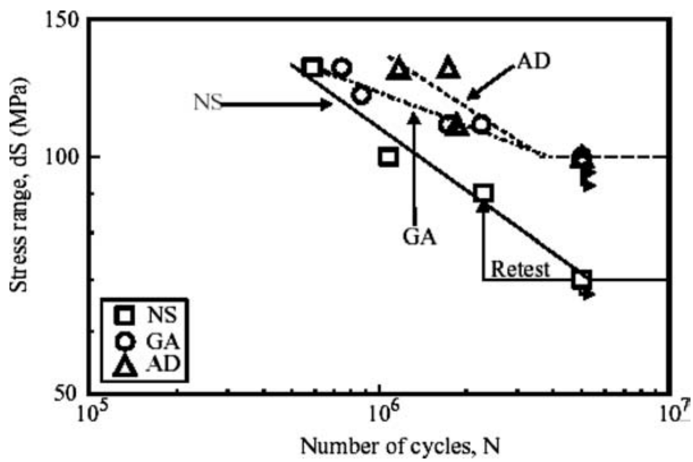

Figure 2. Fatigue test results (Regression curves).

that of GA in the stress range of dS = 130 MPa, but the fatigue life of GA increases according to the decrease of stress range and the strengthening effect of GA becomes equal to that of AD in the stress range of dS = 100 MPa. The reason for this is thought to be as follows: the adhesive on the weld zone has also separated together at the time the GFRP peels off in the high stress range.

4 CONCLUSIONS

(1) The fatigue strength of the welded joint can be improved by pasting the GFRP with the adhesive to the welded joint, and by coating the welded joint with the adhesive.
(2) Pasting the GFRP to the welded joint hardly improves the fatigue strength of the joint in the high stress range, but greatly improves the fatigue strength in the low stress range.
(3) Coating the welded joint with the adhesive greatly improves the fatigue strength of the joint in both the high and low stress ranges.
(4) The improvement of the fatigue strength by coating with the adhesive is nearly equal in the low stress range to the improvement by pasting the GFRP to the welded joint.

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Ultrasonic impact treatment for life extension of bridges with cracked and crack susceptible welded details

E. Nyborg & P. Moffatt
TISEC Inc., Montreal, Quebec, Canada

J. Cavaco
Canadian National Railways, Edmonton, Alberta, Canada

D. Nyborg
Nyborg Maskinteknikk, Montreal, Quebec, Canada

ABSTRACT: This paper assesses the effectiveness of ultrasonic impact treatment (UIT) on in-service bridge members. It presents the results of five years of acoustic emission (AE) monitoring of fatigue crack activity, both before and after UIT, at a large number of locations on a heavily loaded, 30-year-old railway bridge with both cracked and crack susceptible welded details. Positive results were obtained in treating details both with and without active fatigue cracks, however additional research is needed to determine the long term effectiveness of UIT as well as to develop an optimum repair and treatment protocol for details with existing cracks.

This study is summarized in the following two pages.

1 INTRODUCTION

Canadian National Railways (CN) is one of North America's major freight railways, with a rail network of over 31,000 km, stretching from the East coast of Canada to its West coast and down to the Gulf of Mexico in the USA. With a 2004 cost/revenue operating ratio of 66.9, CN is the most efficient railroad in North America. While operating efficiency depends on many factors, much of CN's success can be attributed to the appropriate application of new technologies. This has included a strain gauge testing program to assess the carrying capacity of existing bridges, as well as acoustic emission (AE) monitoring to track the behaviour of crack susceptible structural details and investigation into techniques for safely extending the life of bridge with fatigue susceptible details. Ultrasonic Impact Treatment (UIT) was identified as a technology with the potential to extend the life of bridges fabricated with crack susceptible welded details.

2 ULTRASONIC IMPACT TREATMENT (UIT)

Statnikov (2001) describes the characteristics of UIT for post weld treatment. UIT differs from conventional mechanical peening in that it uses low amplitude (25 to 40 μm), high frequency (27 kHz) impacts. Conventional peening equipment, operates at 50 to 100 Hz with much higher amplitudes. As a result UIT applicators are light weight, quiet and easy to use in relation to traditional peening equipment. These characteristics make UIT suited for field use.

The objective of UIT is to introduce compressive residual stresses in the weld toe and to reduce stress concentration by improving weld toe profiles. Stress relaxation from UIT occurs to a depth of up to 12 mm below the treated surface. Fisher et al. (2001) in laboratory fatigue tests on welded, built up girders reported that UIT significantly increased the fatigue strength of cover plate welds and of transverse stiffener welds on a tension flange and web. They also reported using welding combined with UIT to replace cracked web connection plates on four highway bridges. Uppal et al (2002) reported that UIT to bridge weld details showed an estimated reduction in residual stress of about 50 MPa near the welds. The above results relate to UIT of new welds or welds that have not yet shown crack initiation. There are no reported results on UIT of welds with active cracks nor has the long term in-field effectiveness of UIT been established.

3 UIT ON A 30-YEAR OLD BRIDGE WITH CRACK SUSCEPTIBLE WELD DETAILS

The Port Hope Viaduct is located on the CN main line about 100 km East of Toronto. The 376 m long open deck bridge carries two tangent tracks, consisting of 20 deck plate girder spans per track. It has seen an ongoing large increase in traffic and in 2004 each track sustained a total mass loading of 33 million t from the movement of about 476,000 freight cars and 16,000 locomotives.

The 30-year-old South track spans use welded and bolted construction. About 10 years ago these spans began to experience fatigue cracks near the bottom flange welded connections. Actions to deal with the fatigue problem, included reducing the speed limit on the bridge, increasing the inspection frequency and drilling holes at the tips of active cracks tips as soon as they were detected. In March 2000 AE monitoring was used to characterize the locations with the most active fatigue cracks, while in March 2001 most of the fatigue-prone locations, including those which were monitored in 2000, were treated with UIT and an annual AE monitoring program was begun. In locations containing active fatigue cracks, UIT was not applied to the crack but was instead applied to the weld which initiated the crack and to the periphery of the arresting hole drilled at the end of the crack, to isolate the crack and to retard crack initiation.

As discussed by Nyborg et al. (2001), CN has maintained a program of AE monitoring for over 15 years and has worked with several companies in developing an AE monitoring technology which provides information for the maintenance of fracture critical bridge components. The objectives of AE monitoring are threefold – to detect developing fatigue cracks, to determine the relative activity of existing cracks and to assess if repairs have been effective in arresting crack growth. Nyborg et al. (2000) discuss waveform classification which plays an important part in the AE monitoring program.

4 CONCLUSIONS

Five years of AE monitoring of crack activity, before and after UIT, revealed the following:

(1) UIT applied to the welds of crack susceptible details was effective in inhibiting fatigue crack development over the monitoring period.
(2) At welded details with active fatigue cracks, UIT of both the welds and the peripheries of the crack arresting holes, isolated the cracks and arrested or retarded crack growth. AE monitoring over the five year period showed cessation of emissions from crack tip growth, with an ongoing increase in emissions from crack face rubbing.
(3) Additional research is needed to determine the long term effectiveness of UIT as well as to develop an optimum repair and treatment protocol for details with existing fatigue cracks.
(4) Once long term effectiveness has been better established, using UIT to treat in-service, poor category weld details, could lead to substantial savings in deferring replacement and retrofitting of steel bridge spans and in increasing the life of railway bridges.

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Fatigue lifetime estimation of Chunho steel box bridge on Han River

J.S. Lee
Korea Infrastructure Safety and Technology Corporation, Goyang, Korea

ABSTRACT: Imperfect Penetration (IP) defect is one of the worst defects which could be occurred in the steel bridges. IP defect of tension region of steel girder would induce stress concentration at the tip and eventually could develop cracks at that point. So, many researchers have recommended to fully investigating the welding quality of tension region of steel girder when performing in-depth inspection of the steel bridge and, if IP defect was found, it is needed to perform more detail investigation, such as core test, ambient stress measurement, and fatigue lifetime estimation. Then, we should determine whether the repair should be needed or not.

The KISTEC had performed the in-depth inspection project of Chunho Bridge on Han River in Korea. Chunho Bridge is located in Seoul City crossing Han River. From the ultrasonic test (UT) results, we had found lots of welding defect. Most of the defects were IP defects. IP defect in the butt weld of tension region of box girder causes the lack of sectional area and increases normal stresses. In case the edge of IP defect were sharp, it would cause fatigue crack due to stress concentration.

We would recognize that these box girders had very poor welding quality and those defects would cause structural problem. We needed to know whether the fatigue crack was developed at the tip of IP defect or not. We decided to investigate the welding section of the IP defect by taking cores at the worst points by drilling. It was very difficult to decide the points having the poorest welding quality because, with NDT result, we couldn't find out the exact size and shape of IP defect. However, we could manage to decide the relative worst points with ultrasonic signal intensity of UT. We had chosen three points, which were believed, to have the poorest welding quality. We took steel cores with a drill and investigated the size and the shape of the IP defect.

After taking 3 cores (C1, C2 and C3) from G1 and G2 girders, we cut the cores perpendicular to IP defect and polish and etch the surfaces. Then we measured the sizes and investigated the boundaries of each IP defect with electronic microscope. We found cracks were developed at the tips of the IP defect of C1 core. We checked the stresses induced in the lower flanges of girders due to design dead and live loads and estimate fatigue lifetime of critical sections using stress measurement result due to ambient traffic loads.

To check structural safety of welding joints of lower flange of steel box, we did structural analysis, calculated normal stresses at the welding joints, and compared them to allowable stress. We chose 6 sections among the joints of lower flange considering bending moment of each section and lack of sectional area. If there were no defects in the welding joints, the stresses would be under allowable stress. But because of the lack of sectional area due to IP defect, the stresses exceeded the allowable stress at the critical sections. It means that this bridge is not safe to design loads.

The fatigue crack is initiated where out-of-plane deformation is occurred, where the cross section is suddenly changed, where the stress level is so high and where the stress is concentrated. Generally, butt weld of lower flange of steel girder is known to be safe to the fatigue crack. But in this case, because of the IP defect in the butt welding line of lower flange, the stresses due to dead and live loads are exceeded the allowable stress. From the core investigation, we identified cracks at the tips of the core C1.

We measured the stresses due to ambient traffic loads at critical sections and estimated fatigue lifetime for verifying the possibility of occurring fatigue crack at the critical section. We installed

strain gauges at the sections and measured the stress count for 3 days. Then we calculated accumulated fatigue damage indices at each point and estimated remained lifetime of the sections.

In order to get stress count for calculation of accumulated fatigue damage, we chose the same sections which we took the cores and installed 5 strain gauges at both sides of butt welding line of each section as shown in the figure 7. We used foil-type strain gauge for sensor and Histogram Recorder and Histogram Analyzer for hardware.

According to bridge design specification of Korea, the fatigue category of butt weld having perfect welding quality is C. There were no special specifications concerning welding defects. Juhn et al.[1,2] performed two research projects for the estimation of welding quality of steel members. As the result of these projects, he presented the guideline for the estimation of welding quality of steel members when inspecting the steel bridges. He made more than hundreds of welded steel specimen with various size of IP defect inside the butt weld, performed fatigue test and got the fatigue lives of each specimen for various loading stages. Then, he classified and plotted the test results according to the size of IP and stress levels and had gotten modified S-N curves for each case.

From the NDT results, the core investigation and the result of former research, we decided the fatigue category of each section. Yearly accumulated fatigue damage (D_f) is calculated from the field measurement and allowable fatigue stress range due to modified fatigue category. The fatigue lifetime (Y_f) can be calculated with D_f and remained lifetime is obtained by subtracting use duration from lifetime.

The least remained lifetime of steel box girder of Chunho Bridge is 5 years. This result is so closely coincide with the result of the investigation of the core. Of course, these are not the real remained lifetimes of the girders. We couldn't measure the real stress variation of box girders, couldn't know the past traffic volume of this bridge, and couldn't know how fast the crack develop. Anyway, all we can conclude is that the condition of butt weld joints of the steel box girders is very poor and the crack can be initiated and/or developed to the surface of the lower flange in the near future.

According to the result of structural analysis, the critical sections were known to be unsafe to the design load. This was verified by the core investigation and fatigue lifetime estimation. Because the cracks are already initiated at the critical section, we concluded this was so serious. So, we reported this circumstance to Seoul City Government and suggested that the urgent reinforcement should be needed at the butt weld joints of critical sections. We chose the sections to be reinforced among the critical sections according to the NDT result, i.e., among the critical sections, the sections without any welding defect were excluded. This reinforcing work was started just after we suggested the drawings to Seoul City Government and finished before the inspection project was finished.

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Fatigue monitoring of steel railway bridges

O. Hechler & M. Feldmann
Lehrstuhl für Stahlbau und Leichtmetallbau, RWTH Aachen, Germany

B. Kühn
PSP, Planung und Entwicklung im Bauwesen GmbH, Aachen, Germany

ABSTRACT: Bridge rehabilitation to increase the service life and to reduce the maintenance costs becomes more and more important for steel railway bridges. Thus assessment procedures are required that comprise insight and comprehensive knowledge. This can be achieved by the combination of theoretical and experimental investigations by monitoring. For the application of technical monitoring on steel railway bridges various systems to particular monitoring tasks exist. The final choice, which system is most beneficial, depends on the aim of the monitoring measure. In this paper monitoring is defined and monitoring systems are introduced. Further the implementation of monitoring in the service life analysis for steel railway bridges and a draft of a risk analysis for the evaluation of the benefit of the appliance of monitoring is presented.

1 INTRODUCTION

Due to the age of existing bridges, increasing loads and changing requirements for use, maintenance and strengthening of bridges is focused to a higher extend. Corrective maintenance of old structures is increasingly a cost efficient alternative to build new structures. Simultaneously, sustainability aspects and a sensitive treatment of energy and material resources are, to a greater extent, in the focus of designers and financiers. In addition historical and social aspects may lead to a cultural value to be preserved. Thus the general trend is against replacing the old structure and goes towards rehabilitation [PEIL 2003]. A well prepared rehabilitation supported by monitoring allows for an efficient allocation of money.

In the scope of the project [SUSTAINABLE BRIDGES] hints, methods and tools for designing, implementing and maintaining a monitoring system are provided. The content of the guideline for fatigue assessment of steel railway bridges [HECHLER 2005] is presented in the full paper.

2 MONITORING FOR STEEL RAILWAY BRIDGES

Monitoring has to be divided into two categories. The first category of monitoring comprises regular visual inspections according to the specifications of the national standards in pre-defined time periods. The second category includes technical monitoring, focussed in the full paper. A technical monitoring system is a data acquisition and processing unit providing continuously and autonomously real-time information about a structure or structural component. Technical monitoring should contribute to an exact estimation of the bridge condition. It is applied in different ways and follows diverse aims.

A monitoring health system, for example, is designed to record, analyse and display the loading history of structures and the structural reactions while operating. All data should be recorded and provided if required. To assure an errorless operation of the monitoring system environmental influences and basic conditions for steel railway bridges are to be considered. Further requirements on precision, sensitivity, reliability etc. of physical sensors have to be specified depending on the monitoring task. The world market already offers a variety of different health monitoring systems. The decision on which health monitoring system should be applied highly depends on the task and requirements of the specified monitoring application. More information on monitoring systems can be found in [FELTRIN 2004].

Nevertheless monitoring has also got limitations. The goal of continuous monitoring is not to supersede periodical inspections but to improve the efficiency of the inspection process. Technical monitoring could especially support the estimation of more accurate input parameters for the residual service life estimation. If the result of the service life analysis leads to the conclusion, that the residual life expectations are unsatisfactory, measures have to be considered. With more accurate parameters in the service life analysis the residual life determined might increase and measures could be avoided.

Options and aims as well as hints on conditions and requirements for technical monitoring of steel railway bridges are given in [HECHLER 2005]. Further a general approach for the service life analysis extended by technical monitoring is presented.

3 EVALUATION OF THE BENEFIT OF MONITORING

The decision for monitoring always depends on the benefit of its application. Thus an analysis has been invented to evaluate the risk minimization due to monitoring using a fixed scheme and therefore to objectively estimate the profit of monitoring. In this paper a draft of a risk analysis for the evaluation of steel railway bridges with respect to the application of technical monitoring is presented.

In this risk analysis three categories of risk factors are differentiated, structural factors (S), environmental factors (E) and other risk factors (R). Further a classification for the global risk is introduced in the form of an evaluation scale from 1 to 10. The change from one category to the other is based on the shift of potential risk of a damage occurring. In a further step the scale is divided in the 6 risk classes, for a very small risk up to an extremely high risk. With the application of monitoring the possibility is given to attenuate risks in this analysis. Therefore it is analysed how monitoring can be beneficially applied in each of the categories (S, E, R) and what is the influence of technical monitoring on the risk coefficients. The justification of the reduction factors due to monitoring are given in [LÖSCHE 2005].

After the determination, the risks, attenuated or not, can be compared and the benefit of monitoring is estimated. A scale unit for the risk can be €.

4 SUMMARY

The aim of the project [SUSTAINABLE BRIDGES] is to arm railway bridges to meet the demands of the 2020 scenario and to provide methods and tools to the bridge owners to upgrade their bridges if they fall short. The full paper presents project results to a well prepared rehabilitation. One main aspect for the justification of the appliance of monitoring is, especially for the bridge owner, to which degree the risk of failure of the structure is reduced due to monitoring. The benefit of monitoring is determined according to a risk analysis presented in the full paper. The analysis is designed to be a tool for convincing the bridge owner about the effectiveness and benefit of the application of a monitoring system, if reasonable.

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Probabilistic fatigue life estimates for riveted railway bridges

B.M. Imam, T.D. Righiniotis & M.K. Chryssanthopoulos
School of Engineering, University of Surrey, Guildford, Surrey, UK

B. Bell
Network Rail, UK

ABSTRACT: A large percentage of the railway bridges in the UK rail network and around Europe are of riveted construction exceeding in many cases 100 years of age. The remaining fatigue life of these bridges is difficult to estimate due to the uncertainties regarding the fatigue behaviour of wrought-iron and older steel material which were used for their construction. The problem is further compounded by the uncertainties associated with the loading both past and future. Previous global finite element analyses of a typical wrought-iron riveted railway bridge have shown that the fatigue critical details are the inner stringer-to-cross-girder connections (Imam et al. 2005). The analyses were carried out under a historical load model (Imam et al. 2005), developed to represent rail traffic in the period 1900–1970, and present day traffic (BS5400 1980) for the period 1970 onwards. Deterministic remaining fatigue life estimates of the connections were found to be sensitive to the level of dynamic amplification as well as the fatigue classification of the details. Following this work, this paper presents probabilistic fatigue life estimates for the most highly damaged stringer-to-cross-girder connection, as identified by the global analysis of the riveted bridge. On the loading side, the problem is randomised through the frequency of train traffic, dynamic amplification and uncertainties regarding the difference between actual and calculated stresses. On the response side, different assumed S–N curves used for detail classification and the Miner sum are also treated as random.

The probabilistic analysis, which is carried out using Monte Carlo simulation, shows that the most heavily fatigue-loaded stringer-to-cross-girder connection has considerable fatigue life reserve. Through a sensitivity study, it is found that for a 2.3% probability of failure, the remaining fatigue life of the investigated connection is equal to 68 years for a pessimistic scenario. Under a more realistic combination of variables (base model), the 2.3% characteristic remaining fatigue life is found to be 480 years.

Figure 1 shows the effect of different variables on the time to attainment of a 2.3% probability of failure assuming a base model. It can be seen that fatigue life estimates exhibit the highest sensitivity to detail classification, in other words the constant amplitude fatigue behaviour of the detail, and the factor α, which takes into account the difference between measured and calculated stresses.

In particular, by changing the detail classification from Class WI (base model), which is used to represent wrought-iron riveted details, to Class D (BS5400 1980), which is used to represent bolted or riveted steel connections with a high clamping force, there is a 180% increase in the time required to attain a 2.3% probability of fatigue failure. On the other hand, a change to a modified Class B detail, which may be thought of being representative of a riveted connection with no or low clamping force in the rivets, is found to result in a 40% decrease in the same time. The use of a mean value of 0.70 for the factor α is found to result in an increase in the fatigue life at the 2.3% failure probability by about 110%. However, a mean value of α of 0.90 is found to result in a decrease in this fatigue life by about 50%.

The mean values of the cumulative damage model (Δ) and the dynamic amplification factor (DAF) may be seen in Figure 1 to be of less importance.

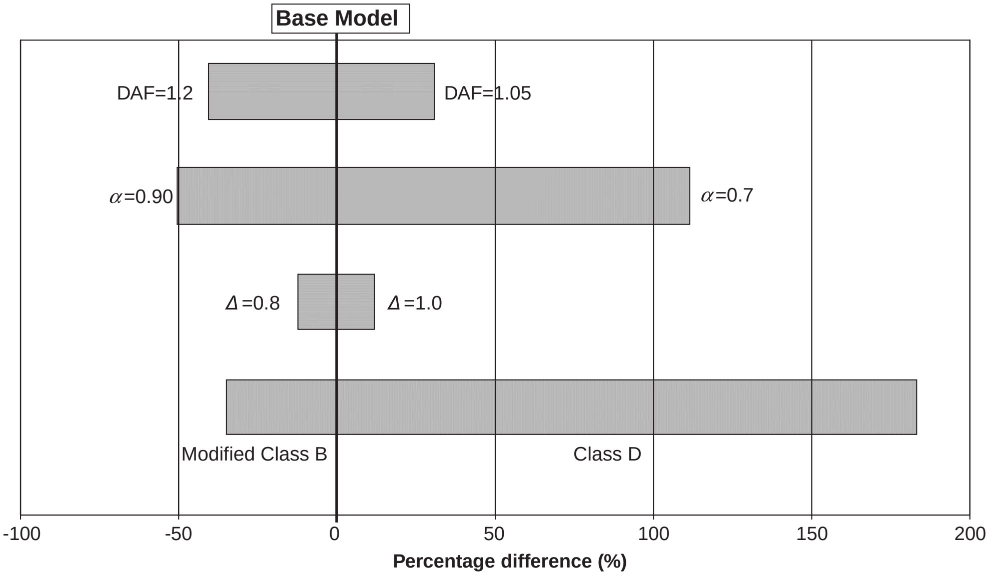

Figure 1. Effect of different variables on remaining fatigue life for a 2.3% probability of failure.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Fatigue on metallic railway bridges: Methodology of analysis and application to Alcácer do Sal Bridge

D. Ribeiro, R. Calçada & R. Delgado
Faculty of Engineering of the University of Porto, Porto, Portugal

ABSTRACT: The characteristics of structural materials deteriorate as a result of the application of repeated loads, such as railway traffic loads. The application of these loads originates lower stresses than the failure stress of the materials, which are experimented repeatedly throughout the design life of the structure, thus potentially resulting on the occurrence and propagation of cracks in certain elements or joints. This phenomenon is defined as fatigue.

Such situation may require complex reinforcement operations, which might lead to traffic in-eruptions, particularly relevant in the event of occurrence of fatigue phenomena in the main structural elements of the bridge.

The vibrations induced by the passage of trains can contribute to the magnification of the fatigue phenomena. Such contribution may be especially important in cases of resonance of the structure, which tend to occur predominantly for speeds greater than 200 km/h.

In this paper, a methodology for the fatigue analysis of metallic railway bridges, known as the method of damage accumulation, is presented. The application of this methodology involves the stress cycles count by an algorithm generally used for this purpose, the rainflow counting method, which was implemented within the scope of this work. The method of damage accumulation was applied to the fatigue analysis of structural elements of the main beams of the deck of the Alcácer do Sal bridge, a bowstring metallic bridge, located in the Southern Line of the Portuguese Railways, for the passage of real high-speed trains and fatigue trains, as established in EN1991-2 (2003). The damages were obtained for a hypothetic traffic scenario and for the traffic mixes established in the regulation, that is, normal, heavy and light traffic.

Concerning the real high speed trains the analyses were performed for the passage of the Alfa Pendular train and the high speed trains ICE2, EUROSTAR, TGV, TALGO, THALYS, ETR-Y and VIRGIN.

The time records of the axial forces, from which the normal stresses were obtained, resulted from dynamic analyses for the passage of the referred trains at speeds between 145 km/h ($\approx$40 m/s) and 420 km/h (1.2 $\times$ 350 km/h), except for the Alfa Pendular train in which case speeds ranging from 145 km/h and 265 km/h (1.2 $\times$ 220 km/h) were considered.

The fatigue strength curve used in these analyses corresponds to detail category 80, as defined in prEN1993-1-9 (2003). The calculation curve has been obtained from the characteristic curve considering a partial safety factor equal to 1.35, corresponding to a cut-off limit ($\Delta\sigma_L$) of 24 MPa.

The results regarding high speed trains have shown that the damage is only significant near the resonance speeds.

As an example, it is presented the stress histories in bar UC for the passage of the TALGO train at two speeds: 185 km/h, at which there is no resonance of the deck and 265 km/h, to which resonance occurs in the deck. The results show that for a speed of 185 km/h, there are no stress range cycles higher than 24 MPa, therefore the induced damage is null. For a speed of 265 km/h, a significant number of cycles at stress ranges higher than 24 MPa has been detected, hence there is a contribution to damage.

The assumption of a hypothetic traffic scenario of real high speed trains, corresponding to the passage of 156 daily trains and a design life of 100 years, has revealed that the damage is

greater than unity in the speed interval of [240,300] (km/h) for the TALGO train, in the speed interval of [300,360] (km/h) for the EUROSTAR train, and in the interval [360,420] (km/h) for the EUROSTAR, TGV and THALYS articulated trains.

As for the fatigue trains, the greatest damages were obtained for the CF5 to CF8, CF11 and CF12 freight trains.

In terms of individual passages, it was observed that the damage induced by real high speed trains in case of resonance is greater than that caused by the most adverse freight trains.

In what concerns the traffic mixes (heavy, normal and light), the damages were lower than unity for the three scenarios and for a design life of 100 years.

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Fatigue life improvement of existing steel bridges

T. Ummenhofer & I. Weich
Institut für Bauwerkserhaltung und Tragwerk, Technical University Braunschweig Carolo Wilhelmina of Braunschweig, Germany

T. Nitschke-Pagel
Institut für Füge- und Schweißtechnik, Technical University Carolo Wilhelmina of Braunschweig, Braunschweig, Germany

ABSTRACT: Research has been initiated on the application of weld improvement methods to increase the fatigue life of existing steel bridges. The research is particularly relevant because a large number of welded bridges will reach their design fatigue life during the next years. The investigations involved fatigue tests on specimens which have been treated with weld improvement methods after they reached their calculated life time and tests on virgin specimen.

The effect of the application of Ultrasonic Peening (UP) and Ultrasonic Impact Treatment (UIT) has been compared to the effect of shot peening as a commonly used method in the mechanical industry. Further investigations focussed on the improvement effect of UP on preloaded details whereas preloading has been defined as the application of the full design fatigue loading. These enabled the investigation of the influence of the weld treatment method on the improvement of the fatigue life of "old" weld details. The tests included butt- and fillet welds as well as double bevel seams. The specific stress ratio was set to $R = 0.1$ at different stress levels to derive slopes of the SN-curves.

The different effects of the improvement methods in the micro and macro structural scale have been analysed. Surface compressive residual stresses with values up to the yield strength could be proved after the application of all three methods. Shot peening caused the most uniform surface stresses up to a depth of 0.2 mm whereas UP and UIT introduced stresses up to a depth of 1mm but only at the treated weld toe. Shot peening causes nearly no further geometric improvement whereas the indents of the UP/UIT lead to an overall increase of the macro radius of the weld toe. However, the investigations show that also sharp burrs occur at the edges of the indents causing sharp notches so that a general geometric improvement is not generally achieved.

The fatigue tests prove that the three improvement methods analysed in this research project can increase the fatigue strength of the welded notch details strongly. The test series show that the application of mechanical methods not only produce an high enhancement of the fatigue strength of the virgin welds but also increase the fatigue life of existing preloaded structures when optimally adapted treatment parameters are used. For the same number of load cycles respectively two million the fatigue strength can be enhanced by a factor of two. Regarding the fatigue life for higher numbers of load cycles respectively low stress levels an even better increase has been found. First analyses of the slope of the treated specimens prove that m values of a minimum of five will be a good approximation for UIT treated welds. For structures loaded by high load cycles respectively low stress levels, flatter SN-curves will result in considerable higher service lives.

The effect on butt welded plates with a thickness of 8 mm and 30 mm has been analysed to determine the influence of the thickness could be determined (Fig. 1). The test results show that the improvement rates are as good for the thick plates as for thin ones so that no thickness effect on the improvement rate has to be taken into account.

It appeared that the enhancing effects on the fatigue strength are based on strain hardening combined with beneficial inherent compressive stresses. It is assumed that further optimization of

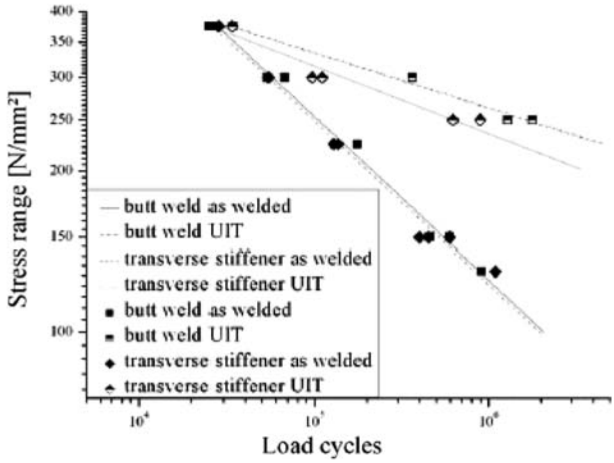

Figure 1. SN-curves (Pü50) for butt welds (S460TM, t = 30 mm) and transverse stiffeners (S460TM, $t = 30$ mm).

parameters of the ultrasonic methods will lead to even better reproducibility of the weld improvement. It can be assumed that even higher improvement rates will be gained especially if high strength steels are used because a higher level of residual stresses can be reached.

The investigations demonstrate that UP and UIT are effective and practical means to improve the service life of new and existing steel bridges, to increase their fatigue strength and last but not least for repair procedures. Further investigations will be carried out in order to derive reliable SN-curves even for higher strength steels.

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Bridge owners benefits from probability-based assessment and maintenance management

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Principles for a guideline for probability-based management of deteriorated bridges

Jørn Lauridsen & John Bjerrum
The Danish Road Directorate, Ministry of Transport, Denmark

Mette Sloth
COWI A/S, Denmark

Finn M. Jensen
Ramboll, Denmark

ABSTRACT: This paper describes how the Danish Road Directorate, Ministry of Transport is developing a guideline for the procedure for probability-based management of deteriorated bridges. This guideline is a supplementary guideline to "Reliability-based classification of the load carrying capacity of bridges".

The main objective of a probability-based maintenance management plan is to make it possible for a bridge owner to extend the lifetime of a deteriorated bridge or bridge component. The core principle in probability-based bridge maintenance management is to make rational decisions on the maintenance management taking into account the inherent uncertainty. The application of probability-based maintenance management for special deteriorated structures has proven very economically beneficial in cases where the deterministic options of repair or replacement would prove extremely costly.

The guideline will focus on the procedure for carrying out probability-based maintenance management of deteriorated bridges, including the inspection and sampling required to model deterioration rates, modelling of deterioration rates, comparative evaluation of possible actions and so forth. It will also include setting up policies and possibilities for maintenance and guidance on continuous updating and validation of the safety-based management plans. The supplementary guideline is made in order to extend the use of the method and is completed early 2006.

The DRD fully supports the use of probability-based bridge management, and the supplementary guideline is made both in order to unify and in order to disseminate the procedure among consultants working for the DRD.

The DRD has been one of the first bridge managers to utilize probability-based bridge management for making rational decisions regarding the increase of load capacity on bridges and for postponing costly repair and rehabilitation options. From the point-of-view of the DRD, applying safety-based bridge management is an important approach for extending the service lifetime, reducing or postponing costly rehabilitation projects thereby supplying more degrees of freedom when managing the entire bridge stock. The main result has been major cost savings and a reduction of uncertainty on future maintenance and repair needs thereby making overall long-term budgets more accurate.

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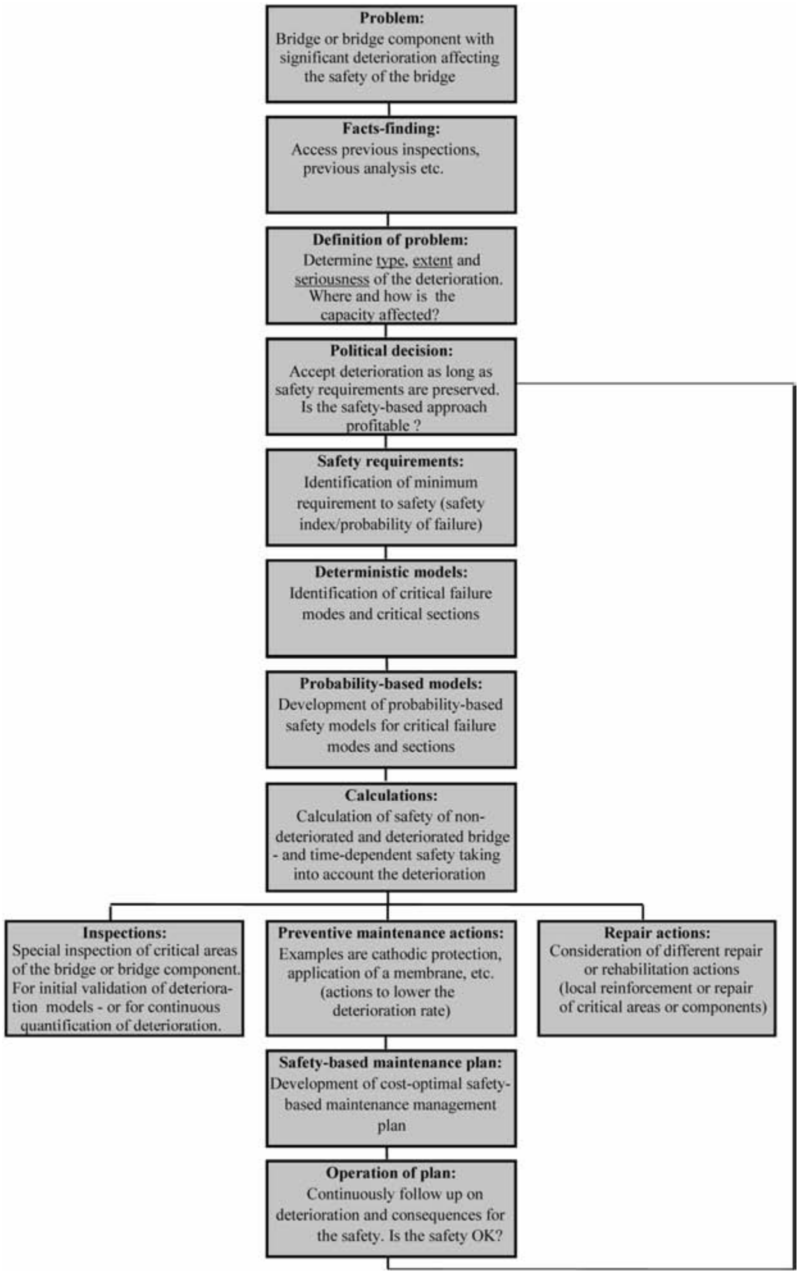

Figure 1.

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The intended work flow or procedure for establishing a safety-based maintenance management plan is shown in the figure above. The tasks in the work-flow also indicates the principles that need attention in the development of a guideline for safety-based management of deteriorated bridges.

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Experience with probability-based assessment of bridges based upon the Danish Guideline

Jørn Lauridsen & J. Bjerrum
The Danish Road Directorate, Ministry of Transport, Copenhagen, Denmark

A.J. O'Connor & I. Enevoldsen
RAMBOLL, Virum, Denmark

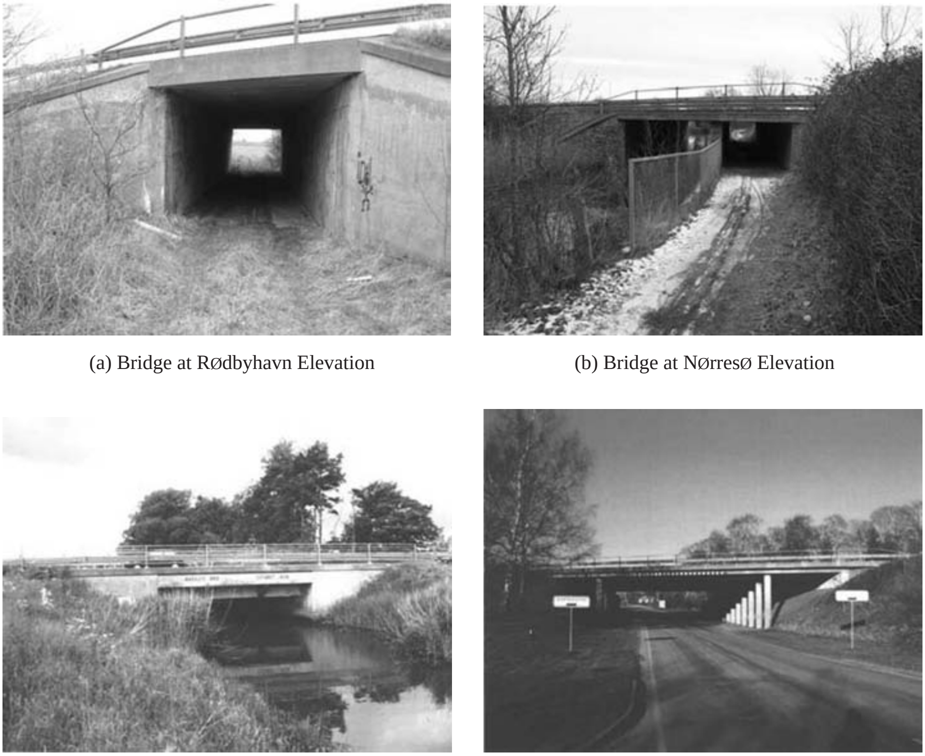
This paper describes the application of the newly developed Danish guideline to a probability-based assessment of 4 bridges in Denmark. The guideline was developed by the Danish Roads Directorate with RAMBOLL as one of the consultants. For each structure analysed, modelling of the critical limit states is presented as are the statistical techniques employed in modelling the loads and resistances. The overall aim of each analysis was to achieve a higher load rating for the structures than the results of deterministic analysis. Ultimately the cost benefits to bridge owners/managers of performing a probabilistic assessment are apparent from the results, which provide a higher load rating for the bridges than were achieved through deterministic assessment. While the guideline is intended to be applied in probability-based assessment of Danish bridges it is anticipated that it could equally be applied in other countries.

Figure 1(a) presents a simple slab bridge located at Rødbyhavn. The bridge built in 1942 has a structural form consisting of a simple span, which carries the motorway over a disused railway. The deterministic classification was governed at ULS by the bending capacity of the slab. Figure 1(b) presents a slab bridge located at Nørresø. The bridge built in 1942 has a structural form consisting of two continuous spans, carrying the motorway over a footpath and canal. The deterministic assessment of the structure was governed at ULS by the bending capacity of the slab. Figure 1(c) presents a beam and slab bridge located at Åkalve. The bridge built in 1935 has a structural form consisting of a span of 8 m, which carries traffic over Tuse Å. The deterministic assessment of the structure was governed at ULS by the bending capacity of the longitudinal edge beams. Figure 1(d) presents a prestressed post-tensioned slab bridge located at Nystedvej. The bridge built in 1959 has a structural form consisting of three spans. The deterministic assessment of the structure was governed at ULS by the hogging bending capacity of the slab at the outermost column support. The results of the deterministic load rating of each of the structures are presented in Table 1.

Recognising the inherent conservatism of the deterministic assessment it was decided to perform a probability based assessment of the structures to determine their formal level of safety in accordance with the relevant safety standards. This probabilistic assessment involves modelling the variables of load and resistance as random and identifying the formal probability of failure of the structures based on the FORM technique. This *'site-specific'* failure probability is then compared with minimum allowable value permitted, to ensure the relevant safety criteria are met. It is

Table 1. Deterministic classification results.

Passage type	Normal passage	Restricted passage 1	Restricted passage 2	Restricted passage 3
Rødbyhavn	70	70	100	150
Nørresø	50	50	80	200
Åkalve	80	80	100	200
Nystedvej	80	150	175	200



(a) Bridge at RØdbyhavn Elevation

(b) Bridge at NØrresØ Elevation

(c) Åkalve Bro Elevation

(d) Nystedvej Bro Elevation

Figure 1. Analysed bridges.

Table 2. Guideline safety requirements.

Failure consequences (Safety class)	Failure type I: Ductile failure *with* remaining capacity	Failure type II: Ductile failure *without* remaining capacity	Failure type III: Brittle failure
Very serious: High safety class	$P_f \leq 10^{-5}$ $\beta \geq 4.26$	$P_f \leq 10^{-6}$ $\beta \geq 4.75$	$P_f \leq 10^{-7}$ $\beta \geq 5.20$

important to stress that at no stage is the safety of the structure compromised, rather the process derives a bridge specific code, removing the conservatism inherent in deterministic codes, which must necessarily generalise to be widely applicable.

The requirements at the ultimate limit state for the structural safety are specified with reference to failure types and failure consequences, i.e. safety class with requirements for the formal yearly probability of failure P_f. Table 2 outlines the requirements of the Danish safety assessment guideline in this regard. For the structures presented P_f corresponding to *Failure Type II – Ductile Failure* was selected.

The probabilistic assessment of the structures at their critical limit states demonstrated that they all had sufficient capacity to achieve a Class 100 rating. The results represent a significant saving for the bridge owner both in terms of the direct replacement cost and of the indirect costs, which would have been incurred in replacing the structures. It is important to stress that at no stage has the safety of the structures been compromised, rather a bridge specific safety assessment, free from the generalisations of deterministic codes, has resulted in considerable savings for the owner.

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The Öland bridge – a case study for probability-based service life assessment

Mette Sloth & Ruth Sørensen
COWI A/S, Kongens Lyngby, Denmark

Adriano Maglica
Vägverket Region Sydöst, Jönköping, Sweden

1 INTRODUCTION

The Öland Bridge is a 6,072 m long concrete girder road bridge that connects the island of Öland with the mainland of Sweden. The bridge was constructed in the period 1968 to 1972. Only about ten years from the opening, the bridge piers showed the first signs of deterioration due to the fact that the concrete piers were not sufficiently resistant to the very aggressive environment, and the substructure were rehabilitated during the 90'ties.

Comprehensive technical and economic investigations have been carried out in 2004, with the main purpose of preparing a maintenance strategy for a period of 70 years. The present paper is only focusing on the maintenance plan for the superstructure – main girder and bridge deck.

Basis for the maintenance strategy was existing data from earlier inspections and material investigations. The latter included measurements of the concrete cover, the carbonation depth and chloride profiles. The visible damages were identified to spalling, rust deposits, corroded reinforcement and cracks both with and without white deposits.

Evaluation of the available data material lead to the conclusion that two deterioration processes were relevant to be considered: Corrosion due to carbonation and chloride penetration. Important tasks for the choice of an optimum maintenance strategy are the extent of possible repairs and when to carry out the repairs. In order to be able to make a best estimate of the extent and the time for doing a possible repair a service life assessment was carried out. The paper is focusing on this part of the maintenance management plan for the Öland Bridge.

2 PROBABILITY-BASED EVALUATION OF CORROSION RISK

The risk of chloride induced corrosion is highly relevant to assess and previous concrete investigations have indicated that also carbonation is a potential problem.

In 1994 an investigation of the chloride content of the main girders of the low-level bridge and the superstructure of the high-level bridge was carried out. Evaluating these data revealed the following deficiencies considering the possible use in a lifetime assessment during 2004: In general a very limited data basis taking into account the size of the bridge is available, some of the chloride profiles showed atypical variations with the concrete depth and for some of the profiles the chloride content in the reinforcement depth had already exceed the critical chloride content.

Taking this into account it was chosen not to apply a probabilistic approach. Instead a more pragmatic approach was applied. The result of this analysis was that the risk of chloride induced corrosion is significant and it has been suggested to established a new service life program for the future assessments, see section 3.

In recent years it has been realised that carbonation may be a significant risk for corrosion of the reinforcement in the main girders of the low-level bridge due to a very porous concrete (w/c up to

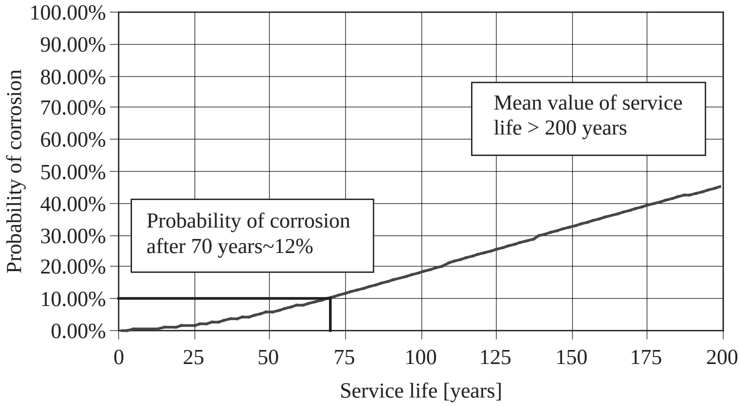

Figure 1. Calculated probability of carbonation induced corrosion as a function of the time.

0.6). A study of the carbonation depth has therefore been carried out in 2004 taking into account measured carbonation depths (measured in 1999) from different locations, longitudinal placing, the north and south side and inside or outside of the main girder. It was clear from the measurements that the carbonation depth varies significantly and therefore a probabilistic approach was obvious. Worst case result from the analysis is shown in figure 1.

As seen from figure 1 the mean value of the remaining lifetime considering carbonation induced corrosion is larger than 200 years and the probability of corrosion at the ending time for the maintenance strategy is 12%. However, it is also seen from figure 4 that the remaining lifetime is subject to significant uncertainty. In general, it was found that the expected value of the remaining lifetime was very high and that the probability of corrosion at the ending time for the maintenance strategy was acceptable (0.8–12%). It was therefore concluded that even though the measured carbonation depths varies significantly there is still sufficient reliability even after 70 years.

3 PLAN FOR FUTURE MAINTENANCE MANAGEMENT AND CONCLUSION

The dominating deterioration process for the reinforced concrete is expected to be chloride ingress. The plan for future maintenance management has therefore been concentrated on this deterioration mechanism. Since the risk of having corrosion due to carbonation is much lower compared to the risk of chloride induced corrosion it is ensured that the maintenance plan will result in a repair decision before carbonation becomes a problem. The suggested management plan for corrosion consists of two parts:

– A corrosion monitoring system
– A probability-based model for the remaining lifetime including a relevant data basis like chloride profiles and concrete cover measurements.

The benefit from the planned future maintenance management will be the possibility to follow the deterioration development continuously and take action in due time. A probability-based approach will ensure that the uncertainties are quantified and included in the conclusions. Further, the approach facilitates the possibility to combine best knowledge from the past and any new information which will become continuously available in a consistent way applying a Bayesian approach.

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Probabilistic-based assessment of a concrete arch bridge

A.J. O'Connor, C. Pedersen & I. Enevoldsen
RAMBOLL, Virum, Denmark

J. Bjerrum
The Danish Road Directorate, Ministry of Transport, Copenhagen, Denmark

This paper describes the techniques employed in the probabilistic assessment of a concrete arch motorway bridge. The probability-based classification of the structure serves as an example of how the newly developed Danish guideline for probability-based assessment of bridges is applied. Modelling of the critical limit state is presented. The statistical techniques employed in modelling the traffic loads are presented. The overall aim of this analysis was to achieve a higher load rating for the structure than that resulting from a deterministic analysis. The sensitivity of the reliability index to the modelled stochastic variables is presented. Ultimately the cost benefits to bridge owners/managers of performing a probabilistic assessment are apparent from the results, which provide a higher load rating for the concrete arch bridge than that achieved through deterministic assessment.

Avdebo bridge is reinforced concrete arch bridge built in 1932. The structural which is skewed at 56.6° has a clear span of 23.0 m and height 3.2 m. The arch bridge carries two lanes of traffic over a river, Figure 1. The width of the structure is 9 m. The arch thickness varies from 0.3 m at the crown to 0.6 m at the base. The wing walls on either side of the arch have length 5.9 m, giving a total structure length of 36.3 m. The wing walls are supported by a single rib on either side of the bridge, where the bridge has 4 ribs in total.

Deterministic assessment of bridge assigned Class 50 in accordance with the Danish Road Directorate Assessment (DRD) Code (Vejdirektoratet 1996). The implication of this deterministic classification was that the structure had insufficient carrying capacity to receive the rating necessary for the Danish *Blue Motorway Network*. As it is the aim of the DRD to achieve this rating for all motorway structures the alternatives presented for a structure failing to achieve a Class 100 rating are costly rehabilitation or replacement.

Recognising the inherent conservatism of the deterministic assessment it was decided to perform a probability based assessment of the structure to determine its formal level of safety in accordance with the recently produced Danish Roads Directorate *Guideline for probability-based assessment of bridges* (Vejdirektoratet 2003).

The critical limit state for the arch is reached when the capacity of the footings is exceeded in combination with the formation of two internal hinges within the arch. The capacity of the arch

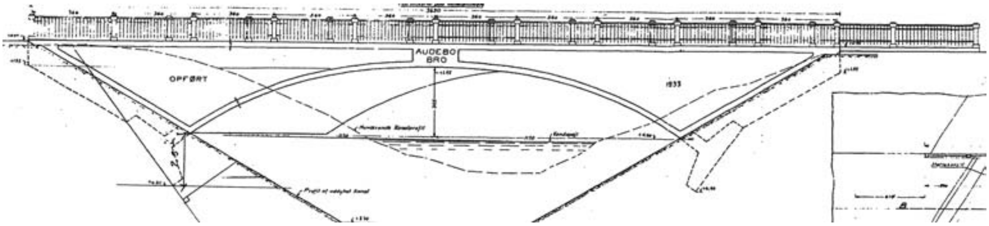

Figure 1. Avdebo bridge elevation.

Table 1. Guideline requirements (Vejdirektoratet 2003).

Failure Consequences (Safety class)	Failure type I: Ductile failure *with* remaining capacity	Failure type II: Ductile failure *without* remaining capacity	Failure type III: Brittle failure
Very Serious: High safety class	$P_f \leq 10^{-5}$ $\beta_t \geq 4.26$	$P_f \leq 10^{-6}$ $\beta_t \geq 4.75$	$P_f \leq 10^{-7}$ $\beta_t \geq 5.20$

M_{cap} is therefore a function of (f_{cu}, f_y, SBC, Gs) with f_{cu} the characteristic concrete strength, f_y the steel yield strength, SBC the bearing capacity of the soil and Gs the soil density.

The requirements at the ultimate limit state for the structural safety are specified with reference to failure types and failure consequences, i.e. safety class with requirements for the formal yearly probability of failure p_f. Table 1 outlines the requirements of the Danish safety guideline in this regard.

For the arch structure under consideration in this paper the assessment criteria is defined by Failure Type II, Ductile failure without remaining capacity.

Determination of the ULS capacity of the arch is performed using the program FRAME. FRAME assesses the critical load factor for failure of the arch based upon the formation of a mechanism (i.e. 4 plastic hinges in the arch).

For the purpose of the probabilistic analysis it was decided to derive a response surface for the critical load factor for the arch based upon 5 principal random variables which were (i) the intensity of the applied load, (ii) the concrete strength, (iii) the steel strength, (iv) the soil bearing capacity and (v) the density of the soil. The response surface was trained using a total of 405 combinations of values of the above mentioned variables, three each for the intensity of the applied load, concrete strength, soil bearing capacity and soil density and five values of steel strength.

Reliability analysis is carried out using the software package PROBAN. Determination of the probability of failure p_f is performed using the FORM techniques. Model uncertainties were incorporated in the analysis according to the guideline for probability-based assessment.

The minimum safety indices computed by PROBAN for the normal passage case was:

$$\boxed{\text{Normal Passage Class 100: } \beta = 5.44 > 4.75}$$

As the computed value is greater than the required minima the structure is considered adequate to receive the Class 100 load rating. The justification for this conclusion comes from the fact that: (1) the probabilistic live loading model is more realistic for the structure under consideration, (2) the resistance models for the structures are more realistic taking account of the natural variability of the variables which contribute to the structural resistance and thereby reducing the partial factors required.

The result represents a significant saving for the bridge owner both in terms of the direct replacement cost and of the indirect costs which would have been incurred in replacing the structures. It is important to stress that at no stage has the safety of the structure been compromised, rather a bridge specific safety assessment, free from the generalisations of deterministic codes, has resulted in considerable savings for the owner.

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Probability-based maintenance management plan for corrosion risk – a case study from Faro Bridges

Mette Sloth & Birit Buhr Jensen
COWI A/S, Kongens Lyngby, Denmark

Erik Stoltzner
The Danish Road Directorate, Ministry of Transport, Denmark

ABSTRACT: The Faro Bridges (1985) were constructed using state-of-the-art concrete technology, using dense concrete with high quality materials and using state-of-the-art production and curing technology. In the late 80'ies dust samples were taken and analysed for chloride content. The chloride ingress was somewhat higher than expected. This lead to a number of concrete investigations with the purpose of assessing the time dependent corrosion risk due to chloride penetration. Partly due to the very extensive data basis, the Faro Bridges have also been in front adopting the probability-based methods for maintenance management. As early as 1996 the first evaluations, taking the uncertainties into account, were carried out. Since then the data basis has grown significantly and in 2005 a probability-based evaluation of the residual service life has been carried out. The paper presents the extensive data basis and demonstrates how the information has been included in the probability-based plan for maintenance management. Further the benefits for the bridge owner are highlighted.

In Denmark one of the major durability problems considering concrete bridges is chloride induced corrosion. This is partly due to the use of de-icing agents on the roads during the winter period and partly due to the many bridges exposed to a marine environment. Assessment of corrosion risk due to chloride penetration has therefore during the last decade been a key issue for maintenance management of the bridges operated by the Danish Road Directorate (DRD).

When planning the necessary maintenance activities for large bridges like the Faro Bridges it is important to decide on the different condition stages that triggers different maintenance strategies. The Faro Bridges constitute a large investment and a land mark for DRD and cost benefit analysis has shown that a preventive maintenance strategy is more feasible when compared to traditional repair methods. For this reason a preventive maintenance strategy has been adopted. From this point of view it is the initiation of corrosion that describes the service life which in this case is the point in time were maintenance action is called for considering the concrete columns. It is well-known that the prediction of the initiation time is subject to significant uncertainty. Assessment of the service lifetime has therefore been based on probability-based calculations taking the uncertainties into account.

In 1996 the first probability-based calculations were carried out. The results are presented in Table 1. Time intervals are given as the analysis included probability-based calculations for all measured chloride profiles.

The conclusions were that the on-going monitoring of the chloride penetration should continue and further investigations were necessary.

In 2005 the probability-based calculations were updated taking the new information available at this point in time into account. The probability of having corrosion initiation in the tidal zone in 2004 (where the last chloride profile is from) is below 5% for the concrete surfaces and approximately 10% for the corners. The expected time to onset of corrosion is in year 2020 and 2037 considering the surface and corners of the columns, respectively.

Table 1. Service life estimation (corrosion initiation), 1996.

Percentiles	50%	5%	1%
Number of years	18–55 years	8–15 years	0–10 years

The probability-based assessment of the time to onset of corrosion for the Faro Bridges has been described as a "point" in the structure. When planning maintenance activities it is however not discrete points but moreover a description considering concrete surfaces that is needed. It would also be beneficial if all relevant inspection results could be introduced in this spatial modeling of the deterioration. In the recent years a framework for this has been developed. A short introduction to the methodology is given in the following presenting the main actions for implementing the framework.

- The probabilistic modeling of degradation stages is described considering localized deterioration.
- Formulation of framework where the condition indicators (parameters that provide information on the state of the structure with respect to the considered deterioration mechanism) are quantified such that they can be included in the decision support. Hence it is necessary for all relevant condition indicators (e.g. in-service condition indicators such as visual inspections and half cell potential measurements) to quantify the probability that the inspected component is in a certain state given correct indication and the probability that the inspected component is in a certain state given spurious indication.
- The spatial variability is represented by dividing the concrete surface into sub-areas with homogeneous deterioration characteristics and a further subdivision of these areas into elements which thereafter define the areas for which a given inspection result should be considered representative.
- The interdependency between the different elements is caused by the common influencing variables like the diffusion parameters and the critical chloride content. These are termed the global parameters.
- When inspection results become available, the probability of the condition state of the inspected element is updated first using Bayes' law.
- Secondly all other elements are updated by updating the distribution of the global parameters also applying Bayes' law.

The framework is also developed with the possibility to optimize the expected cost of different inspection and repair strategies. For this purpose it is necessary to formulate typical decision rules for when to carry out the considered repairs. Further the inspection strategy needs to be described by the methods and the extent for different inspections. A cost model is also to be established quantifying the cost of the different repair and inspection methods.

The framework is planned for in the near future and the Faro Bridges will possibly be the first case study of this new approach.

Reinforced concrete bridges are by far the most common road bridge type in Denmark. The DRD has therefore always focused on how to improve the management of their concrete bridges. Special focus is paid to the assessment of the corrosion risk due to chloride penetration as this is the deterioration mechanism in Denmark resulting in the majority of damages and thereby major costs for repair.

A repair of a bridge will very often result in a dislocation of the traffic. The traffic intensity in Denmark is dense and the DRD has therefore also a need to minimize the traffic disruption.

The presented approach for probability-based maintenance management is one of the initiatives that the DRD has initiated and is developing. As the road operators are facing requirements for optimal maintenance the use of the probability-based approach for maintenance management of bridges is foreseen to become state of the art in the future.

Reliability and risk management

*Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Reliability based assessment of the influence of concrete durability on the timing of repair for RC bridges

M.G. Stewart & J.A. Mullard
Centre for Infrastructure Performance and Reliability School of Engineering, The University of Newcastle, NSW, Australia

1 INTRODUCTION

For many designers and asset owners, the timing of maintenance and repair for reinforced concrete (RC) bridges can be a major economic and logistical consideration. The primary cause of deterioration in RC bridge decks is poor concrete durability (low concrete quality and/or insufficient cover) leading to reinforcement corrosion. The corrosion products have a volume three to six times greater than the original steel, leading to tensile stresses within the concrete and resulting in longitudinal cracking and spalling at the surface. This type of deterioration is of significant concern to asset owners, and it is estimated that at the time of severe cracking the structural capacity is reduced by no more than 10 to 20% so at the time of severe cracking loss of safety is not significant. Hence, it is appropriate to view severe cracking as an important and costly mode of failure for the estimation of service life of RC structures.

It is recognised that the material and dimensional properties of a concrete structure will not be homogenous due to the spatial variability of workmanship, environmental and other factors and as such, corrosion damage will occur spatially over any exposed surface. Considering this, a two-dimensional spatially variable time-dependent reliability analysis is developed to predict the likelihood and extent of cracking for RC surfaces exposed to chloride ion attack.

The present paper presents results from a 2D spatial time-dependent reliability analysis of an RC bridge deck exposed to an aggressive marine environment. This model will consider the random spatial variability of concrete properties, concrete cover and surface chloride concentration and will allow the prediction of corrosion-induced crack widths up to 1 mm. The analysis considers an existing random field model (Vu & Stewart 2005) with an improved crack propagation model (Vu et al. 2005) and the effect of durability design specifications, surface area and limit crack width. The outcomes are presented in terms of the probability that at least x% of a concrete surface has severe cracking, $\Pr(d_{crack}(t) \geq x\%)$. A typical maintenance strategy is a patch repair where deteriorated concrete is removed and replaced with a new repair material. In the Netherlands the criterion for the onset of a patch repair is usually based on 0.5%–1.5% of the surface area exhibiting visual signs of concrete damage. Thus, the first repair will occur when the extent of damage is observed to exceed $X_{repair} = 1\%$ of total surface area. The present paper uses this as the criterion for the timing of first repair.

2 EXAMPLE APPLICATION: REINFORCED CONCRETE BRIDGE DECK

The example considered herein is a RC bridge deck located adjacent to the coast and subject to atmospheric sea-spray. The RC bridge deck is of area (A) of 900 m^2. Reinforcing bars of 16 mm diameter are spaced at 250 mm centres. The likelihood and extent of cracking of the top of the deck will be evaluated at annual time increments over 120 years.

Results from the spatial time-dependent reliability analysis can be shown as probability contours as shown in Figure 1, for surface areas (A) of 36 m^2 and 900 m^2 and fair durability design specifications (cover = 50 mm, w/c = 0.5). The probability contours represent $\Pr(d_{crack}(t) \geq$

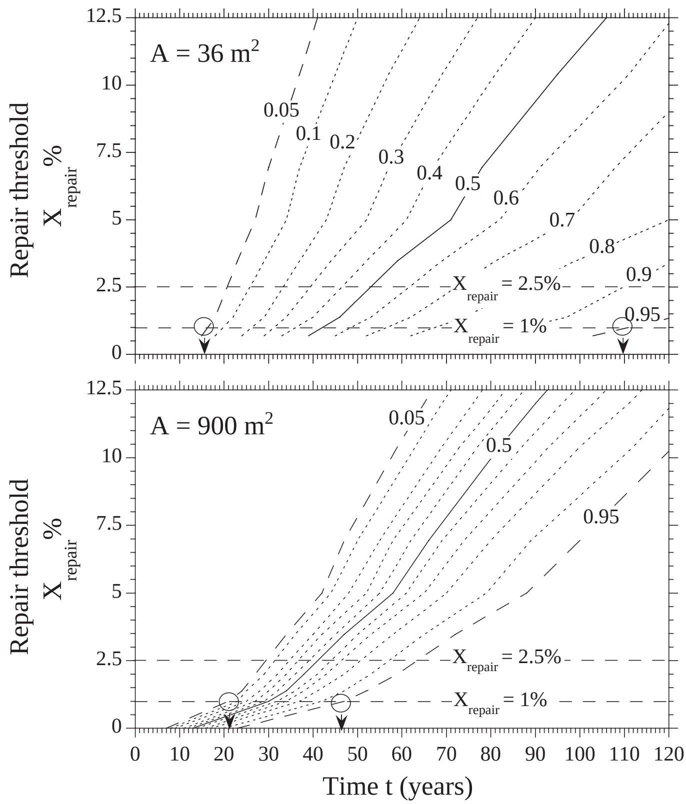

Figure 1. Probability contours for $\Pr(d_{crack}(t) \geq X_{repair}\%)$, for different surface areas and fair durability design specification.

$X_{repair}\%$) and can be used to predict the probability of cracking damage for any repair threshold. Figure 1 shows, for example, that for $X_{repair} = 1\%$ there is a 90% probability that a first repair will be needed between 16 years and 110 years (probability of occurrence between 0.05 and 0.95), for $A = 36\,m^2$ (see Figure 1). If the repair threshold is increased to $X_{repair} = 2.5\%$ there is a 90% probability that first repair will be needed between 20 years and in excess of 120 years. Hence, selecting a less stringent repair threshold will defer repair actions by at least 4 years. If the surface area is larger ($A = 900\,m^2$), then for $X_{repair} = 1\%$ there is a 90% probability that damage will occur between 21 years and 47 years.

This information on the time-dependent structural performance and reliability can be extremely valuable to designers and decision makers. It allows predictions of the time to first maintenance to be considered at the design and costing stage, and is useful in optimising inspections and life-cycle cost management (e.g., Stewart 2005).

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Reliability-based calibration of dynamic load allowance of bridge by numerical simulation

T.J. Chung
Department of Civil Engineering, Halla University, Won-Ju, Korea

C.R. Lee, D.K. Shin & Y.S. Park
Department of Civil & Environmental Engineering, Myongji University, Yong-In, Korea

ABSTRACT: The reliability-based calibration of dynamic load allowance (DLA) of bridge is performed by numerical analysis of vehicle-bridge dynamics. The road surface roughness, bridge dynamics, and vehicle dynamics are included in the simulation program. The road profiles are generated by using a Fourier transform of the power spectral density function. The bridge superstructure is modeled three dimensionally using beam, truss, and shell finite elements. Modal equations of motion are formulated by using 50 fundamental modes of vibration. Three different types of vehicles are modeled using the tire enveloping model. In the calculation of DLA, typical plate-girder bridges and prestressed concrete beam bridges with various span lengths are considered. For each bridge, 10 sets of road profiles are generated and hundreds of simulations are performed to obtain the mean and the coefficient of variation of DLA. Based on the resulting statistical parameters, the DLA of bridge is calibrated.

1 INTRODUCTION

In the present numerical simulation program, improvements on the three-dimensional vehicle model are made by modeling the suspension system in the rear axle as two axles and the tire as surface contact element with roadway surface. Two to five axle vehicles are modeled using the tire enveloping model. The parameters influencing the vehicle dynamics are obtained from the vehicle manufacturers and used in the bridge-vehicle interaction equations. The roughness of roadway surface is generated by using a Fourier transform of the PSD function in the form of exponential function. The mean value of roughness coefficient previously measured for 29 Korean highway bridges is used in generating random roadway profiles. The bridge superstructure is modeled three dimensionally using beam, truss, and shell finite elements. Modal equations of motion are formulated by using 50 fundamental modes of vibration. The numerical program developed herein is verified by comparing the calculated DLA with available previous experimental ones. In the calculation of DLA of typical plate-girder bridges and prestressed concrete beam bridges with various span lengths, 10 road profiles for each bridge are generated to reflect the random nature of roadway profile. Hundreds of simulations are performed and the mean and the coefficient of variation of DLA are obtained. Based on the statistical parameters, the DLA is calibrated using the procedure proposed by Bahkt and Pinjakar (1989).

2 PROGRAM VERIFICATION

The dynamic responses by the present procedure are compared with the experimental results available in the literature (Whitmore 1970; Fenves et al. 1962) to validate the present program. Figure 1 shows the maximum impact factor of the bridge by the present program and by the experiment (Whitmore 1970). The current numerical analysis using tire enveloping model agrees well with the

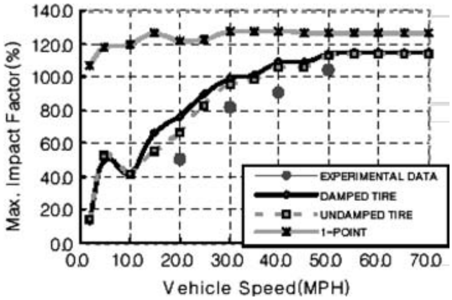

Figure 1. Comparison of maximum impact factor with experiment by Whitmore (1970).

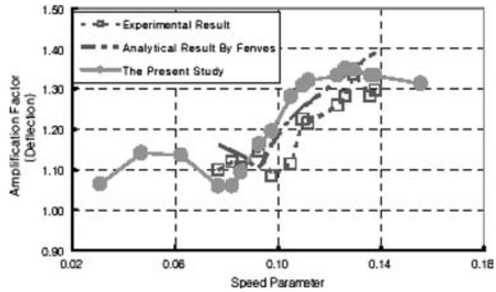

Figure 2. Comparison of amplification factor with AASHO test (1962).

Table 2. DLA statistics based on numerical analysis.

Bridge	Span (m)	Mean	COV(%)
PC-beam	20	9.89	39.55
	25	11.2	46.28
	30	19.19	64.35
Plate-girder	20	9.9	38.25
	30	20.63	39.07
	40	26.67	31.15
Total		16.25	43.11

experimental result while the numerical result by the point-contact tire model overestimates the impact factor especially at the relatively slower vehicle speed. The dynamic amplification factors of the bridge obtained by the measurement of AASHO load test, by the analysis of Fenves et al. (1962), and by the present program are presented in Figure 2. It can be observed from Figure 2 the current program gives slightly higher values of dynamic amplification factor compared with experimental results but the two results agrees well with each other.

3 STATISTICS AND CALIBRATION OF DLA

Using the numerical program, the DLA for simple-span PC-beam and plate-girder bridges with various span lengths are calculated. Gross weights of two-axle truck, three-axle truck, and five-axle tractor-trailor are 160 kN, 300 kN, and 420 kN, respectively. Ten sets of roadway roughness are generated using the roughness coefficient of 64×10^{-6} m^2/cycle/m that corresponds to the average value of road surface roughness coefficient for 29 Korean highway bridges for which road profiles are previously measured.

The analysis results for typical PC-beam and plate-girder bridges are summarized in Table 2. Based on the DLA statistics in Table 2 and using the formula proposed by Bahkt and Pinjakar (1989) with parameters, $\beta = 3.5$, $\text{LL}\ \alpha = 1.75$, and $s = 0.57$, the calibrated value *DLAs* is calculated to be 17.3%.

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An application of the probabilistic SBRA method in bridge structures design

D. Pustka & P. Marek
Technical University of Ostrava, Faculty of Civil Engineering, Department of Structures, Ostrava, Czech Republic

P. Marek
Czech Academy of Sciences, Institute of Theoretical and Applied Mechanics, Prague, Czech Republic

ABSTRACT: The development of computer and information technologies allows for researchers/engineers to consider the transition from the current structural reliability methods applied in codified design to the probabilistic methods based on simulation technique and applicable to the designer's everyday work. This paper focuses on the application of one such method, the Simulation-Based Reliability Assessment (SBRA), documented in Marek et al (1995, 2003). An example of the reliability (safety) assessment of a reinforced concrete cross-section exposed to a combination of normal force and bending moment is demonstrated and discussed.

1 INTRODUCTION

The increasing number of papers dealing with probabilistic structural reliability assessment indicates a transition towards qualitatively new methods of reflecting the advances in reliability theory and the potential of computer technology, see, e.g. Schuëller & Spanos (2000), Marek et al (2002). One such method, the SBRA (Simulation-Based Reliability Assessment), see Marek et al (1995, 2003), using the direct Monte Carlo technique, is applied in the following text to outline the main issues related to the development and introduction of non-traditional methods applicable by bridge/structural designers. Using such an approach, the designer can analyze the interaction of numerous variables involved in the probabilistic assessment procedure without applying complicated mathematical operations. Numerous recent publications focused on the application of the Monte Carlo simulation (see e.g. Schuëller & Spanos (2000), Marek et al. (2002)) and they indicate the potential of such concepts.

2 SBRA METHOD

Using the SBRA method (see textbooks Marek et al (1995, 2003)), all input variables are represented by truncated non-parametric distributions since they can express the actual physical limitations of the variables better than the common parametric ones. The reliability evaluation is based on the analysis of the interaction between the reference function R and the load effect S. This interaction is evaluated using the stochastic simulation and the Monte Carlo technique. The corresponding empirical distribution in the (R, S) space is created and represented by a set of dots ("ants") corresponding to the number of simulation steps. The space ("anthill") is divided into the "safe" and the "unsafe" domains by the line $R - S = 0$, and the probability of failure P_f can be computed by counting the sum of the dots in the unsafe domain divided by the total number of dots. Introducing the time dependent magnitudes R(t) and S(t), the analysis can be extended to the durability assessment problems, i.e., to the determination of the time dependent probability of failure function $P_f(t)$, see

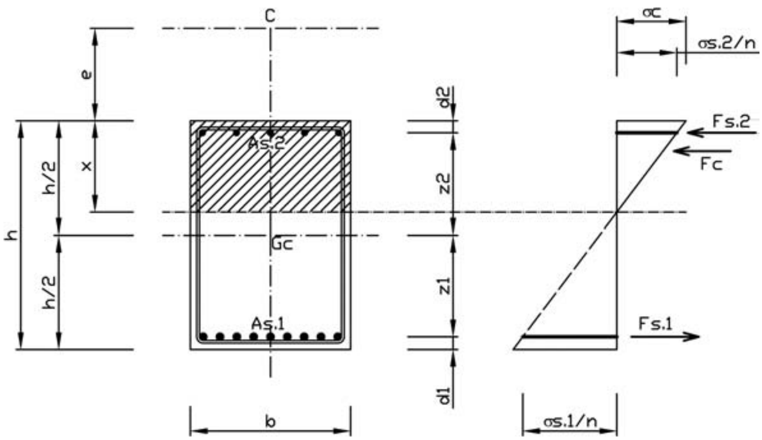

Figure 1. Scheme of reinforced concrete cross-section exposed to a combination of normal force N_S and bending moment M_S.

Marek et al (2002, 2003). The safety is expressed by comparing the probability of failure P_f with the target probability P_d contained in codes. More information regarding the SBRA method can be found by the reader on the internet page www.sbra-anthill.com.

3 APPLICATION OF THE SBRA METHOD

The full paper on CD ROM includes a focused study on the probabilistic reliability assessment of a reinforced concrete cross-section (see Fig. 1) exposed to a combination normal force N_S and bending moment M_S, where M_S and N_S are resulting from combination of numerous mutually independent variable loads. Individual input quantities are considered as random variables, expressed by bounded non-parametric distributions histograms. Using the Monte Carlo simulation technique, the safety of the cross-section is evaluated.

ACKNOWLEDGEMENT

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Harnessing social perception of a bridge's condition

J.D. Birdsall & E. Brühwiler
Swiss Federal Institute of Technology Lausanne (EPFL), Switzerland

ABSTRACT: Operating and maintaining a bridge is a union of two dichotomous entities: the analytical civil engineer and the experiencing public. In a perfect society, the concerns of the former would be actively and completely supported by the later, but in practice this support is commonly incomplete. This paper employs findings from the field of applied-psychology, heuristics in particular, and an example series of interactions with the Brooklyn Bridge to detail how social perception of a bridge's condition evolves over time. From these findings, the authors propose to introduce partial high-quality but incomplete maintenance actions, implemented prior to a bridge reaching a critical state, to serve as an evaluation benchmark to harness and align the social perception of the bridge's condition with that of the analytical engineer.

1 EVOLVING EVALUATION PROCESS

From existing research it can be seen that people employ the relative change of a parameter, and not the current state of the parameter, in the evaluation process (Kahneman and Tversky, 1979). Additionally, when two items are evaluated against each other it is only the characteristics they share that influence the valuation process (Hsee, 1996). People also employ their previous set of experiences, their evaluation norm, as an anchor upon which they make minor adjustments to form their expected performance for a subsequent interaction (Tversky and Kahneman, 1974). Furthermore, people exhibit heightened valuation sensitivity to probabilities that approach zero and absolute probability (Kahneman and Tversky, 1979). Finally the reformation of the evaluation norm following an additional interaction is equivalent to the average of the peak and end occurrence valuations (Kahneman, 2000).

2 EXPERIENCING AN EVOLVING EVALUATION

When an individual interacts with and evaluates his environment, the evaluation process is limited to parameters personally experienced. For a civil engineering structure, such as the Brooklyn Bridge, this set of evaluation parameters include such items as the architectural appeal, the traffic volume, the surrounding views, the walkway surface, and the general cleanliness. Additionally, it is not the bridge's current condition that is employed in the evaluation process, for a number without a norm is valueless. Rather it is the structure's changing performance with respect to the individual's evaluation norm, formed from the individual's experience with the given bridge and similar bridges.

3 HARNESSING SOCIAL EVALUATION

The first step is to observe that the majority of parameters society interacts with are beyond the influence of the civil engineering field. The second step is to recognize the incongruent evolutionary rates of a bridge's deteriorating condition and society's adaptation to this changing environment.

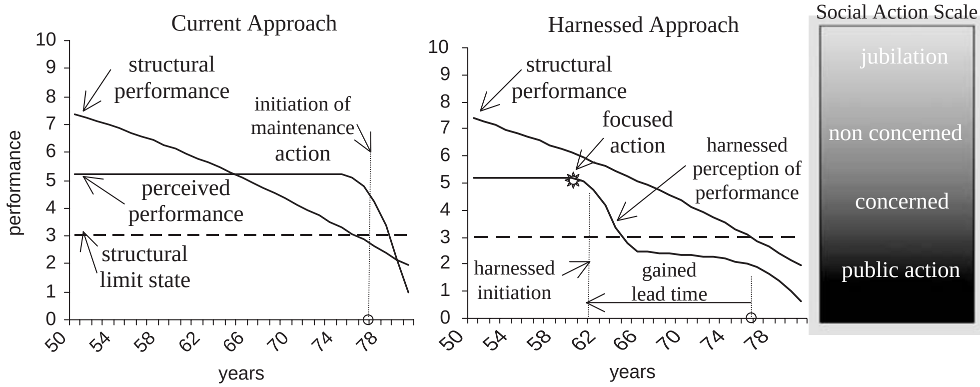

Figure 1. Gained maintenance lead time by employing a focused incomplete action.

Bridges commonly have life-spans of 80 or additional years but society is able to adapt to and with a changing environment in a matter of weeks or months. Therefore, society only recognizes and considers the condition of a bridge when the structure is newly built, when infrequent critical events occur or when the structure reaches a critical deterioration state.

Civil engineers communicate to society by employing their voices to advise society of potential dangerous situations (ASCE, 2005) and by limiting or closing bridges in critical condition (Hartle, 2002). The authors believe that these limited communication avenues are insufficient to adequately express the pressing issues facing the existing infrastructure. The authors therefore propose to employ focused, high quality but incomplete maintenance actions to elements society regularly interacts with at a point in time when the bridge is approaching an increased deterioration state. These actions will provide society with a contrasting evaluation benchmark against which the deteriorating bridge can be evaluated. An example of a partial action is to repair only a third of the length of roadway surface on a bridge thereby providing a benchmark to accurately evaluate the deteriorated condition of the unaltered roadway sections. Such focused and incomplete actions are, on the face value, inherently logistically and economically inefficient, but when employed harness the social perception of complacent and unresponsive societies, the long-term economic savings can significantly outweigh the initial investment.

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Probabilistic evaluation of time to corrosion initiation in RC elements exposed to chlorides: 2-D modelling

D.V. Val & P.A. Trapper
Technion – Israel Institute of Technology, Haifa, Israel

ABSTRACT: Corrosion of reinforcing steel is one of the main causes of deterioration of reinforced concrete (RC) bridges. The deterioration may propagate relatively fast and initially manifests itself in cracking of the concrete cover that affects bridge serviceability. Usually, the time between corrosion initiation and the serviceability failure caused by cracking is shorter than the time required for the corrosion initiation. Therefore, the time to corrosion initiation represents a major parameter controlling deterioration of RC bridges exposed to chlorides.

The time to corrosion initiation depends on the ingress of chloride ions into concrete, which is a complex process involving such transport mechanisms as ionic diffusion and convection. The process is affected by a large number of factors including the properties of concrete (i.e., its composition and microstructure), the degree of concrete pore saturation, and the exposure conditions. Another important factor is chloride binding (i.e., the interaction of chloride ions with the cement past hydration products) since only free chloride ions can penetrate into concrete. Since chloride ions are charged particles their ingress into concrete will depend also on their achieved concentration and on the content of other ions presented in the concrete pore solution. A number of these factors are inter- and time- and temperature-dependent.

Models of different level of sophistication have been proposed to describe the chloride ingress into concrete. The most sophisticated models consider it as a process involving ionic diffusion and convection, which are affected by heat transfer. More simple but still quite complex models take into account diffusion, convection, and heat transfer but neglect the ionic nature of diffusion (i.e., that chloride ions are charged particles) (e.g., Saetta et al. 1993, Martin-Perez et al. 2001, Meijers et al. 2005). However, in practice, chloride ingress is still usually modelled as a pure diffusion process described by Fick's second law (e.g., DuraCrete 2000).

Modelling such a complex process as chloride ingress into concrete entails major uncertainty due to inevitable simplifications made to develop predictive models as well as due to inherent variability of concrete properties and environmental conditions. A number of studies accounting for various sources of uncertainty and considering the problem of chloride ingress and corrosion initiation in RC structures in probabilistic terms have been undertaken. In most of these studies chloride ingress was modelled as a one-dimensional (1-D) diffusion process (e.g., Engelund & Sørensen 1998, DuraCrete 2000, Vu & Stewart 2000, Kong et al. 2002).

While for such elements like RC decks or walls 1-D modelling of chloride ingress is certainly justified, for RC beams and columns this may result in overestimation of the time to corrosion initiation, especially for reinforcing bars in corners of the elements. This was demonstrated by Frier & Sørensen (2005), who evaluated the probability distribution of time to corrosion initiation for a RC bridge pier in a marine environment modelling the chloride ingress as a 2-D pure diffusion process (i.e., chloride binding and convection were not considered).

In the present paper a 2-D model for chloride ingress into concrete, which accounts for both diffusion and convection, is employed. Initially, a 1-D deterministic analysis is carried out to examine the influence of chloride binding isotherms (Langmuir and Freundlich) on the evaluation of chloride penetration into concrete. According to results of the analysis this influence is insignificant and further, in probabilistic analysis, the Langmuir isotherm is used since the use of the Freundlich

isotherm creates numerical difficulties when values of chloride concentration are very low. The analysis has been carried out for two different boundary conditions – timevarying ambient relative humidity and constant relative humidity, whose value is equal to the average value of the time-varying one. The results show that replacing time-varying humidity by its average value leads to underestimation of chlorides inside the concrete. This indicates the importance of taking into account the effect of convection on chloride ingress.

A probabilistic analysis is then performed to estimate the probability of corrosion initiation in RC wall (1-D analysis) and column (2-D analysis) with the same thickness of the concrete cover. Uncertainties in concrete properties, models describing moisture and chloride diffusion, the concrete cover thickness, and the threshold chloride concentration are taken into account. Spatial variability of a number of parameters (such as the humidity and chloride diffusion coefficients, the surface chloride concentration, the concrete cover thickness) is not considered in this study as well as possible correlation between some of them (e.g., between the humidity and chloride diffusion coefficients). According to results of the analysis the probability of corrosion initiation in the corner reinforcing bars of the RC column is much higher than in reinforcing bars in the middle part of the RC wall. This demonstrates the importance of 2-D modelling for correct prediction of corrosion initiation in such RC elements as beams and columns.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Life cycle reliability assessment based on advanced structural modeling – nonlinear FEM

R. Pukl & V. Červenka
Červenka Consulting, Prague, Czech Republic

D. Novák & B. Teplý
Faculty of Civil Engineering, Brno University of Technology, Czech Republic

A. Strauss & K. Bergmeister
Institute for Structural Engineering, Department of Civil Engineering + Natural Hazards, University of Natural Resources and Applied Life Sciences, Vienna, Austria

ABSTRACT: A complex approach to life cycle reliability assessment of concrete bridges is presented. It consists of three basic parts: nonlinear modeling of concrete, reliability calculation, and degradation aspects. The software system SARA – Structural Analysis and Reliability Assessment – integrates nonlinear finite element software ATENA with stochastic and reliability program FREET into an advanced engineering tool which can be used for reliability assessment of structures based on an advanced nonlinear computer simulation. Degradation phenomena are formalized in a special version of statistical software FREET-D. The methodology, computational methods and software tools are briefly described.

1 INTRODUCTION

The safety evaluation of existing structures is usually based on codes and different specific regulations. It has been found that reliability assessment which is going beyond the boundaries of codes (sometimes called individual approach) can bring a significant money saving and provide a new insight into administration of structures and decision-making process.

The authors combined efficient techniques of nonlinear numerical analysis of engineering structures and stochastic methods to offer an advanced tool for assessment of realistic behavior of concrete structures from reliability point of view. Within the framework of this complex system a special attention is paid to modeling of degradation phenomena, like carbonation of concrete, corrosion of reinforcement, chloride attack, etc. The combination of all three parts (structural analysis, reliability assessment and degradation modeling) is integrated in the software system SARA.

2 NONLINEAR ANALYSIS

Nonlinear finite element software ATENA is a well-established tool for realistic computer simulation of damage and failure of concrete and reinforced concrete structures. The constitutive model (constitutive relation in a material point) plays the most crucial role in the finite element analysis and decides how the structural model represents reality.

Since concrete is a complex material with strongly nonlinear response even under service load conditions, special constitutive models and numerical approaches for the finite element analysis of concrete structures are employed.

An efficient solution of engineering problems based on the advanced material models is encapsulated in user-friendly graphical environment (ATENA GUE), which supports the user during pre- and post-processing and enables real-time graphical tracing and control during the nonlinear analysis.

3 RELIABILITY ASSESSMENT

For time-intensive calculations like nonlinear fracture mechanics of concrete, the small-sample simulation techniques based on stratified sampling of Monte Carlo type represent a rational compromise between feasibility and accuracy. Therefore Latin hypercube sampling (LHS) was selected as a key fundamental technique. The method belongs to the category of stratified simulation methods. It is a special type of the Monte Carlo simulation, which uses the stratification of the theoretical probability distribution function of input random variables. It requires a relatively small number of simulations to estimate statistics of response – repetitive calculations of the structural response (tens or hundreds).

The multipurpose probabilistic software for statistical, sensitivity and reliability analysis of engineering problems FREET is based on the efficient reliability techniques described above. It contains three basic parts: stochastic modeling, sampling, and assessment. Results of the statistical simulation are: statistical characteristics of response (ultimate load, deflections, crack width, stresses etc.), information on dominating and non-dominating variables (sensitivity analysis), and estimation of reliability using reliability index and theoretical failure probability.

4 DEGRADATION MODELING

The *Performance-Based Design* (PBD) is the leading trend in structural engineering design presently; this approach deals with the durability and reliability issues, which both belong to decisive structural performance characteristics.

Different aggressive agents affect the structural material (concrete, reinforcement) in the course of time causing the cumulative degradation. The deterioration may be described by a variety of analytical models whose difference is their complexity and the number and availability of input parameters necessary. For degradation modeling of concrete structures a special version of FREET, named FREET–D, has been developed.

The input data are considered to be statistically independent random variables (optionally the deterministic values for individual variable may be chosen as well). In this way the inherent uncertainties in inputs (concerning technological and environmental aspects) were accounted for and by the statistical analysis the realistic scatter of output values can be assessed. The randomized form of all degradation effects is introduced. For all models the factor of model uncertainty Φ is provided too as an input random variable to compensate the possible inexactness or incompleteness of results.

5 APPLICATIONS

The SARA system has been successfully used during last 5 years for probabilistic nonlinear analysis of concrete structures, mainly bridges. Selected complex application is documented on life cycle (degradation and retrofitting) reliability analysis of an existing highway bridge in another consequent paper.

6 CONCLUSIONS

An integrated interdisciplinary approach consisting of non-linear fracture mechanics analysis, statistical and reliability assessment and degradation phenomena modeling is presented.

The complex software system enables to use the stochastic non-linear analysis and reliability assessment consistently in a homogeneous user-friendly environment. The proposed methodology allows evaluating of the structural safety using reliability index during and after the degradation and retrofitting processes in order to calculate the safety level for the service live conditions.

The potential application is mainly for deteriorated concrete bridges, where advanced analysis including realistic prediction of behavior in time can certainly help owners and authorities to decide how to allocate limited budget for repair and maintenance.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Lifetime reliability profiles for evaluation of corroded steel girder bridges

A.A. Czarnecki
Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, MI, USA

A.S. Nowak
Department of Civil Engineering, University of Nebraska, Lincoln, NE, USA

ABSTRACT: The load carrying capacity of steel bridges can be affected by corrosion. In addition to natural aging and increasing load spectra, corrosion is one of the main causes of deterioration. Structures exposed to aggressive environmental conditions, are subjected to time-variant changes of their load carrying capacity. Therefore, there is a need for evaluation procedures to accurately assess the actual condition and predict the remaining life of the structure. The load and resistance parameters are random variables and reliability can be used as a rational measure of structural performance. The objective of this paper is to present time-variant system reliability profiles developed for steel girder bridges.

Conventional deterministic approaches are not efficient and/or effective for assessment and evaluation of bridge condition. On the other hand, reliability methods provide a rational tool for assessment of bridge structures. They provide a good basis for the decision about repair, rehabilitation or replacement. The structural performance depends upon strength of components, loads and load sharing capabilities that are random variables. A traditional deterministic approach does not allow for consideration of interaction between the components that form a structural system. For instance, when a member reaches its ultimate capacity, it is not necessarily eliminated from the structure. It can continue to resist the load while additional loads are distributed to other members. Therefore, it has been observed that the load carrying capacity of the whole structure (system) can be much larger than what is determined by the design of individual components. The difference can be attributed to the system behavior. Quantification of this difference is the subject of the system reliability.

The present study deals with the calculation of the reliability of the whole bridge, taking into account site-specific load and resistance parameters. The objective is to formulate a limit state function for the whole bridge, identify the load and resistance parameters, develop an analysis procedure to assess the reliability of the bridge as a structural system, and develop the time-dependant reliability profiles including deterioration due to corrosion. In addition, the major steps of the procedure include the selection of representative structures and formulation of recommendations for practical bridge assessment. It was assumed that the selected bridges are representative for mid-span and short span steel structures.

The limit state functions are formulated based on the available models. Special models are considered for partially deteriorated structures and components. The basic load combination considered in this study includes dead load, live load and dynamic load. Two limit states are considered in this study: (1) moment carrying capacity of an individual girder, (2) ultimate limit state of the whole bridge that is defined as reaching the live load deflection in any of the girders equal to 0.75% of span length.

In addition to carrying the dead load, a bridge has to carry the expected live load (static and dynamic) applied during its lifetime, without violating any of the limit states. The expected live load is a statistical variable that depends on many parameters including the span length, truck weight, axle load, axle configuration, position of the truck on the bridge, number of vehicles on the bridge (multiple presence), girder spacing and stiffness of structural members. The live load effect is increased by dynamic load. Dynamic load effect is considered as an equivalent static load added to the static portion of live load. Multiple presence of trucks is considered in lane and side-by-side. The parameters that affect the analysis include headway distance and degree of correlation (with regard to weight). The frequency of occurrence is a site-specific parameter.

The analysis is performed for the ultimate limit states (moment and shear) and serviceability limit states (deflection). The major parameters that determine structural performance include load components, strength of material, and dimensions. The live load is applied in a form of the design truck. The reliability analysis is performed for different values of span length, truck position (transverse and longitudinal), number of vehicles on the bridge (multiple presence), girder spacing, and stiffness of structural members (slab and girders). For each combination of these parameters, the bridge resistance (system resistance) is determined in terms of the bending moment or corresponding gross vehicle weight of a truck (or trucks) causing an unacceptable deflection or instability of the considered bridge. The resistance is determined separately for each limit state and each transverse position of the truck (with a fixed set of axle spacings). The structural reliability (performance) is measured in terms of the reliability index, β, using one of the available analytical procedures, in particular, the first-order reliability method. The analysis is performed for components and systems.

For the structural analysis of a steel girder bridge, a three-dimensional nonlinear finite element analysis is adopted. It is assumed that the transverse reinforcement provides redistribution of truck load over the bridge width. In case of ultimate limit states as well as deflection limit states, the behavior of a bridge is defined by nonlinear element models. Consequently, failure of the bridge in the context of these limit states can be determined by the singularity of its global stiffness matrix. In practice, however, nonlinear problems are solved numerically using iterative techniques.

Corrosion reduces the effective cross-section area causing a decrease of the plastic moment capacity and leading to an increased deflection of the bridge under service loads. Under certain conditions, the deflection of corroded steel bridges can become uncomfortable for the public. Excessive deflection can cause a deterioration of the wearing surface, cracking of the concrete slab, or discomfort of users. In this study, however, these factors were not considered.

The corrosion penetration in the upper surface of the bottom flange and corresponding remaining cross section area are calculated based on assumed corrosion rate model. The FEM analysis of the bridge loaded with design trucks is carried out to determine the maximum deflection of any of the main members of a bridge. It is assumed that the limit state is reached when the maximum deflection exceeds the deflection limit.

The results of computations are presented in form of graphs relating the girder reliability index with the system reliability index. The reliability indices are calculated for various rates of corrosion, as time-varying parameters, for time periods up to 120 years.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Structural reliability of the Tampico Bridge under wind loading

D. de León
Facultad de Ingeniería, Universidad Autónoma del Estado de México, México

A.H-S. Ang
School of Engineering, University of California at Irvine, Irvine, CA, USA

L. Manjarrez
Facultad de Ingeniería, Universidad Autónoma del Estado de México, México

ABSTRACT: Bridges are important infrastructure because of its intended function to provide communication between urban centers and to allow the flow of products and goods that determine the progress of economic and industrial growth of a region. In particular, the Tampico bridge, which was built over the Pánuco river in 1988 serves as a link between the Tamaulipas and Veracruz states and joins Tampico to the highway to Mexico City.

This bridge, located on the east coast of Mexico, is analyzed to determine its structural reliability against wind loading. The inherent variabilities of the random wind force and of the mechanical properties of steel constitute the aleatory uncertainty; this contributes to the probability of failure of the steel girder. The idealization of the loading and of the bridge structure, and the analysis of the structural response to wind loading, contribute to additional uncertainty of the epistemic type, which leads to a range of possible (or distribution of) failure The effects of the uncertainties on the bridge design are discussed from the standpoint of the expected life-cycle costs and its expected failure probability. As a consequence of the epistemic uncertainty the mean damage index becomes a random variable and, also, the failure probability and the expected life-cycle cost.

For different designs, the respective expected life-cycle costs are obtained as a function of the mean failure probability. The failure probability is estimated from the FORM typical reliability format and the state limit corresponds to the compression-bending interaction ratio for the critical bridge structural member, which turns to be a main column under the gravity + wind load combination.

From a previous statistical analysis a Type II extreme value distribution was found to be the best fit to the wind velocities from past hurricanes recorded in the east cost of Mexico. This distribution is used to model the aleatory uncertainty on wind velocity. However, an epistemic uncertainty is considered for the mean wind velocity which is, in turn, considered as a random variable.

Epistemic uncertainty is also include in the costs: initial and damage or failure costs (repair or reposition costs, economic and injury and fatality costs). The economic costs are due to the service interruption in the bridge after a failure occurs. A curve of acceptable failure probability is added for several cost of failure consequences.

A double-loop Monte Carlo simulation leads to the histograms of reliability index and the expected life-cycle cost for the optimal design. The design with the minimum expected life-cycle cost is the optimal design. However, for this optimal design, the 90% value (or the mean plus one standard deviation value) of the corresponding failure probability or safety index may be selected for a risk-aversive design. The mean plus one standard deviation of the life-cycle cost may also be used for a conservative estimate of the life real whole cost of the bridge.

By doing this, the epistemic uncertainty on the prediction model for the main design parameter, the wind velocity, is incorporated and taken into account in the design selection process.

Recent changes on the meteorological and hurricane parameters and patterns, with stronger effects on urban and industrial developments on the shoreline, is becoming a concern for safety requirements and mitigation measures, specially for infrastructure located on the coast and exposed to very intense wind.

Under these conditions, a more detailed assessment of the impact of the epistemic uncertainties on the load prediction and response models, as the one formulated and applied here, is appears to be justified. Authorities in charge of coastal development planning and industry managers may use the above mentioned results for conservative decision making depending on their risk-averseness.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Statistical inference for Markov deterioration models of bridge conditions in The Netherlands

M.J. Kallen & J.M. van Noortwijk
HKV Consultants, Lelystad, and Delft University of Technology, Delft, The Netherlands

ABSTRACT: The focus of this paper is on the statistical estimation of parameters in various types of continuous-time Markov processes using bridge condition data in the Netherlands. The parameters in these processes are transition intensities between discrete condition states. These intensities may depend on the current state and on the age of the structure. The likelihood function for the observed transitions is derived and an outline of the steps to determine the parameters which maximize the likelihood function is given. The results of the fitting procedure are presented in the form of the expected bridge condition over time and the random time to the final state. For each type of Markov process, the quality of fit is compared to that of the other types. The influence of the inspector on the condition process is also discussed.

1 INTRODUCTION

The Civil Engineering Division of the Ministry of Transport, Public Works and Water Management in the Netherlands is responsible for the inspection and maintenance of highway bridges nationwide. For the purpose of maintenance and inspection planning, ongoing research is focused on determining the uncertain rate of deterioration over time. An approach using a Weibull probability distribution, fitted to observed and censored bridge lifetimes, is reported on in van Noortwijk and Klatter (2004). Unfortunately, observations of bridge lifetimes are very scarce so in this paper a different approach is presented.

Bridges in the Netherlands are subject to periodic visual inspections. At each inspection, the inspector records individual damages and uses a damage rating scheme with 6 states ranging from excellent (no damage) to very bad (damage posing extreme safety threat) to indicate their severity.

Also, the overall condition of the structure is indicated using a similar rating scheme.

To model the process with which the bridges move from the initial state 0 to the final state 5, a continuous-time Markov process is used. Four types of Markov processes are considered, which are characterized by the dependence of their transition intensities on state or age. The four types are labelled A to D and defined as shown in Table 1.

2 MATHEMATICAL MODEL

Periodic inspections result in what is referred to as current status data in the theory of statistics. The paper presents the mathematical model required for fitting the Markov processes to the bridge condition data. First, the likelihood function is derived which calculates the probability of the data given the model. This likelihood function is calculated using the transition probability function, which is discussed next. Finally the procedure for maximum likelihood estimation is outlined. Figure 1 shows the results of the parameter estimation in the form of the expected condition over time and the probability density of the random time to the final state for each model.

Table 1. The four types of continuous-time Markov processes applied in this paper.

	State	
Age	independent	dependent
independent	A	B
dependent	C	D

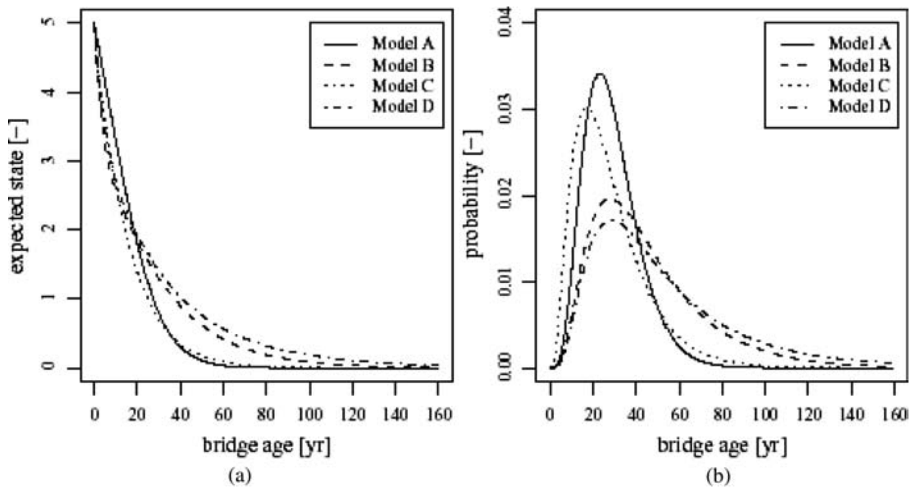

Figure 1. Expected condition (a) and probability density of the time to the final state (b) for all four models in Table 1.

3 CONCLUSIONS

There are three important conclusions to be drawn from the analysis of the bridge condition data in the Netherlands:

- the data contains incorrect and incomplete information, but there are sufficient observations for a pattern to arise, therefore the dataset can be used for the purpose of predicting the condition as a function of bridge age.
- The uncertainty in the condition process is very large. This uncertainty is not only due to the natural variability of deterioration, but also due to the measurement variability resulting from the subjectivity of condition evaluations by inspectors. The analysis has shown that the influence of the inspector on the rate of transitions is substantial.
- Out of the four model types in Table 1, type D fits best to the data. This model type has different transition intensities for each state and these intensities also depend on the age of the structure.

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Lifetime seismic reliability analysis of deteriorating bridges

H.-N. Cho & K.-M. Lee
Hanyang University, Korea

C.-J. Cha
Korea Infrastructure Safety and Technology Corporation, Kyounggi-Do, Korea

ABSTRACT: A realistic lifetime seismic-reliability based approach is unavoidable to perform Life-Cycle Cost (LCC)-effective optimum design, maintenance, and retrofitting of structures against seismic risk. However, since most developed LCC model has focused on the methodological point, they have not been quantitatively considered critical factors such as the effects of seismic retrofit, maintenance and environmental stressors on lifetime seismic reliability assessment of deteriorating structures.

Recently, a number of researchers (Shinozuka et al., 2000; Choi, 2002; Lee et al., 2005; etc.) have conducted comprehensive studies to assess the seismic reliability analysis of structures. Generally, the researches for seismic reliability analysis of a structure are performed considering the following studies: i) analytical modeling of the structure; ii) seismic hazard analysis; iii) probabilistic evaluation of the structural response and damages subjected to a given earthquake intensity; and iv) convolution of the seismic hazard with the probabilities of structural damage to obtain the life-cycle damage probabilities. Though, a structure subjected to environmental attack can experience time-variant degradations, unfortunately, these researches have not considered the effects of lifetime maintenance action as well as strength degradation due to environmental stressors on lifetime seismic reliability.

Thus, in this study, a systematic procedure for lifetime seismic reliability assessment considering the effects of seismic retrofit, maintenance strategy and environmental stressors is proposed, and then a program HYPER-DRAIN2DX-DS (latin HYPERcube sampling based DRAIN2DX for lifetime seismic reliability analysis of Deteriorating Structure) is developed to perform the desired lifetime seismic reliability assessment of deteriorating bridges. To demonstrate the applicability of the program for the study, it is applied to a simply supported continuous three span steel bridge with or without seismic retrofit strategies. The effect of resistance losses in piers due to the corrosion of steel reinforcement is considered and lifetime seismic reliability analyses under costal corrosion environment with or without specific maintenance strategies are performed to investigate the influences of maintenance strategies on bridge lifetime seismic reliability.

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Multi-objective probabilistic optimization of bridge lifetime maintenance: Novel approach

L.C. Neves
University of Minho, Guimarães, Portugal

D.M. Frangopol
University of Colorado, Boulder, Colorado, USA

P.J.S. Cruz
University of Minho, Guimarães, Portugal

ABSTRACT: The cost of repairing, retrofitting, or replacing an existing bridge or bridge network is usually very high. In many cases this cost is significantly larger than the initial construction cost. This justifies the use of very detailed analysis of each deteriorated structure, in order to evaluate the need to perform maintenance.

However, the cost of performing this analysis for each existing structure would be too high for most highway agencies or governments. In particular, for structures presenting little or no deterioration this detailed analysis would represent a waste of funds. Moreover, this type of analysis can be used with confidence to predict current performance, but looses accuracy if used to forecast future performance. In most cases, a less detailed analysis is sufficient to predict future performance of a large group of similar structures.

Such methods for a long term analysis of large groups of similar structures have been developed in most modern bridge management systems. The main aim of such a system is to provide, using relatively low cost, an accurate prediction of future structural performance.

Most existing bridge management systems (BMS) use the condition index as an indicator of the need to apply maintenance or to perform a more detailed analysis. However, this performance indicator can not be considered an indicator of the safety of the structure. In fact it disregards the effect of deterioration not visible to the inspector, the initial safety of the structure, and/or the overall behavior of the structure as a redundant or weakest-link system.

Several authors (Thoft-Christensen 1998, Frangopol 1998) proposed simplified safety analysis tools for large groups of bridges. In these methods, a relatively simple time dependent safety function is used.

These tools allow the definition of life-cycle maintenance management policies for existing structures, using a more consistent measure of the need to perform maintenance.

Furthermore, the probabilistic maintenance model presented in Frangopol et al. (2001) allows the inclusion of the effects of maintenance actions in a consistent and realistic manner. This is a significant improvement over existing BMS which, by using Markovian chains, have a limited ability to consider all effects of maintenance actions. The methods based only on the safety index are limited, however, in terms of the use of large databases collected, over the years, in most developed countries. Furthermore, the updating of the safety index, based only on routine visual inspections is difficult.

For these reasons, the authors use in this paper a model where the time dependent safety index is complemented by a time dependent condition index. In the proposed model it is considered that each maintenance action can have effects on both condition and safety. These effects can be considered as independent, dependent, or correlated.

The cost of each maintenance action is also defined as independent or dependent of the effect of the action (Neves et al. 2004, Kong and Frangopol 2004).

All parameters associated with profiles under no maintenance, effects of maintenance actions, and cost, are defined as probabilistic correlated or independent random variables. This results in time dependent probabilistic condition, safety, and cumulative cost profiles.

Two types of maintenance actions are considered: preventive action and essential actions. Preventive maintenance actions are applied at probabilistic defined intervals. Essential maintenance actions are applied when a deterministic or probabilistic condition index and/or safety index threshold is reached.

To decide if a structure must undergo extensive repair or replacement, a detailed structural assessment must be performed. This assessment should include non-destructive testing, assessment of traffic and loads, and comparison of different maintenance actions, among others.

Recent experience has shown that applying maintenance only if deterioration has endangered the safety of the structure, usually results in large life-cycle cost. Preventive maintenance can help in reducing costs significantly. However, a detailed analysis can not be performed in advance for all structures, due to large costs and uncertainties in the deterioration process.

For this reason, the definition of optimal maintenance policies must be made using the limited information, available at present. This makes the use of simplified models particularly suitable if coupled with optimization. In this paper, multi-objective optimization is used so that the decision maker can have a set of optimal solutions from which the best balance between cost and performance can be selected for a specific situation.

Results obtained emphasize the importance of preventive maintenance, in terms of reduction of overall cost. However, these results show that preventive maintenance alone is not enough to keep deteriorating structures safe and serviceable, during their entire lifetime.

The proposed probabilistic framework for multiple objective optimization of bridge lifetime maintenance allows the inclusion of the uncertainty associated with the performance of deteriorating structures in a consistent and rational manner. This is further explained in Neves et al. (2006a, 2006b).

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Damage magnitude analysis of industrial accidents by risk curve

S. Hanayasu & K. Ohdo
National Institute of Industrial Safety, Kiyose, Tokyo, Japan

1 INTRODUCTION

This study involves the probabilistic assessment of various industrial accidents in terms of the damage caused by them. The distribution of damage due to industrial accidents can be described using a Pareto distribution that appears a log-log straight curve, which is referred to as a risk curve. The parameter of the damage distribution can be estimated as a general posterior distribution by the Bayesian estimation method. By compounding the damage distribution with the estimated parameter distribution, the data-based *predictive distribution* is determined. The probabilistic assessment scheme that can analyze damage magnitudes by this *predictive distribution* using only observed accident data without estimating a distribution parameter is presented.

2 FREQUENCY–MAGNITUDE RELATIONSHIP OF INDUSTRIAL ACCIDENTS

Here, define the probability density function and its upper probability distribution function of the normalized damage due to industrial accidents by the following equations:

$$p(r) = nr^{-n-1}, \qquad R(r) = r^{-n} \tag{1}$$

where $p(r)$ is the density function of accident occurrence of a given magnitude r; $R(r)$ is the upper probability that damage magnitude exceeds r; and n is a constant parameter. From above equations, the relation between damage magnitude due to accidents r and their frequencies can be described by a simple log-log linear straight curve.

3 PROBABILISTIC ANALYSIS OF ACCIDENTS BY THE *PREDICTIVE DISTRIBUTION*

The general posterior distribution of the damage parameter under the condition that the damage data of accidents $r_i (i = 1, \cdots\cdot, N)$ is observed, can be obtained using *Bayes' theorem* as the following probability density function:

$$f(n|r_1,\cdots,r_N) = \frac{n^N}{N!}[\ln(\prod_{i=1}^{N} r_i)]^{N+1}(\prod_{i=1}^{N} r_i)^{-n} \tag{2}$$

Then the *predictive distribution* of the damage can be determined by integrating the mixture of two probability density functions of damage r and parameter n for the entire distribution field of n. The probability density function of the *predictive distribution* deducted is represented by the following equation:

$$f(r) = \int_0^\infty nr^{-n-1} \cdot f(n|r_1,\cdots,r_N)\ dn = \frac{(N+1)}{r} \cdot \frac{[\ln(\prod_{i=1}^{N} r_i)]^{N+1}}{[\ln(r \cdot \prod_{i=1}^{N} r_i)]^{N+2}} \tag{3}$$

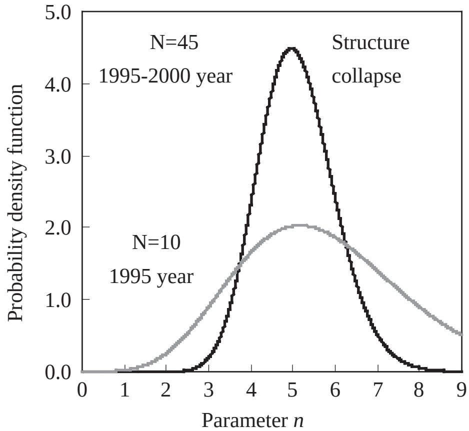

Figure 1. Posterior distribution of parameter by the Bayesian method (structure collapses).

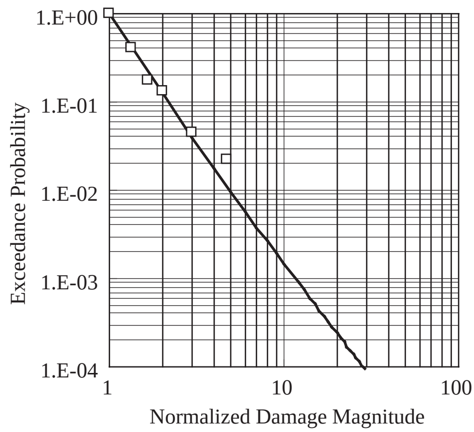

Figure 2. Predictive distribution of damage magnitude (structure collapses).

The *predictive distribution function* as determined here is described only by the observed accident data without a damage distribution parameter of *n*. The probability of a damage that exceeds a specific magnitude value r of $P(r)$ can be determined by the following equation:

$$P(R \geq r) = \int_r^{\infty} \frac{(N+1)}{r} \cdot \frac{[\ln(\prod_{i=1}^{N} r_i)]^{N+1}}{[\ln(r \cdot \prod_{i=1}^{N} r_i)]^{N+2}} dr = \left[\frac{\ln(\prod_{i=1}^{N} r_i)}{\ln(r \cdot \prod_{i=1}^{N} r_i)} \right]^{N+1} \tag{4}$$

Then the return period or MTBA whose damage magnitude exceeds a specific value of magnitude r can be calculated as below:

$$T_r = \frac{1}{P(r) \cdot (N/L)} = \left[\frac{\ln(r \cdot \prod_{i=1}^{N} r_i)}{\ln(\prod_{i=1}^{N} r_i)} \right]^{N+1} \cdot \left(\frac{L}{N} \right) \tag{5}$$

where N is the number of accidents observed; and L is the duration of the observation.

Figure 1 shows the posterior distribution of the damage parameter analyzed by the proposed *Bayesian method* for industrial accidents due to structural collapses for two different periods. The variance of the parameter determined by 6-year data (1995 to 2000, $N = 45$) becomes smaller than that of determined by one year data (1995, $N = 10$). Figure 2 illustrates the exceedance probability of the normalized damage due to structural collapses determined by the *predictive distribution* based on the 6-year observed accident data. As shown in this figure, the *predictive distribution* provides a fairly good fit to the observed accident data.

4 CONCLUSION

The data-based *predictive distribution* is determined by compounding the damage distribution with the posterior parameter distribution. Using the proposed *predictive distribution,* various probabilistic analyses of damage only by observed data without estimating distribution parameter are performed. An example of analysis for structure collapse is presented.

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Reliability-based life-cycle bridge management using structural health monitoring

Thomas B. Messervey
Department of Civil and Mechanical Engineering, United States Military Academy, West Point, NY, USA

Dan M. Frangopol
Department of Civil, Environmental and Architectural Engineering, University of Colorado, Boulder, CO, USA

Allen C. Estes
Department of Civil and Mechanical Engineering, United States Military Academy, West Point, NY, USA

ABSTRACT: Existing reliability-based life-cycle bridge management models (LCM) seek to take advantage of the best information available for the input parameters of random variables. Experimental studies, theoretical models, visual inspections, and non-destructive evaluation are commonly used for this purpose. The growing field of structural health monitoring (SHM) offers a potentially powerful means to obtain data specific to a particular structure. In this paper, a

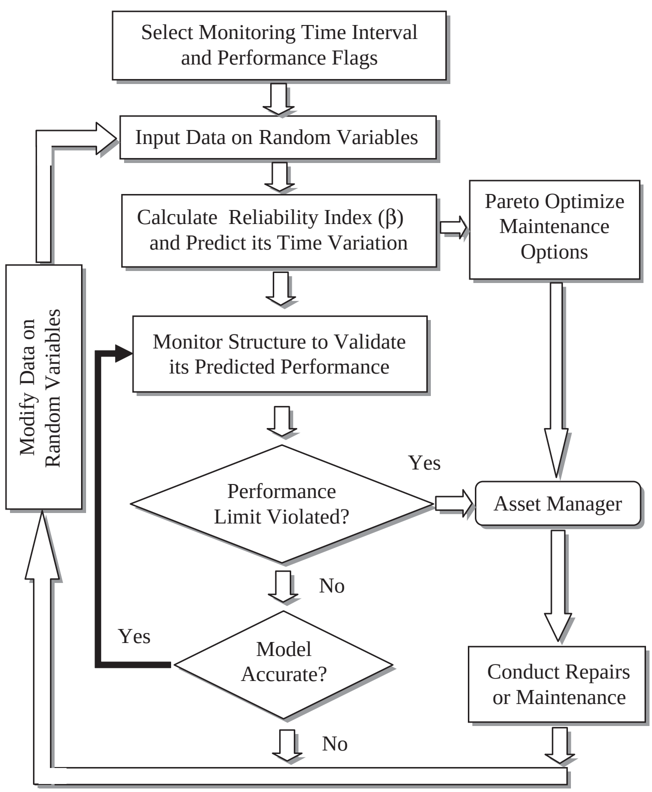

Figure 1. Framework to incorporate SHM data into a LCM model for bridges.

framework for incorporating data obtained via SHM to update random variable input parameters into LCM bridge models is presented. The idea is analogous to the common practice of using modal analysis to validate or improve a finite element model. Two flowcharts detailing key ideas and steps are provided with accompanying graphs depicting how the process could improve the prediction of the reliability over time. One of these flowcharts is shown in Figure 1.

A numerical example of a simply supported bridge beam subjected to the HS-20 truck load and corrosive effects is developed. The example bases itself upon two performance functions for elastic flexure and compares the reliability index over time for both models. Monitoring occurs through means of a strain gauge.

The results of the example show that the monitoring-based model has a higher initial reliability index and a slower rate of deterioration over time. It is also noted that in this specific example the two performance functions can be used to support each other. The first performance function is associated with high uncertainty, but the limit state is based on a theoretical elastic model that allows prediction of future behavior. The second performance function has less uncertainty which provides a greater reliability, but the causes of the increased stress are unknown. The increased stress could be caused by section loss due to corrosion, increased traffic load, fatigue cracking, or even an inoperable expansion joint or frozen bearing. All that is known is that the stress is increasing over time and prediction of future behavior is more difficult. If the strain readings can be combined with some non-destructive inspection of section loss, the strains can be used to verify the theoretical model and can confirm that girder corrosion is indeed the primary cause of increased stress. Similarly, the results may diverge and the bridge manager at least knows to investigate some other cause of the distress.

The example shows that a monitoring-based approach is of potential merit. It is concluded that combining the advantages of structural health monitoring methods and life-cycle management models may provide significant synergistic benefit to both approaches and to the bridge manager.

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Seismic design and retrofitting strategies for bridges

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Application of displacement-based seismic analysis of bridges: Case study of the Taiwan Chi-Chi earthquake

C.C. Fu
University of Maryland at College Park, MD, USA

Hamed Alayed
The Applied Science University Amman, Jordan

Yao T. Hsu
Feng Chia University Taichung, Taiwan, R.O.C.

ABSTRACT: Recently there is a trend and an increasing interest in simplified nonlinear analysis or what is referred to as the pushover analysis that can give acceptable results with less effort compared to the nonlinear time-history analysis. These methods were developed basically to perform seismic evaluation and retrofitting of buildings only. This study may contribute in developing a simplified nonlinear static procedure (NSP) for bridges, which will be an important step to apply performance-based design for bridges.

Nonlinear Static (Pushover) Procedure (NSP) is specified by FEMA-273 (1997) as an analytical procedure that can be used in systematic rehabilitation of structures. However, guidelines are presented to apply the Displacement Coefficient Method (DCM), which implements the well-known equal displacement rule with some modifications to estimate target (demand) displacement, only for buildings. This study is intended to evaluate the applicability of NSP by implementing the DCM to bridges. For comparison purposes, the Nonlinear Dynamic Procedure (NDP) (or nonlinear time-history analysis), which is considered to be the most accurate and reliable method of nonlinear seismic analysis, is also performed.

A case study is associated with Hsin Shi-Nan bridge which was suffered some damage in the transverse direction during the Taiwan Chi-Chi earthquake in 1999. The bridge is analyzed using the seismic record obtained from local stations. Analysis results proved that the rotation of plastic hinges for the damaged column is beyond the limit, which agrees with the observed damage. This paper will also report the bridge damage in Taiwan Chi-Chi earthquake. Damage to bridge structures may occur in the superstructure, the substructure or the approaches. Typical types of damages, bridge strengthening and retrofitting strategies are discussed and illustrated in this paper.

Applicability of the NSP to bridges is investigated in this study using the DCM, which was presented by FEMA-273 (1997). A three-span bridge was presented and described as a case study. Comparison of results obtained from the NDP (NTHA), which is considered the most reliable method for nonlinear analysis, with the results of the NSP by implementing the DCM was performed to evaluate the validity of the later procedure.

As a result of the work that was completed in this study, the following conclusions were made:

1. Conservative results are obtained from the DCM in the longitudinal direction of the bridge and those results become to be over-conservative as the structure is driven farther into the inelastic range.
2. Reasonable results are obtained from the DCM in the transverse direction and those results become more conservative as the structure is driven further into the inelastic range.
3. DCM is applicable for bridges, in general. However, it inherits the same shortcoming associated with this method when it is implemented for buildings, which is the overestimation of target displacement.

*Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Advancements in seismic vulnerability assessment and retrofitting strategies

A.H. Malik & M. Asce
New York State Department of Transportation

ABSTRACT: Coordination of the research work among various seismically active countries plays an important role in developing seismic damage reduction methodologies. International conferences like this one and joint workshops among countries contribute towards better understanding of the seismic demands and means to minimize the damage .The paper describes the trend towards comprehensive procedures, and new technologies which are instrumental to evaluate the performance of the structure under seismic loading.

Performance based assessment advocating multi-level hazard, advance protective devices, rigorous analysis tools and large scale simulation testing facilities are helpful in saving lives, minimizing disruption to transportation systems and cutting down the gravity of threat and seismic vulnerabilities to acceptable levels.

1 BACKGROUND

There has been significant number of earthquakes causing major damage to all kind of structures throughout the world. As the 20th century came to close, substantial progress has been made in seismic research and seismic design provisions.

2 CURRENT TRENDS

2.1 *Comprehensive seismic research*

In 1990's, under the United States Federal Highway Administration (FHWA) sponsorship, a comprehensive seismic research program was started by Multidisciplinary Center for Earthquake Engineering Research (MCEER), involving nationwide experts from universities, and practicing engineers covering various tasks of the six-year program (MCEER 1999).

2.2 *Multi-level hazard and performance based design*

The recent earthquakes have shown that single level approach carries a high risk against infrequent events, resulting in severe consequences. The multi-level approach considers separate assigned performance criteria for each hazard level, depending on the operational and/or life safety objectives of the bridge.

In 1998, New York City Department of Transportation (NYCDOT), New York State Department of Transportation (NYSDOT), and FHWA agreed to implement the performance based multi-level hazard criteria for the New York City and the surrounding area bridges (NYCDOT 1998, MALIK 2001).

2.3 *Technical evaluations*

In United States, the Civil Engineering Research Foundation (CERF), through a cooperative agreement with the FHWA, created the Highway Innovative Technology Evaluation Center (HITEC) to expedite the introduction of innovative products into the U.S. highway and bridge markets.

3 ANALYSIS METHODS

Time history method and Pushover analysis are being utilized to encompass the variable mode of failures. Inelastic Time history analyses are more rigorous and are applied to long span structures considering multiple support excitations.

4 JOINT VENTURE

The U.S. National Science Foundation (NSF) and the Japanese counterpart signed an agreement on September 11, 2005 to enhance both countries' efforts to work together to combat the devastation of natural disasters (NSF-NEES 2003, NIED 2005).

5 CONCLUSIONS

The extent of damage due to large earthquakes can be minimized through research and engineering, as well as improved implementation of quality control construction practices and high performance materials.

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Seismic vulnerability assessment of bridges in Germany

P. Renault & K. Meskouris
Chair of Structural Statics and Dynamics, RWTH Aachen University, Germany

1 INTRODUCTION

One of the main causes for a high number of deaths during earthquakes is that rescue measures can often only be accomplished insufficiently and delayed. Strategic important roads are impassable due to collapsed bridges, debris or fires so that rescue forces are not able to attain incident places. Hence there is a need to assess the seismic vulnerability of existing bridges in order to guarantee their functionality in case of an earthquake. The assessment procedures should consider the demands of the current standards and must be applicable in the engineering practice. Furthermore the forthcoming introduction of the Eurocode 8 – Part 2 (CEN 2004) increased the interest for reliable assessment procedures considerably.

In the present paper a general seismic vulnerability assessment system for bridges, developed within the framework of the ongoing research project "Seismic vulnerability assessment of bridges in Germany" funded by the German Federal Highway Research Institute, is introduced. The basis of the system is a hierarchical classification of the bridges into different types. For each bridge type a vulnerability assessment procedure consisting of three different levels with an increasing expenditure of time is provided for the user of the system. The developed system is linked to a national database (Bast 2005), which provides information of all strategic important bridges in Germany. The level of assessment can be chosen depending on the scope and the accuracy required.

2 DESCRIPTION OF THE PROCEDURE

On level I a fast and relatively inexpensive assessment based on information taken from construction plans, site characteristics and bridge inspection results is performed. Furthermore the bridge importance for traffic and rescue measures is taken into account. The evaluation results of the recorded information are used to detect the structural deficiencies and to determine the fragility curves, which quantify the likelihood of the occurrence of certain damage states. On level II a linear response spectrum analysis based on a two dimensional finite element model is carried out to determine the damage indicators. The generation of the numerical model is executed by the system on the basis of the geometry and material input parameters. The vulnerability function of the bridge is obtained by running the level II calculation for different return periods. On level III a significantly more precise time history analysis with a detailed three dimensional finite element model is carried out to determine the damage indicators. The model includes geometric and physical nonlinearities as well as soil-structure interaction effects. The vulnerability function is calculated by using artificially generated time-history accelerograms for different return periods. Measurements of the eigenfrequencies are carried out to calibrate the numerical model. A level III investigation is only required when the results in the first two levels identify a critical bridge behavior under seismic loading.

3 PRACTICAL APPLICATION

The practical use of the developed system is demonstrated for important bridges located in western Germany. The results of the level I assessment with respect to the specific characteristics of the bridge type are discussed. The generation of the two and three dimensional finite element models for the investigations in level II and III are presented. Both models were calibrated by the results of in-situ measurements of the bridge. The vulnerability of the bridges is discussed on the basis of the results in the different investigation levels. The deficiencies of the bridges are shown and general conclusions for the seismic vulnerability of bridges are derived for the engineering practice.

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Full-scale pseudo dynamic test for bridge retrofitted with base isolations

K.B. Han
University of Nebraska, Lincoln, Nebraska, USA

S.-K. Park & S.N. Hong
Sungkyunkwan University, Suwon, Gyeonggi-Do, Republic of Korea

H.-Y. Kim
Korea Institute of Construction Technology, Goyang, Gyeonggi-Do, Republic of Korea

ABSTRACT: This paper presents the performance of bridge retrofitted with seismic isolation. For the experiment, part of the real bridge was made to its full scale model and pseudo-dynamic test method was selected for the testing this model. Pseudo-dynamic test is an appropriate method which meets the aim of this experiment because it can only produce the part of the target structure into a real model and the remaining parts only exists as an internal model. Also by inputting the earthquake load, the dynamic qualities can be identified.

El Centro earthquake (NS, 1940) was the seismic acceleration used for the pseudo-dynamic test of a pier. We set 15 seconds for earthquake application and used PGA (peak ground acceleration) of earthquake acceleration value with the increasing amounts of 1.154 g, 0.22 g, 0.34 g, and 0.7 g. With the limited number of specimens, we had to utilize a specified earthquake history data. However, in order to set up a number of meaningful levels of the earthquake load, we allowed a multiple levels of earthquake load using a controlled method of PGA. Then, we placed each individual specimen to undergo the earthquake load with an increasing earthquake acceleration level for 60 seconds.

In this paper, a total of four different specimens were fabricated as Specimen PILOT without the bearing, Specimen POT with the existing bridge bearing, Specimen RB with the RB base isolation, and Specimen LRB with the LRB base isolation. The section view of the fixed support of the example bridge is reproduced in full-scale size. However, its foundation and copping should be flexible according to the experimental methods and locations. That is, the foundation should be designed in a way to have the stiffness higher than that of the pier so that it can be fixed completely, and the height of the foundation should be adjusted to each specimen to even the loading height, as the surfaces of seismic isolation equipments differ from one another. The section view of the copping is determined by the size and the formation of the selected seismic isolation equipments. The longitudinal reinforcements and the struts arranged in the piers of the specimen follows the actual arrangement as indicated in the drawings of the selected bridge. The clear height of the pier is 363 cm, diameter is 80 cm and the aspect ratio exceeds 4.5. Thus, we can expect a behavior of flexural fracture and the plastic hinge is most likely to develop in the lower part of the specimen. As a result the steel strain gauges were mounted to the lower section of the specimen. The production, installation process, and detail drawings of the specimens are shown in Figure 3.

In the case of Specimens PILOT and POT, the measured load-displacement response represents the upper part of the whole bridge because the displacements condition of the bearing acts as a fix-end for the superior part of the bridge. However, in the case of Specimens RB and LRB, a related displacement exists between the bridge and the base isolation, thus the load-displacement history diagram was represented by distinguishing the total displacement of the bridge, the displacement of the top pier and the displacement of the base isolation by related displacement.

As the result, in the case of bridges without retrofit process (Specimens PILOT and POT), the bridge ranges from 0.154 g PGA level to a yielding point and does not satisfy the functional level. When comparing it with the retrofit bridge with RB and LRB (Specimens RB and LRB) it reached up to 0.154 g PGA and did not reach the yielding point, thus satisfying its functional level.

In the higher earthquake load level of 0.22 g PGA, non-seismic retrofit bridges starts to radically form a plastic hinge after its yield progressing from lower part of the pier with horizontal and vertical subsurface cracks. However, seismic retrofit bridges did not reach to its yielding point of longitudinal reinforcement and only produced horizontal cracks caused by tension fracture of the cover concrete.

Under the 0.34 g PGA, Specimen RB showed a yield of longitudinal reinforcement and further progress of horizontal cracks, whereas, the non-seismic retrofit bridges increased the crack width and reached to compression crushing point. Specimen LRB has maintained a functional level of performance equal to non-yielding state.

In the level of 0.7 g PGA, horizontal resistance capacity of the non-seismic bridge was totally lost due to the flexural fracture. Specimen RB has shown higher seismic performance than the non-seismic retrofit bridge by incorporating the vertical cracks of pier and reaching to its initial stage where concrete cover starts to cause a compressive fracture. Specimen LRB showed yield of longitudinal reinforcement. However, the seismic retrofitted bridge using LRB has shown higher seismic performance because the state of failure mode was lower level than any other specimens.

The results showed that a base isolation system was very effective in reducing the magnitude of the forces transferred to the substructure and in shifting the period of the bridge. LRB system can effectively reduce the peak acceleration transmitted to the structure more than those with RB system under earthquake. Using the test results, proposed seismic retrofit method was found to be valid.

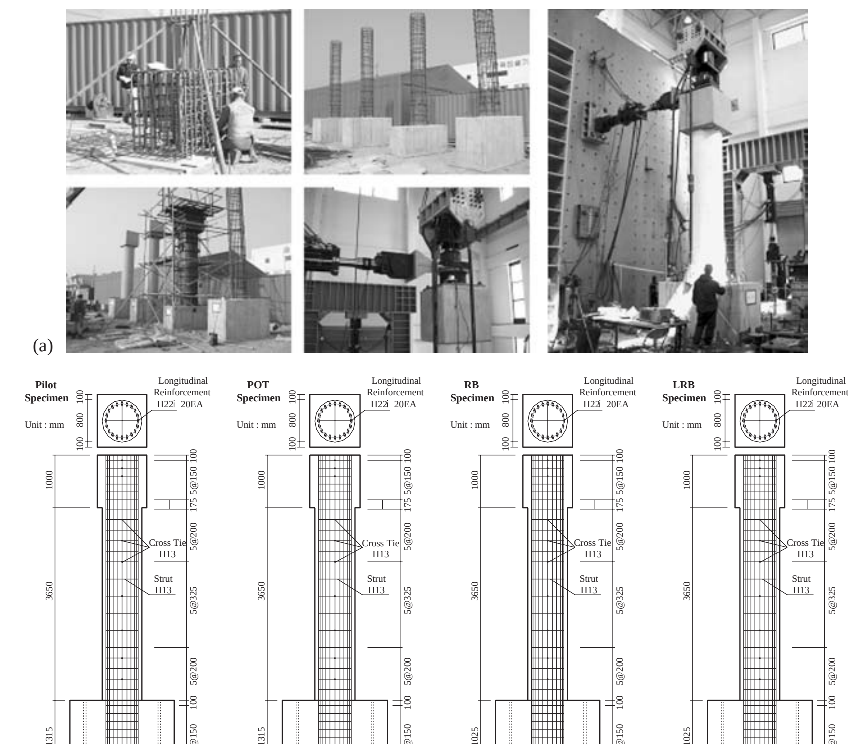

Figure 3. Production of specimens: (a) production and installation of specimens, (b) detail drawings of specimens – PILOT specimen, POT specimen, RB specimen, LRB specimen.

*Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)*
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ANN-based damage detection using dynamic responses of seismically isolated bridge structure

J.F. Choo, H.-M. Koh & C. Chung
Seoul National University, Seoul, Korea, Korea Bridge Design & Engineering Research Center, Seoul, Korea

S.K. Kook
Pukyong National University, Busan, Korea

J.H. Lee
Dongguk University, Seoul, Korea

ABSTRACT: Civil structures degrade since their construction, which leads to inherent uncertainties and nonlinearities that cannot be modeled accurately. Detecting anomalies in the structural properties that may occur in such structures appears thus as a daunting task using conventional analysis tools. Artificial neural networks (ANN) offer a fair alternative to deal with such uncertainties and nonlinearities. Therefore, an ANN-based approach using measured dynamic responses is proposed to model the structure and, detect, locate and evaluate eventual damages that may occur in the structure. The technique relies on the direct use of vibration data measured on the undamaged system to train a network predicting the restoring force of each member, which models the structure and provides a reference for damage detection purpose. The damage parameter is assumed to be the time histories of the restoring force and corresponding stiffness. The network, fed with data of the structure subject to different loading and stiffness degradation cases, reconstructs then the actual restoring force loop and extracts the corresponding stiffness. Comparison of the reconstructed loops with the original ones allows damages to be detected as well as their location and extent to be evaluated. Application of the proposed ANN scheme on a representative seismically isolated bridge structure is shown to provide efficient and accurate diagnosis of its structural changes. The proposed technique is believed to support further development in structural health monitoring of civil structures.

1 PROPOSED ANN-BASED DAMAGE DETECTION APPROACH

The dynamic characteristics of a structural system can be used to confirm the quality of construction regard to the original design, improve the analytical model of the system or monitor eventual anomalies that may occur in the structure. The proposed algorithm begins by constructing a model of the healthy structure and a model representing the actual and eventually damaged structure. All the networks implemented in this study are feedforward, multilayered networks using backpropagation algorithm. After the modeling process, the stiffness of each of the constitutive structural elements is selected to identify the state of the structure. Changes in the stiffness indicate thus damage in the structure. The proposed algorithm processes by neuro-computing the acceleration, velocity and displacement responses of the system step-by-step through integration-like operations. Once the acceleration, velocity and displacement responses of the system are obtained through neural operations, these values are fed to another network which computes the stiffness as well as the restoring force of each of the structural member. Through this process, the restoring force loops and stiffness time histories of the members can be obtained using very simplified networks. Thereafter, the restoring force loops obtained for both reference and actual states of the structure at hand are compared so as to identify the presence, severity and location of damages.

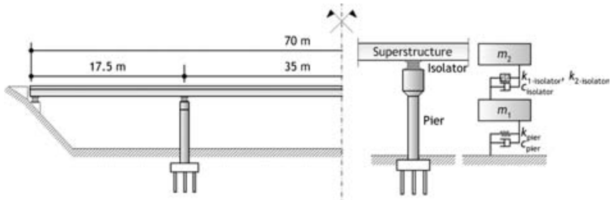


Figure 1. Longitudinal view and structural model of the selected seismically isolated bridge.

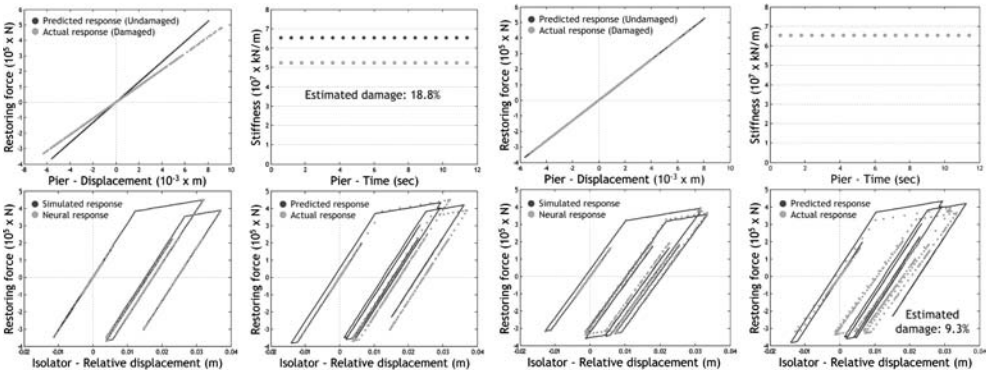


Figure 2. Comparison of restoring force and stiffness for damage case 2 (left) and damage case 7 (right).

2 APPLICATION OF THE PROPOSED DAMAGE DETECTION ALGORITHM

The structure selected for this application is the Seo-Busan IC Bridge shown in Fig. 1. It is a three-span continuous steel plate girder bridge with a total length 70 m ($17.5 + 32 + 17.5$ m) and a width of 15.2 m. The rahmen type piers are constituted by alignment of 6 piers of 6.6 m height. In the present application, the seismic isolators are lead plug rubber bearings (LRB) that have been designed according to lifecycle-cost concept for low to moderate seismic regions. Damage detection has been performed for several damage episodes, of which representative results are depicted in Fig. 2. The obtained results show that the proposed approach detects and measures accurately damages even in structures presenting nonlinear behavior. This shows that the capability of the proposed process to reconstruct the hysteresis loop based on measured data.

3 CONCLUSIONS

A neural network-based algorithm has been proposed to identify the location and extent of damage for civil structures. The algorithm processes by computing successively the acceleration, velocity and displacement responses from measured ground accelerations, and the velocity and displacement responses from measured accelerations. Using the computed responses, the actual and reference restoring forces of each structural member are also computed via neural networks to produce the actual and reference stiffness of the corresponding members.

The applicability and efficiency of the proposed damage detection algorithm have been verified through application on a nonlinear seismically isolated bridge. The applications demonstrated that the proposed approach may be readily exploited for structural health monitoring as it makes direct use of measurement database of the applied loading. The use of ANN makes it possible to identify damage in structure without assumptions regarding the complexity of linear as well as nonlinear interactions that may exist between the different structural members. Early damage detection can be performed as the network is sensitive for small changes in the measured responses, which makes the proposed algorithm very attractive for structural health monitoring.

Bridge testing and assessment

*Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)*

Technical evaluation of the bridge crossing Olt River in Râmnicu Vâlcea – Romania

N.V. Popa, A.I. Dima & I.R.I. Răcănel
Technical University of Civil Engineering, Department of Bridges, Bucharest, Romania

ABSTRACT: The construction of the new Romanian railway line VÂLCELE – RÂMNICU VÂLCEA (Fig. 1) have been approved by the State Council order No. 544, on December 1978. Economical and strategical importance of this railway was shown especially during the first world war due to the possibility to reduce with more than 100 km the transport distance between the main harbor to the Black Sea, Constanta and capital city Bucharest with the west border of Romania, compared with the solutions offered by the existing railway network built in the 2nd half of the 19th century. As a consequence from 1920 this railway was found in all development programs of the Romanian railway network. Technical and especially cost reasons determined the delay of this project. Only after the second world war it were carried out complex geological and topographical studies which allowed the choosing of the approved alignment, with a length of 38.0 km, which was considered the optimum solution for this railway in service.

The works to the railway line started in 1981 and about 85% was finished till 1989. After 1990 due to the changes well known, in Romania, this project was stopped. Now, after 15 years, due to also of SECOL COMPANY from ITALY (SOCIETA EDILE COSTRUZIONI E LAVORI S.p.A), involved from 1994 in this project, there is the possibility to completion works for the railway line Vâlcele – Râmnicu Vâlcea and open it to traffic.

In the first stage it was necessary to evaluate the technical state of all works already carried out till 1990. On this railway line there are 8 bridges and underpasses, 17 viaducts, 107 culverts and 2 tunnels. The bridge crossing Olt river (Fig. 2), which is the subject of this paper, has a length of 277.60 m, 5 spans (40.0 m + 3 × 64.20 m + 40.0 m) and the largest span on this railway line (64.20 m).

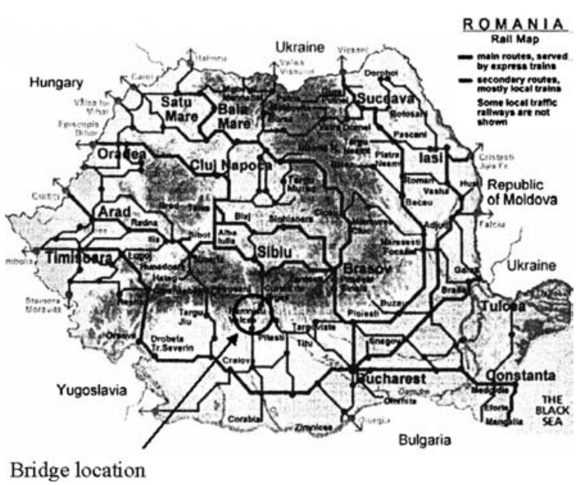


Figure 1. Romania rail map with the location of Vâlcele – Râmnicu Vâlcea railway and the bridge crossing Olt river.

Figure 2. General view of the bridge.

A brief description of the bridge conception, construction and inspection results are reported. As the bridge construction work was finished 15 years ago we present the results of recently technical state evaluation which was necessary both of the lack in maintenance during the time pass from construction and the new requirements for fatigue safety of steel railway bridge structure which are adopted in 1998 (SR 1911-98). The results of fatigue assessment of superstructure steel components specifications using both the Romanian standard and Eurocodes are discussed.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Multi mapping in evaluation of concrete bridges

A. Wawrusiewicz
Bialystok Technical University, Bialystok, Poland

ABSTRACT: The conception of the unified presentation of reinforced concrete bridges tests results realized implementing varying diagnostic methods is presented in the paper. The Multi Mapping procedure comprises maps charting for the investigated surfaces incorporating corrosive state of reinforcement, concrete strength, the range of carbonation, the thickness of concrete cover, the contents of chlorides, the permeability of concrete cover etc. Maps created on the same base are more precisely outlining existing deterioration factors. The overlapping of these maps will enable to specify the areas requiring special diagnostic control, or estimation of complex hazards of structure durability.

1 MULTI MAPPING WITH USING VARIED INVESTIGATION METHODS

1.1 *Estimation of hazards caused by individual deterioration factors*

For comparing the results, different hazards are to be analysed utilizing the same units. The hazard of destruction or arising of deterioration factor in critical value will be expressed in percentage *d* of its probability.

For example, hazard d_E resulting from reinforcement corrosion can be taken in dependence to corrosive potential *E* measured relating to the electrode $Cu/CuSO_4$:

$$\begin{aligned} \mathrm{E} > -200\ \mathrm{mV} &\Rightarrow d_E = 0\% \\ -350\ \mathrm{mV} \le \mathrm{E} \le -200\ \mathrm{mV} &\Rightarrow d_E = \frac{E+200}{-150}\cdot 100\% \\ \mathrm{E} < -350\ \mathrm{mV} &\Rightarrow d_E = 100\% \end{aligned} \tag{1}$$

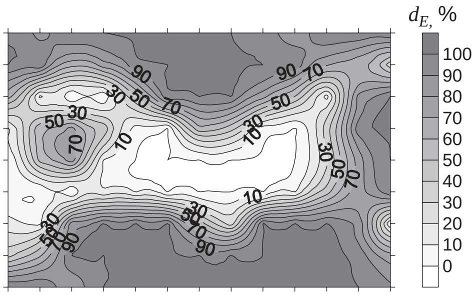
Hazard d_R in result of large velocity of corrosive processes in reinforcement can be characterized on the basis of measured resistance *R* of concrete cover:

$$\begin{aligned} \mathrm{R} > 20\ \mathrm{k\Omega cm} &\Rightarrow d_R = 0\% \\ 5\ \mathrm{k\Omega cm} \le \mathrm{R} \le 20\ \mathrm{k\Omega cm} &\Rightarrow d_R = \frac{20-R}{15}\cdot 100\% \\ \mathrm{R} < 5\ \mathrm{k\Omega cm} &\Rightarrow d_R = 100\% \end{aligned} \tag{2}$$

The graphically result of implementation of this equations is shown in Figures 1–2.

1.2 *Estimation of hazards defined by several parameters*

As an example of hazard defined by several common parameters is corrosion hazard d_c caused by carbonation of reinforcement cover (Fig. 3). Hazard occurs when layer of carbonized concrete



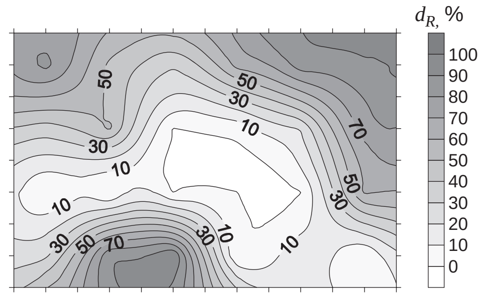
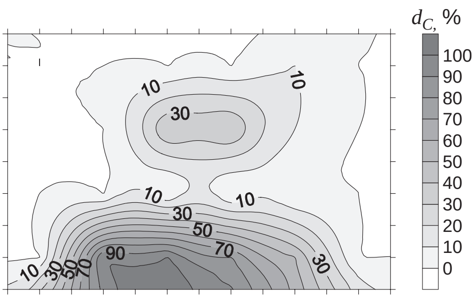


Figure 1. Hazard d_E resulting from reinforcement corrosion.

Figure 2. Hazard d_R in result of large velocity of corrosive processes in reinforcement.



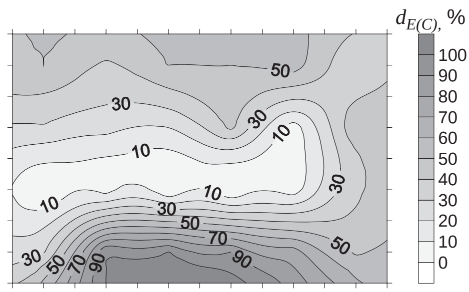


Figure 3. Hazard d_C of carbonation of reinforcement cover.

Figure 4. Hazard $d_{E(C)}$ of reinforcement corrosion in result of carbonation.

cover reach reinforcing bars. Hazard d_c depending on thickness of reinforcement cover *a* and the depth of carbonization *c* given in [cm] can be estimated as follows:

$$\begin{aligned} c < a - 1\text{cm} &\Rightarrow d_C = 0\% \\ a - 1\text{cm} \le c \le a &\Rightarrow d_C = (1 - a + c)\cdot 100\% \\ c > a &\Rightarrow d_C = 100\% \end{aligned} \tag{3}$$

1.3 *Identification of hazard sources*

The hazard sources can be identified due to comparing maps of existing hazards with the maps of deterioration factors potentially generating these destructions. Except visual comparison of the maps of effects and probable causes, the quantitative maps also can be generated.

For example, the hazard $d_{E(C)}$of reinforcement corrosion in result of carbonation (Fig. 4) is evaluated from formula:

$$d_{E(C)} = \frac{d_E + d_C}{2} \tag{4}$$

2 CONCLUSIONS

Graphic analysis of hazard maps is readable for every language, bringing people together.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Damage detection using reflectorless electronic distance measurements: Results of the first epochs

K. Zilch, E. Penka & M. Hennecke
Zilch + Müller Ingenieure GmbH, Munich, Germany

U. Willberg
Autobahndirektion Südbayern, Munich, Germany

Th. Wunderlich
Chair of Geodesy, Technische Universität München, Munich, Germany

1 INTRODUCTION

The assessment of existing bridges is becoming more important in the Federal Republic of Germany, as most German infrastructure was built between 1950 and 1970. However, the actual condition or damage of the structures could rarely be detected using standard methods. The following paper outlines an economical method of damage detection for a problem occurring on a prestressed concrete bridge built in the sixties. This method uses vertical displacements caused by damage to the underside of the bridge. These displacements are determined using reflectorless electronic distance measurements along with numerical FE simulations.

2 DAMAGE TYPE AND DETECTION

The bridge concerned has a length of about 600 m and a width of up to 40 m. The main bridge consists of 20 spans. In most areas, the box girder bridge has a cross-section of 10 cells. The bottom slab was built first and afterwards the webs together with the upper slab were added. During use of the bridge, water containing chlorides penetrated the construction joints between the webs and bottom slab through leaks in the drainage. Because of holes in the webs the water was able to infiltrate large areas. This led to strong corrosion in the relevant areas. Consequently, in one area of the bridge the shear reinforcement failed and the bottom slab was separated from the web.

A FE Simulation showed that adequate bearing capacity is still available due to a redistribution of forces within the bridge. But the damage must be repaired so as to avoid further structural degradation and respectively cause limitations in traffic usage. In addition, it can be seen in the simulation that deformations up to 8 mm occur on the bottom of the bridge due to losses of web stiffness. With these deformations, the crack width in the construction joint is less than 1 mm. This crack width could not be easily observed.

These FE simulated deformations should be used for the assessment of the current bridge to enable continuous usage during the planning and construction of a new bridge.

3 REFLECTORLESS ELECTRONIC DISTANCE MEASUREMENT

A concept for the measurement of the vertical displacements has been developed together with the Chair of Geodesy. It enables the determination of vertical displacements with a standard deviation less than 1 mm. The method is based on prismless electronic distance measurement and motorized tacheometry. The system scans the underside of the bridge with a grid of 2 m × 2 m containing

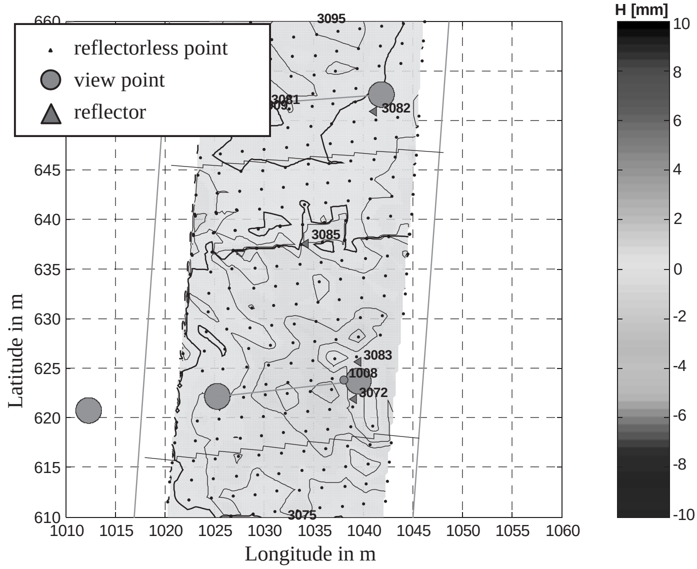


Figure 1. Altitude Changes (span 8).

2800 reflectorless measurement points. In addition, 4 reflectors per bridge span were installed to detect horizontal deformations caused by temperature effects.

4 RESULTS OF THE MEASUREMENTS

At the moment two measurement epochs are evaluated. The horizontal movements of the reflectors are taken into account with the determination of the displacements of the reflectorless measured points. In one of the first analyses, significant buckling formations arose on examination of the displacement differences. It could be shown with an inspection of both measuring devices used, that these displacement figures lead back to one systematic error. It is a matter of different zero point errors of the two devices, which can be remedied through a corrective calculation. In Figure 1 the corrected altitude changes between the first and the second epoch are shown. The differences in vertical displacements are up to 1 mm. But the deformation type which is expected from the damage could not be seen.

This resulted in that with the errors in the first analysis the existing buckle was only a matter of an apparent displacement. However, the error does show that vertical displacement differences of 3 mm are clearly recognizable in the analysis, even though the actual difference in length of 3 mm resulted from the internal discrepancy in the way the devices collect measurements.

5 CONCLUSIONS

If the separation of the bottom slab from the web is detected in time, this type of damage will not become a problem for the bearing capacity of the bridge as the bridge can then be rehabilitated without limiting traffic. The reflectorless electronic distance measurement together with numerical models of the bridge offers the opportunity to detect this damage type in wide areas of the bridge with less time expenditure. The results of the first epochs show that the accuracy of the measurement system is adequate for the detection of the damage because a displacement difference up to 3 mm can clearly be seen in the evaluations. This value is smaller than the expected values of the damage simulation. So with the results achieved up to now, this method enables the use of the bridge until its replacement, whereas no limitations on traffic are currently necessary.

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Statistical damage detection of structures by using system identification with 1-norm based regularization

Hyun Woo Park
Korea Bridge Design & Engineering Research Center, Seoul National University, Seoul, Korea

Hoon Sohn
The Department of Civil and Environmental Engineering, Carnegie Mellon University, PA, USA

Hae Sung Lee
School of Civil, Urban & Geosystem Engineering, Seoul National University, Seoul, Korea

ABSTRACT: Various damage assessment algorithms based on system identification (SI) schemes have been proposed during the last few decades. In these algorithms, the stiffness parameters of structures are estimated by minimizing the least squared errors between measured and calculated responses of structures, and damage status of structures are assessed based on the estimated stiffness parameters. Therefore, a stable SI scheme is essential for reliable damage detection. Unfortunately, however, SI is a type of inverse problem that suffers from ill-posedness. The ill-posedness of inverse problems is characterized by the discontinuity and non-uniqueness of solutions, which are caused by noises in measurements and the sparseness of measured data, respectively.

Recently, the Tikhonov regularization scheme for SI was proposed to overcome these in-stabilities, and applied to continuous and framed structures. In the Tikhonov regularization scheme, the original error function is modified by adding a regularization function, and the optimization is performed for the modified error function. The regularization function defines the solution spaces of SI problems, which resolves the in-stabilities. The majority of previously published works on the regularization schemes utilize the 2-norm of stiffness parameters such as Young's modulus and cross-sectional rigidities.

This paper presents an 1-norm regularization function for the SI of framed structures. The 1-norm regularization function for a framed structure is directly defined by the 1-norm of the stiffness parameters because they are given independently for each member in a discrete fashion. The truncated singular value decomposition (TSVD) is adopted to minimize the error function with the 1-norm regularization function. A truncation number in the TSVD plays a crucial role in SI since the truncation number controls the stability and accuracy of the solution. The bilinear fitting method (BFM) is adopted in this paper to determine an optimal truncation number.

The majority of the published works on statistical damage classification was in the context of normal distribution. Although data are frequently assumed to have a normal distribution for building a statistical model for damage classification, it has been shown that data often do not conform to a normal distribution. In this study, a statistical approach based on extreme value statistics (EVS) is adopted to determine the decision boundary of each structural member for damage classification. System parameters identified from damage-free structure are used to build a baseline distribution through parameter estimation of the generalized extreme value distribution (GEV). Then, the decision boundary for damage classification is calculated based on the baseline distribution.

Numerical simulation studies are performed using the proposed method to determine the damage status of the two-span continuous truss. Figure 1 shows the geometry, support conditions and the locations of 12 observation points, which are depicted as solid circles in the figure. Horizontal displacements are measured at the roller supports and vertical displacements are measured at the other observation points independently for each load case shown in Figure 1.

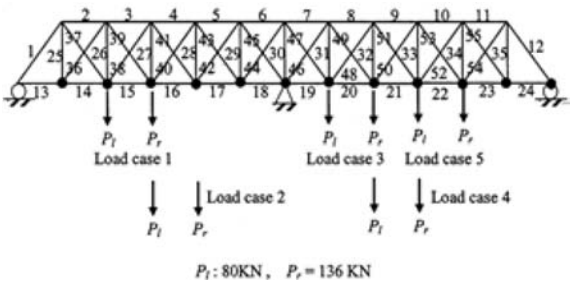

Figure 1. Member ID numbers and load cases of the two-span truss.

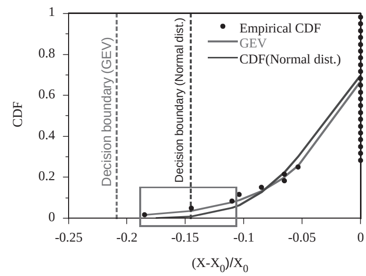

Figure 2. A typical baseline distribution and decision boundaries from normal distribution and the GEV (Member ID #4).

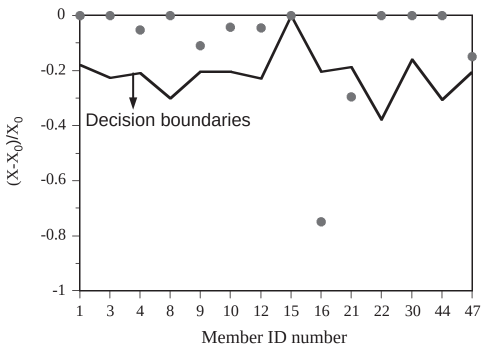

Figure 3. Normalized differences between the system parameters identified from the damaged two-span truss and the baseline values, and decision boundaries using GEV.

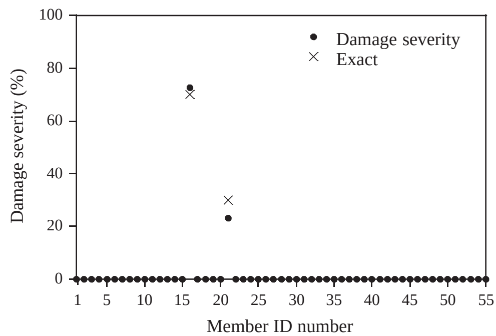

Figure 4. Identified and exact damage severity for the damaged two-span truss.

Proportional random noise generated by a uniform probability function between ±5% noise amplitude is added to the displacement obtained by a mathematical model to simulate real measurements. The significance level α is selected as 0.01 (i.e. 99% confidence level) for the statistical damage classification.

Before damage detection, 30-time Monte-Carlo trials are performed to create the baseline measured responses. Figure 2 shows a typical baseline distribution and the corresponding decision boundary determined by the extreme value distribution (solid line) and normal distribution (dotted line). In Figure 2, it should be noted that the left tail of the empirical distribution in a rectangular box is well represented by the extreme value distribution rather than the normal distribution. This implies that the decision boundary (99% confidence level) determined by the normal distribution is susceptible to false indication of damage rather than the extreme value distribution.

Damage is simulated with 70% and 30% reduction in the sectional areas of two bottom members (member 16 and member 21). The system parameters identified from the damaged structure are depicted as rectangular point in Figure 3. It is shown that the estimated system parameters corresponding to members 16 and 21 are beyond the decision boundaries and the members are accurately identified to be damaged. The damage severity of each member assessed by the statistical approach is given in Figure 4. The damaged members are identified exactly, and the damage severity is accurately estimated by the 1-norm based regularization scheme.

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Evaluation and rating of damaged steel I-girders

H.W. Shenton III
University of Delaware, Newark, Delaware, USA

M. Dawson
2005 NSF REU Fellow, University of Delaware, Newark, Delaware, USA

A. Chavez
2004 NSF REU Fellow, University of Delaware, Newark, Delaware, USA

ABSTRACT: It is not uncommon for bridges with low clearance to be hit by over height vehicles. The extent of damage to steel girders can run from small dents or bends in the bottom flange, to the complete destruction of the girder and bridge. The options for repair usually include: (1) do nothing, (2) repair the damaged girder, (3) replace the damaged girders, or (4) replace the entire bridge. When the damage is so severe, the option to replace a girder or the entire bridge is usually obvious. When the damage is not so severe, the question of whether to repair the girder or not, and how, becomes difficult to answer. When do bends in a steel girder need to be repaired? Should the bridge be closed immediately to traffic or not? The answers to these questions are typically based on experience and engineering judgment.

One way to help answer some of these questions is to conduct an analysis of the damaged girder and to evaluate its load carrying capacity. Put in the context of a typical bridge evaluation, a load rating factor can be determined for the damaged girder. Using the rating factor the bridge engineer can institute a temporary load posting if necessary, and also use the information to make a more informed decision about repairs and closures. A procedure for doing this was developed that makes use of techniques familiar with and available to practicing engineers.

The procedure assumes that the damage caused by vehicle impact has produced a change in the geometry of the cross section along the length of the member. A schematic of a hypothetical damaged girder is shown in Figure 1. It is also assumed that the top flange of the member is still constrained by the deck to be horizontal; consequently, dead and live loads are still applied to the girder in the vertical direction. Note that because of the bent flange the moment of inertia

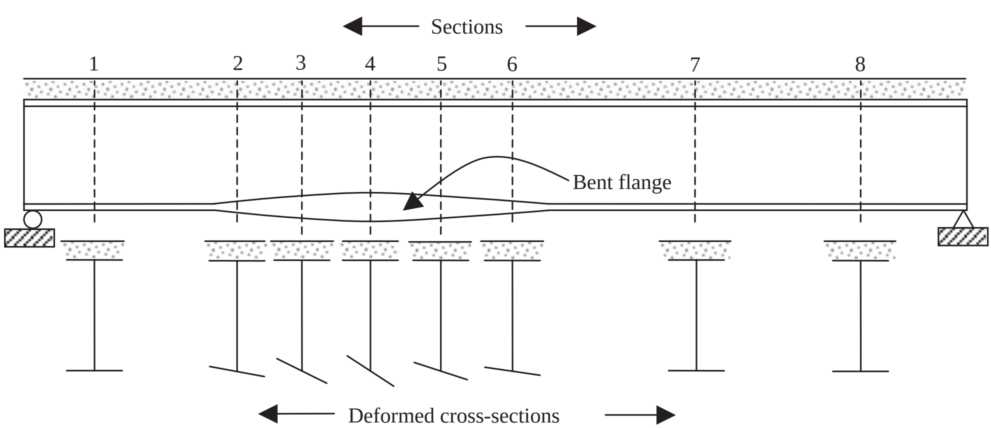

Figure 1. Schematic of hypothetical damaged girder showing bent bottom flange.

of the member now varies along its length. Furthermore, while the original undamaged section is symmetric about the strong and weak axes, the damaged section is not. The result of the new section geometry and the load direction is that the damaged girder is now subjected to a state of unsymmetric bending. This can increase the maximum normal stress in the cross section, beyond what it was designed for, simply because of the change in the geometry.

The capacity of the damaged girder is evaluated as though it were designed to have a varying cross-section along its length. First, sections for evaluating the stress in the cross-section along the length of the girder are defined. The cross-section properties for each location are then calculated based on the deformed geometry at that location. Using the calculated section properties the elastic and plastic moment capacities at each location are determined. Next, the maximum member forces are calculated at the different locations for the rating vehicles. A rating factor is then calculated for each location using the AASHTO LRFR (*Manual,* 2003) procedure, considering the Strength I, Service II, and Fatigue limit states. Finally, the rating for the entire girder is taken to be the smallest rating factor calculated for the girder.

An example case of an over height vehicle collision from Delaware is illustrated. On April 5, 2005, a tractor hauling refinery equipment was traveling northbound on Interstate 95 in Delaware. Around 9:00 a.m. the load struck the 10th Street Bridge in Wilmington, Delaware (Aviola, 2005). The over-height load struck the south fascia girder, producing a minor bend in the bottom flange. The load then struck the north fascia girder, as it passed under the bridge. This girder sustained the most damage; it was bent outward about the web while the flange experienced minor buckling.

A segment, approximately 12.2 m (40 ft) in length, of the north fascia girder was altered from its original configuration. For the analysis, ten sections along the span were defined at distances of 0.1, 3.1, 4.3, 4.9, 5.5, 6.1, 6.7, 8.5, 12.2, 15.3 m from the support. The geometry of each of the cross-sections was created in Section Wizard and the calculated properties input into the developed Excel spreadsheet. Load rating factors were calculated for these sections, for the HS20 truck.

The lowest rating factor for the undamaged girder for the Service II limit state was 1.43. The corresponding rating factors for the damaged girder range from a low of 0.28 to a high if 17.3. This represents a fairly significant decrease in the rating for the bridge. The lowest rating occurs in the region of the largest deformation to the girder. The results show that a segment of girder more than 12 m (40 ft) in length does not satisfy the Service limit state. The rating factors for the undamaged girder for the Fatigue limit state are all well above 1, with the smallest factor being 4.31. The corresponding rating factors for the damaged girder are also all above 1, with the smallest being 2.30. The results show that the damaged girder has adequate capacity for fatigue, but will not satisfy the service limit state. The lowest rating factor for the service limit state could be used to calculate a temporary load posting for the bridge. In this case an appropriate posting would be $0.28 \times 36 = 10$ tons. Information such as this could be used by bridge engineers to direct the post event traffic control and repair procedures.

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On the detection of damage in bridge structures using dynamic testing

V. Zabel & C. Bucher
Institute of Structural Mechanics, Bauhaus-University Weimar, Germany

1 INTRODUCTION

The paper describes the research effort regarding the application of dynamic response measurements to the detection of structural damage as carried out between 1996 and 2005 at the Institute of Structural Mechanics (ISM) of Bauhaus-University Weimar. The studies were initiated by the cooperative research project EXTRA II dealing with the assessment of the structural safety of concrete bridges. The project was supported by the German Federal Ministry of Education and Research.

The main focus of the research activities in this project was directed to the development of engineering tools for the assessment of the static load bearing capacity of existing short and middle span bridges. The developed approaches involve static load tests of the respective structure. In order to verify that the investigated structure is not damaged due to these investigations, the static load tests were combined with dynamic tests. These experimental dynamic investigations are the subject of the first part of the article.

The major advantages of dynamic tests are the facts that they are non-destructive and typically less intricate than static load tests. However, investigations by many researchers as well as our own experience have shown difficulties with respect to the detection of structural damage from identified modal parameters such as natural frequencies. Consequently, several research initiatives are focused on alternative approaches for the extraction of damage indicators from dynamic test data. One methodology that is based on the widely accepted wavelet analysis is presented in the second part of the paper.

2 DYNAMIC EXPERIMENTAL INVESTIGATIONS IN COMBINATION WITH STATIC LOAD TESTS

The common approach for the assessment of the load bearing capacity of an existing bridge is a numerical analysis following to a recognised code of practice. However, if the required information about the structural details is not available or if the result of calculations according to current codes is a load bearing capacity below the limits expected by the bridge owner, a non-destructive experimental investigation may give more detailed or "better" information about the structural behaviour of the specific structure.

Within the research project EXTRA II a concept was developed that is based on static load tests in order to assess the load carrying behaviour of reinforced concrete bridges (Steffens, Bucher, Opitz, Quade, and Schwesinger 1999). In addition to the continuous monitoring of the static deflections of the structure before, during and after the load tests the dynamic structural behaviour can serve as an indicator of the structure's condition before and after the tests. Consequently, dynamic tests were incorporated in the investigation procedure. During research project several reinforced concrete bridges with spans shorter than 20 m were investigated. Especially those bridges which span less than 10 m are very stiff. As a consequence it is very difficult to extract modal information from ambient or traffic induced vibrations. In this project a special drop-weight system and a servo-hydraulic shaker were applied in several tests. The test equipment had not only to satisfy the

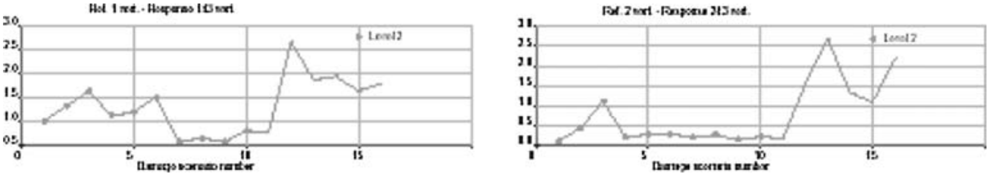

Figure 1. Z24 bridge – evolution of the level energies at level 2 for the transmissibility relation ref. 1 – resp. sensor 143 (left) and ref. 2 – resp. sensor 243 (right) – analysing wavelet Daubechies D4.

requirement of a sufficient dynamic excitation of the structure, it had also to be ensured that closing the bridge for one single night was sufficient for setting up the test equipment and performing both static and dynamic tests (Huth, Bucher, and Meinhold 1999).

3 A WAVELET-BASED DAMAGE-INDICATOR

Not only structural damage can cause modifications of modal parameters of a structure but also environmental factors such as temperature changes. Therefore it might be difficult to distinguish the influence of structural damage on e.g. the natural frequencies from that of other phenomena. As an alternative to the consideration of modal parameters, an approach is presented that analyses wavelet decompositions of data that is acquired during dynamic tests (Zabel 2004; Zabel 2005).

Results obtained from laboratory tests suggest a relatively high sensitivity of impulse response functions' wavelet decomposition energy components. Also a study considering the progressively damaged Z24 bridge in Switzerland has shown that certain damage scenarios could clearly be identified.

As an example, the diagrams in figure 1 indicate various changes of the level energies corresponding to the several transmissibility functions. Specifically, significant changes can be observed due to the settlement of one bridge pier (first 6 damage scenarios) and the settlement of one abutment (failure of a concrete hinge - damage scenario 12). Especially the latter observation is remarkable because from the investigation of modal parameters (Peeters 2000, Maeck 2003) changes due to the failure of a concrete hinge were not reported.

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Research needs for BHM systems of the future and benchmark studies

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)
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Bridge assessment under uncertain parameters via interval analysis

O.D. Garcia & J. Vehí
MiceLab – Modal Intervals & Control Engineering Laboratory, University of Girona, Department of Electronics, Informatic and Automation, Girona, Spain

J.C. Matos, A.A. Henriques & J.A. Figueiras
LABEST – Laboratory for the Concrete Technology and Structural Behaviour – Faculty of Engineering – University of Porto, Civil Engineering Department, Porto, Portugal

J.R. Casas
Civil Engineering Department, Technical University of Catalonia, Barcelona, Spain

An efficient health monitoring scheme for damage detection in civil engineering structures, like bridges, using on-line monitoring is being developed in order to identify any possible damage. The main idea is to perform structural assessment helping owners in achieving a better structural maintenance. The basis of the present work is the treatment of uncertainties, which are among the basic common difficulties to face when modelling structures. Civil engineering infrastructures contain uncertainty in their physical or geometric parameters, such as Young modulus, Poisson ratio or length. In this work, uncertainties are described as interval values. This means that each parameter is not quantified as a fixed real number, but as taking any value between the lower and the upper limits of an interval. Within this approach the input parameters of civil engineering models are replaced by intervals (Garcia et al. 2004). The result of this process is also an interval value which is obtained using the system of equations and theorems of modal interval analysis (*MIA*).

Typical long term monitoring schemes are composed by sensory, data acquisition, communication, data processing and archiving and damage detection and modelling systems (Mufti 2001). A methodology, based on *IA* theory (Hansen 2004, Moore 1966, Neumaier 1990) applied to a numerical *CSP* (Shary 2002), was selected and will be implemented in the damage detection and modelling system of a long term monitoring project in order to reach such objective. An algorithm is being developed to use such methodology with the obtained data.

The problem of damage detection in a bridge can be stated as a numerical Constraint Satisfaction Problem (*CSP*) for which its inconsistence has to be proved. *CSP* combined with the Interval Analysis (*IA*) is a powerful tool for the study of uncertainty present in the structural parameters (Casas et al. 2005).

A *CSP* is said to be inconsistent if one or more of its constraints are inconsistent, what is expressed by the following expression:

$$CSP\ is\ inconsistent\ if\ (\forall \mathrm{x} \in \mathrm{X})\{\neg(C_1(\mathrm{x})), \vee ..., \vee \neg(Cm(\mathrm{x}))\} \quad (1)$$

When constraints are under the form $\boldsymbol{C}(\boldsymbol{x}) = f(\boldsymbol{x}) = 0$, with f a continuous function from $\mathbb{R}^n$ to $\mathbb{R}$, the logic formulation needed for proving the inconsistence of a constraint $\boldsymbol{C}_i(\boldsymbol{x})$ is as follows:

$$(\forall \mathbf{x} \in \mathbf{X}) \neg (f_1(\mathbf{x}) = 0) \quad (2)$$

Inconsistence of a *CSP* has been reduced to proving the exclusion of zero from the range of a set of continuous functions.

Such methodology has been first checked in laboratory with a simple reinforced concrete structure that was loaded up to failure (Fig. 1). The obtained results are very promising and the cracking load was identified. The majority of structures present a linear elastic behaviour during almost all life. However they tend to deteriorate and such degradation reflects on results obtained from

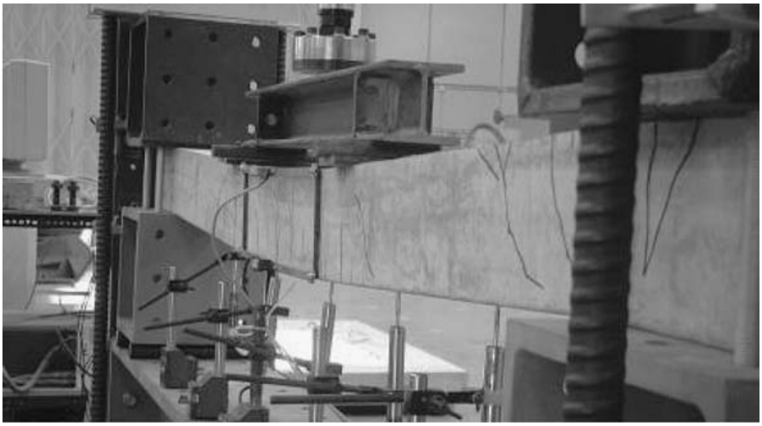

Figure 1. Laboratory test for structural assessment.

the long term monitoring system. Structural assessment was performed in this case with success, enabling it application into real structures.

Within a scope of a research project developed in Portugal (SMARTE Project) a pre-stressed concrete bridge is continuously and remotely monitored (Matos et al. 2004, 2005a, b, Figueiras et al. 2004, Sousa et al. 2005). Such bridge will be a real prototype structure where the methodology will be implemented for the first time.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Suggestions for future research, development and application of bridge health monitoring systems

J.M.W. Brownjohn
University of Sheffield, UK

J.S. Owen
University of Nottingham, UK

ABSTRACT: Large investment has been made in a number of operational systems worldwide that have been self-classified as structural health monitoring (SHM), and there is a proliferation of worthy research results also generically categorized as SHM. While SHM has tremendous promise for infrastructure operators, it is a misunderstood concept and in many countries has achieved very little practical impact. This is partly due to the present very wide definition and the inability of the SHM community to deliver clear messages about purpose and capability of existing and emerging technologies. Another reason is data overload and the inability of most systems to go beyond simple data presentation to interpretation and diagnosis.

This paper reviews the history of SHM, some successes and failures on the wider range of civil infrastructure then briefly describes some past bridge monitoring projects and the lessons learnt from them.

Some suggestions are made for design and implementation of future systems including the need for having clear requirements on the nature and purpose of information to be generated, employing reliable technology alongside new systems, having a versatile and upgradeable data management and mining system, and a user friendly interface for timely presentation of information for decision making.

1 INTRODUCTION: EVOLUTION OF STRUCTURAL HEALTH MONITORING

Structural health monitoring (SHM) is a term that has been increasingly used, mainly since the early 1990s, to describe a technologies whose collective aim is assessment of condition and performance of civil, mechanical and aerospace structures. In civil engineering applications these technologies began life as 'surveillance' (e.g. of dams), structural/integrity monitoring (e.g. of offshore platforms and major bridges) and condition assessment (of shorter span bridges e.g. for posting or load capacity evaluation). In mechanical and aerospace fields the term derivse from condition monitoring (e.g. of rotating machinery), structural integrity assessment (often related to fatigue performance) and non-destructive testing/evaluation (looking for cracks and defects).

Within civil engineering, due to transfer of technology and inter-disciplinary research can now be interpreted as covering all these applications. In fact Ross and Matthews (1995a) and Mita (1999) identified the span cases where structural (health) monitoring may be used:

1. Checking effectiveness of modifications to an existing structure
2. Monitoring of structures that might be affected by external works such as demolition
3. Tracking long term movement/settlement/degradation of structures and materials
4. Using observed structural performance to improve future design and develop codes

5. Fatigue assessment
6. Evaluating novel systems of construction
7. Assessing post-earthquake structural integrity and safety
8. To assist structure maintenance programs via objective performance indicators
9. To provide norms for performance-based design

SHM in it's present form is effectively continuous system identification of a physical or parametric model of the structure using time dependent data. The signals used in SHM derive not only from vibrations but also from slowly changing quasi-static effects such as diurnal thermal cycles. Once a baseline system model is identified, SHM procedures are aimed to identify occurrences when output signals do not correspond to predictions based on the established form.

While the term SHM has only recently been used, historically, monitoring of structures has involved many activities now viewed as SHM, e.g. data collection, processing and diagnosis.

More mainstream SHM research, as reported in the growing number of journals and conference proceedings, aims to develop effective and reliable means of acquiring, managing, integrating and interpreting structural performance data for maximum useful information at minimum cost while either removing or supplementing the qualitative human element.

If SHM technology is to move significantly from academic/technology-driven mode to operator-driven mode, SHM needs to be sold as a complete solution with clearly defined but limited capabilities and goals. This is how a number of commercial organizations (e.g. VCE and Smartec) are successfully marketing. There now sound financial reasons for installing SHM systems whether or not proving or checking performance is built into a construction or retrofit contract, but in some cases there may be major concerns about liability or loss of value due to knowledge of less than perfect structural performance.

This paper summarises historical developments and describes monitoring programmes on Humber and Kessock bridges in the UK and Pioneer and Tuas Bridges in Singapore and summaries the conclusions from their operation that led future developments.

Also based on these experiences and from the historical lessons, some recommendations about future SHM are presented. These include the need for education of operators, leading to well thought out specifications and requirements, optimal systems with robust sensors and infrastructure, finishing with the challenges for future signal interpretation/data mining technology and interfaces with the users. Clearer understanding and definition of the relevant performance indicators as well as the types of damage or structural degradation and how they manifest through performance are required.

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Development of a benchmark problem for bridge health monitoring

F.N. Catbas
University of Central Florida, Orlando, FL, USA

J.M. Caicedo
University of South Carolina, Columbia, SC, USA

S.J. Dyke
Washington University, Saint Louis, MO, USA

ABSTRACT: An international effort has been initiated under the auspices of the International Association for Bridge Maintenance and Safety (IABMAS) to facilitate a systematic examination of bridge health monitoring methodologies through the development of benchmark problems. The purpose of this paper is to propose an analytical bridge health monitoring (BHM) benchmark problem directed toward medium-span bridges. This paper outlines the background and motivation for this benchmark problem, describes the structure considered, and defines the criteria to be used for evaluation of the results of this problem. For this numerical benchmark study, the structure will be evaluated for different conditions which were identified based on challenges proposed by both researchers and bridge engineers from state and federal agencies.

1 INTRODUCTION

In spite of the recent activity in BHM and the successes achieved to date, this technology has not been gaining acceptance by bridge owners and engineers. There is a growing gap between the theory and the practice that must be filled prior to broader implementation of these techniques. During the 2004 International Association for Bridge Maintenance and Safety (IABMAS) Conference in Kyoto, Japan, the Bridge Health Monitoring Committee was formed under the auspices of IABMAS to address the challenges in BHM. The committee discussed the origins of this gap between theory and practice and identified several challenges currently faced by structural health monitoring of bridges. At a subsequent meeting open to all interested parties during the 2005 SPIE meeting in San Diego a plan for a benchmark problem was formulated.

It was suggested that the delay in implementation of the technology is due in part to the lack of examination and validation of the reliability of these techniques for real world applications. Thus, the benchmark problem proposed herein focuses on one of those challenges that has not been examined in great detail in previous studies, the reliability of techniques.

2 BENCHMARK PROBLEM DEFINITION

The goal of this benchmark problem is to assess the reliability of health monitoring methodologies for a typical bridge structure. Participants will create static and/or dynamic simulations using programs provided to simulate the healthy and damaged structures to develop and fine tune their methodologies. The bridge specimen at the University of Central Florida has been selected as the subject of this BHM. This choice will facilitate a later phase of the benchmark problem focusing on experimental data. This bridge model is representative of a significant percentage of the U.S. national bridge inventory, and thus is a logical starting point for any bridge research. Details of the structure and problem statement are available at http://people.cecs.ucf.edu/catbas/benchmark.htm. A finite element model (FEM) of the structure, formed in SAP 2000 and exported to Matlab, is used for generation of the simulated data. Several realistic damage cases were proposed by bridge engineers and are incorporated into this problem statement.

2.1 *Damage scenarios*

Commonly observed damage scenarios were identified through discussions with transportation officials so that the direction of the benchmark problem would ultimately end in a realistic problem statement. In response to feedback from the DOT officials in the three states previously listed and from a meeting held at the SPIE conference in San Diego, California in 2005, three recurring scenarios have been identified and are proposed as damage scenarios: i) scour, ii) changes in the characteristics of the support bearings, and iii) local loss of stiffness.

2.2 *Numerical simulation and evaluation criteria*

Several researchers have expressed their interest in the development of damage assessment methods based on static tests given the availability of appropriate sensors and more frequent use by the bridge owners. In addition, structural health monitoring using dynamic data has been the focus of many researchers during the last decade. This benchmark problem offers participants the opportunity to use either dynamic or static tests facilitating side-to-side comparisons of these two classes of methodologies and encouraging discussion about the relative advantages and disadvantages of each. Several different types of sensors are simulated in this benchmark problem. Global displacements and strains at select locations are available for participants when static test simulations are performed and acceleration and strain records are available for dynamic simulations.

Participants have the choice to select between two types of simulations. *Training simulations* provide participants with test data of one of the damage scenarios included in the benchmark problem. The damage pattern and level is selected by the participant. Researchers can determine the effectiveness of their methodology and improve and tune their algorithms using this data. The reliability of the methodology is evaluated using the *evaluation simulation*. Evaluation simulations consist of a set of 100 records including damaged and undamaged cases. The reliability of the methodology will be evaluated based on the effectiveness of the methodology identifying which cases are damaged and which cases are undamaged. Three evaluation criteria to evaluate the reliability of the participant's health monitoring strategies using the evaluation simulations are proposed. The first evaluation criteria measures the percent success rate of the damage identification technique, the second evaluation criteria quantifies the percentage of false positives identified by the methodology and the last evaluation criteria measures the false negatives detected with the methodology.

3 SUMMARY

This paper describes the first phase of a benchmark problem for structural health monitoring of highway bridges. The goal of this benchmark problem is to examine the reliability of promising structural health monitoring techniques. A numerical model of a grid model available at the University of Central Florida is considered. The damage scenarios considered herein were selected based on several common damage cases suggested by bridge engineers. These include scour, changes at the boundary supports, and reduction in the stiffness at a connection. This initial first phase will likely be followed by additional numerical and experimental studies considering this test structure. Future efforts will also include other important parameters involved in real-world damage detection such as cost analysis and decision making based on the data supplied by the health monitoring system.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Application of ARMAV for modal identification of the Emerson Bridge

W. Song, D. Giraldo, E.H. Clayton & S.J. Dyke
Washington University, Saint Louis, MO, USA

J.M. Caicedo
University of South Carolina, Columbia, SC, USA

ABSTRACT: Large flexible structures such as cable-stayed bridges pose a challenge to structural health monitoring due to their service condition, accessibility and complexity. To provide an accurate estimate of structural damage, the reliable identification of modal properties is a prerequisite. Although forced vibrations provide accurate quantitative modal information, the use of ambient loading constitutes an attractive alternative in terms of cost and simplicity. The auto-regressive and moving average vector (ARMAV) technique is one of the most promising techniques to make use of ambient vibration data. Based on the real-time data measured from the well known cable-stayed bridge (Bill Emerson Memorial Bridge), this paper presents a modal identification process through ARMAV technique.

Below is a diagram showing the location of the sensors on the bridge. There are a total of 84 accelerometers installed on the bridge and in the surrounding soil. As shown by the arrows in Figure 1, in the modal analysis performed in this paper, only 16 vertical motion signals on the bridge deck were used. Each arrow represents two sensors, which are located along the two edges of the deck.

The ARMAV technique is employed to analyze the recorded ambient vibration data from sixteen of the eighty-four channels of acceleration from the bridge. Furthermore, several techniques, for instance, elimination by modal properties, and stability diagram, have been used to remove the non-physical modes during identification process. To validate the results, the modal identification results obtained through ARMAV are compared to the modal parameters from an existing (without updating methods) finite element model of the bridge built based on the drawings. Moreover, FPE and AIC criteria have been employed to show the influence of ARMAV model order on the final prediction error.

Five vertical vibrational modes below 1 Hz are identified and compared with the model. The natural frequencies and mode shapes are shown below. Good agreements have been obtained, which demonstrate the effectiveness of ARMAV techniques in the modal identification of such kind of structure. Further study will be carried out based on more data channels and concentrated on identification of coupled and closely-spaced vibrational modes.

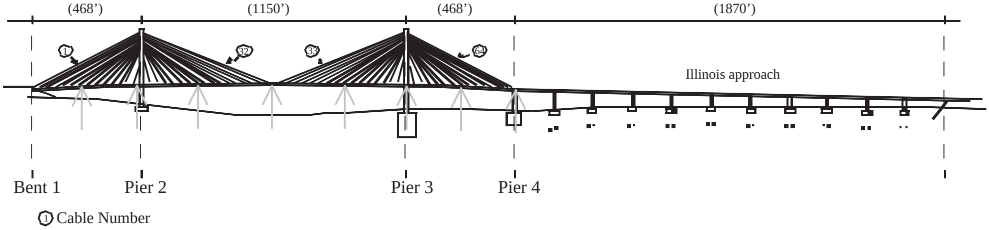
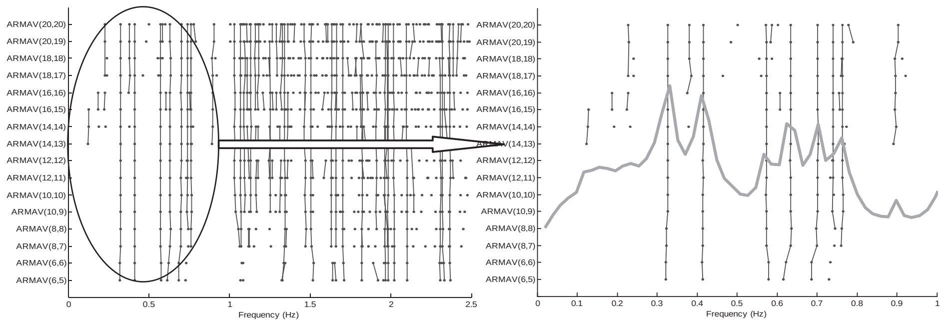


Figure 1. Sensor placement on Emerson Bridge (Arrows denote the sensors).



(a) Frequency range < 2.5 Hz

(b) Frequency range < 1 Hz (The curve indicates the average PSD over all the channels)

Figure 2. Stability diagram.

Table 1. Modal Frequencies comparison.

Modal Freq.*	ARMAV	PSD	FEM
1	0.3264	0.332	0.2884
2	0.4152	0.4102	0.3849
3	0.5737	0.5664	0.4381
4	0.6329	0.625	0.5993 (5th mode)
5	0.7009	0.7031	0.6635 (6th mode)

*Identified frequencies are less than 1 Hz.

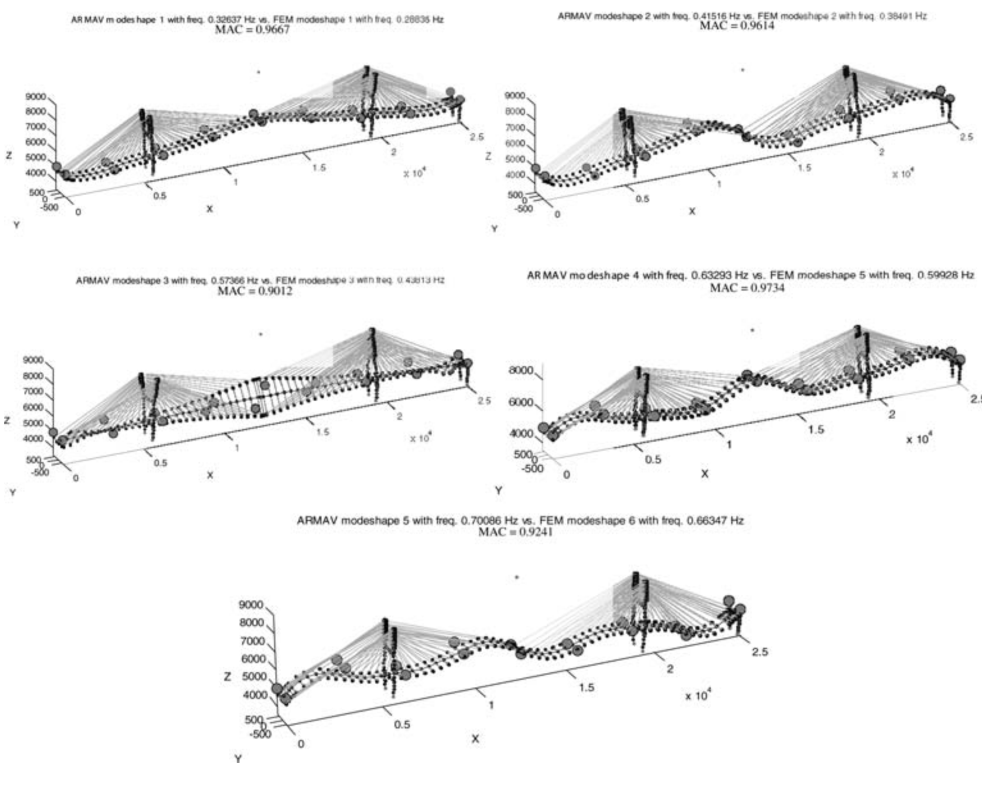


Figure 3. Mode shapes comparison between FEM and ARMAV (dots denote the mode shape calculated by ARMAV).

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Improvement of seismic performance of the Toyosato Bridge with base isolation and response control

H. Iemura
Kyoto University, Kyoto City, Kyoto, Japan

Y. Kawamura & M. Hirano
Public Works Bureau of Osaka City, Osaka City, Osaka, Japan

K. Taira
Sogo Engineering Co., Ltd., Osaka City, Osaka, Japan

ABSTRACT: The Toyosato Bridge is a three-span continuous steel cable-stayed bridge with a main span of 216 m (Figure 1) crossing the Yodo River which flows in the northern region of Osaka City. The bridge was constructed in 1970, after that installation of unseating prevention system against bearing failure was executed.

The supporting condition in bridge axis direction is one fixed support for both the cable-stayed bridge and the approach bridges that were designed to resist Level 1 Earthquake Ground Motions with response acceleration of 200 gal. Therefore, the bridges have extremely low seismic performance against Level 2 Earthquake Ground Motions such as the Hyogo-ken Nanbu Earthquake that has response acceleration of 1000 gal. As the result, it was clarified that the bridge has strong possibilities of suffering critical damages, such as brittle fracture of the piers due to the inadequate shear strength and falling down of the girder.

Under such circumstance, the improvement of the total seismic performance of the whole bridge system is examined in this paper by decreasing seismic force through natural period elongation.

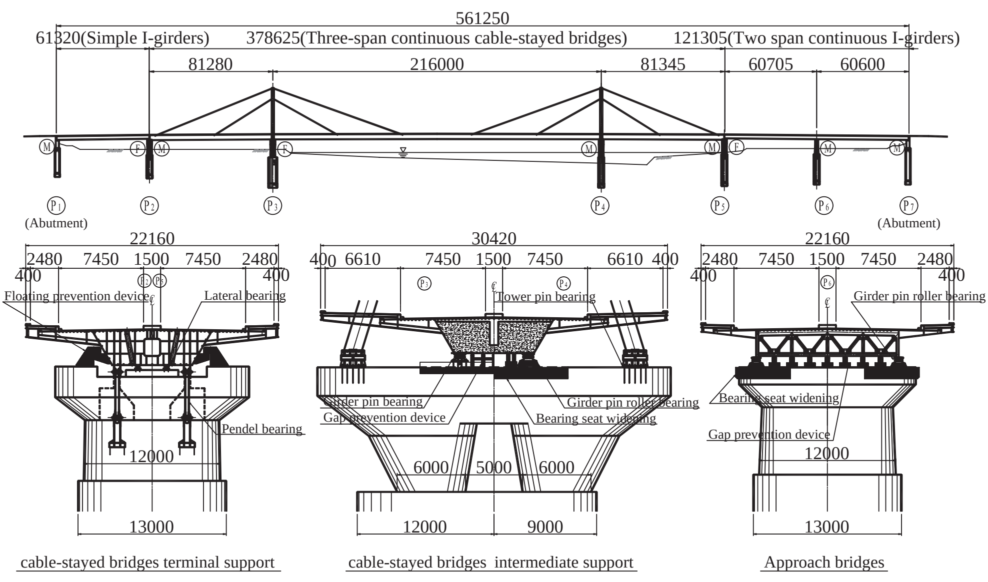


Figure 1. General figure of the Toyosato Bridge.

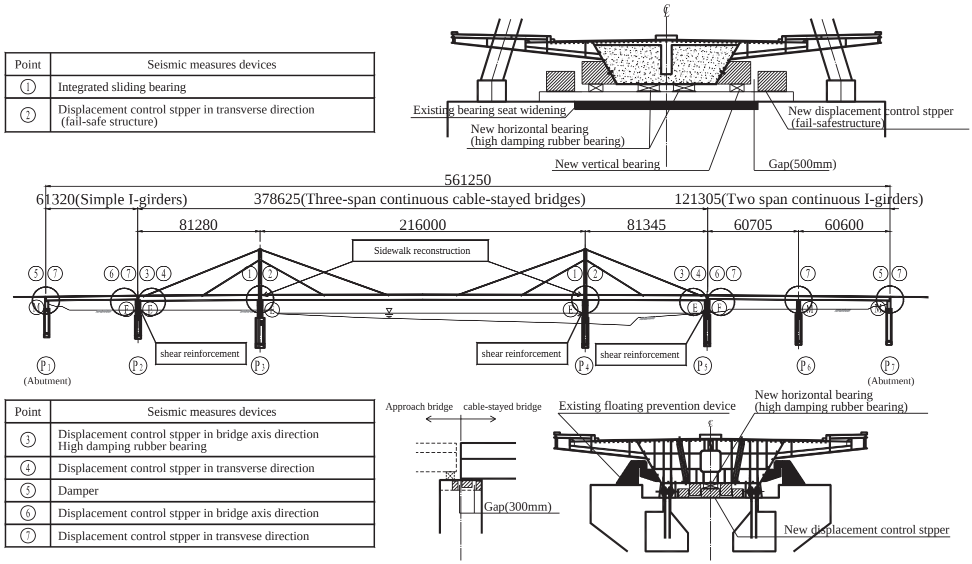


Figure 2. Selected seismic performance improvement.

Table 1. Effect obtained by the selected seismic performance improvement.

Item	Static checking method	Dynamic checking method
	Details of reinforcement	Details of reinforcement
P2 pier	t = 300 mm Bending, Shear and Ductility	t = 250 mm Shear
P3 pier	t = 500 mm Bending, Shear and Ductility	Reinforcement is unnecessary
P4 pier	t = 300 mm Bending, Shear and Ductility	t = 250 mm Shear
P5 pier	t = 300 mm Shear and Ductility	t = 250 mm Shear
P6 pier	t = 300 mm Bending, Shear and Ductility	Reinforcement is unnecessary
Remarks	PC steel rods are necessary	PC steel rods are unnecessary
Influence on foundation	Large	Small
Approximate cost ratio		
Superstructure	0.15	0.48
Substructure	1.15	0.52
Total	1.30	1.00

Seismic performance improvement shown in Figure 2 was selected which satisfies the required seismic performance of the bridge. Furthermore, the comparison result of this improvement with the one derived from static checking method is shown in Table 1.

The selected seismic performance improvement could make number of piers which need seismic retrofit into half. Even for those piers that need reinforcement, the retrofit works could be minimized for fortifying against only shear strength. Therefore, total cost of the reinforcement for the superstructure and the substructure could be decreased. In addition, this could result to avoid damages to the existing piers such as drilling holes, because it became unnecessary to install PC steel rods that penetrate through each pier of the bridge. From the above, the cost minimizing effect and the effectiveness of the selected seismic performance improvement were verified.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Residual strength prediction of reinforced bridge piers under seismic risks

I. Aoki & T. Koike
Musashi Institute of Technology, Setagaya-ku, Tokyo, Japan

ABSTRACT: This paper describes the residual strength prediction of reinforced bridge piers to prevent structural damages due to seismic disaster.

Many existing infrastructures which are deteriorated by corrosive and climate conditions are always threatened by various natural hazards including earthquake loads. Actually, those structures constructed prior to 1980 in Japan were designed for a particular seismic load which is smaller than the Level 2 ground motion caused by the ductility level earthquake (DLE). After the 1995 Hyogoken-Nanbu earthquake, revised guidelines which introduce Level 1 and Level 2 ground motions as the design earthquake load were specified making old infrastructure design fall below acceptable limits. This means these structures are vulnerable to strong earthquake in the future.

The pier of reinforced concrete column bridge has two typical modes of shear and bending failures. Recent maintenance activities for urban highway bridges in Japan are carried out by wrapping the column with steel plates to protect it from shear failure prior to bending failure. Also the revised seismic design guideline for new bridges was issued to provide adequate bending rigidity to columns. It should be noted, however, that this retrofit scheme is not always adequate to prevent failure in both modes.

A structure's original strength generally degrades during its service period, because of the aggressive environment that contributes to the deterioration effects. Corrosion effect often appears in the reinforcing steel bars of the RC column. On top of this, the seismic impact adds significant damage to the structure. So the repair works after the quake must be conducted for both seismic damage and for the daily deterioration damage.

Based on the reliability analysis of an existing bridge pier after possible future earthquakes, the present study discusses various fragility curves which are necessary to obtain the optimal maintenance strategies of deteriorating infrastructures under seismic risks.

A single column RC bridge pier is a typical structural profile used in a highway bridge in Japan. Fig. 1 shows a schematic example of a bridge structure which is designed to resist failure due to bending moment and shear force which is produced by structural responses from earthquake effects.

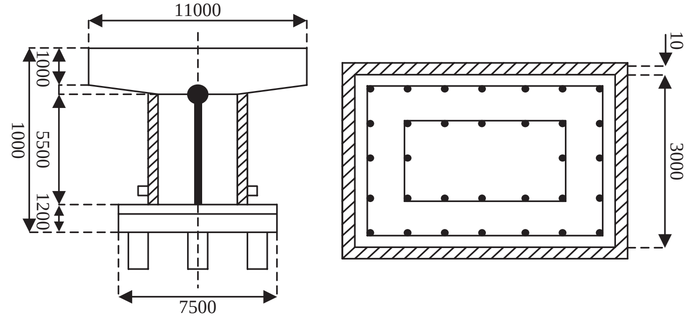


Figure 1. Reinforced bridge pier with steel plate.

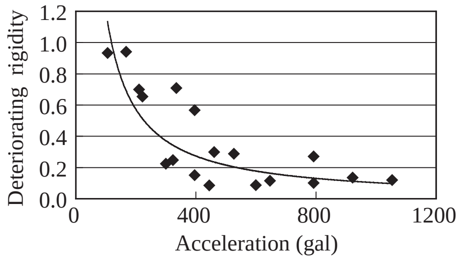


Figure 2. Deteriorating bending rigidity resulting from seismic damage.

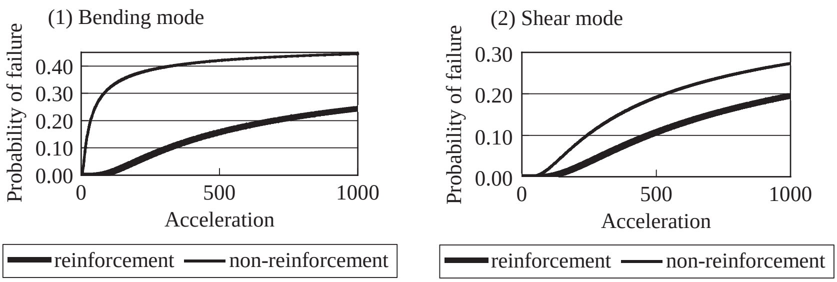


Figure 3. Fragility curves of the reinforcing and non-reinforcing RC column in the bending and shear modes after an earthquake.

When an additional steel plate cover with its thickness of Δw is applied around the RC column, the inertia of moment and cross sectional area of RC column can be modeled as

$$\hat{I}(t,\Delta w)=\hat{I}(t)+\Delta I(\Delta w), \qquad \hat{A}(t,\Delta w)=\hat{A}(t)+\Delta A(\Delta w)$$

When an earthquake denoted by EQ is applied to the bridge structure, any structural damage will be limited to the internal structure of the RC column. This damage could appear as a degradation of bending rigidity of the column after the earthquake. Fig. 2 shows numerical results of structural responses of a bilinear single degree freedom system for various earthquake excitations.

Fig. 3 compares the reinforcement effect by adding a steel plate around the RC column in which the probability of failure in the major damage mode for the reinforcing column shows less probability than that of non-reinforcing one. This trend appears for both modes of bending and shear strength. Especially, the reinforcement can provide a greater improvement in bending rigidity than that in shear capacity.

The numerical results can be summarized below:

(1) Fragility curve is a useful measure to estimate the structural performance of bridge piers under rigidity degradation due to seismic damage accumulation.
(2) Corrosion effect is not so significant in the residual strength estimate if the quality control for corrosion damage is adequately applied.
(3) Reinforcing the column by adding steel plate covers improves the column bending rigidity than the shear capacity.

Integration of bridge management and bridge monitoring

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Health monitoring system using learning system

H. Furuta
Department of Informatics, Kansai University, Osaka, Japan

H. Hattori
Graduate School of Informatics, Kansai University, Osaka, Japan

1 INTRODUCTION

Recently, many researches have been made on health monitoring of existing structures such as buildings, bridge and other civil structures. Many structures are becoming superannuated and deteriorated. Furthermore, in Japan, natural disasters like typhoon and earthquake have occurred frequently so that the damage assessment of existing structures is very important. In order to evaluate the damage state of structures, health monitoring technology is quite promising to provide useful information. In the health monitoring, there are still some problems in modeling, analysis and experimental examination for practical use. In general, it is impossible to detect the existence of deterioration and identify the position of deterioration from only the observed information of displacement. However it becomes possible to do them if the information of displacement of intact situation is available. Of course, accurate analysis or model examination is inevitable to obtain the displacement of intact situation. Then, it requires a lot of time and cost.

At the previous research, a health monitoring system that can adapt to the structural systems and environments through the learning ability was developed which can obtain the recognition rate of over 80%.

In this paper, an attempt is made to improve the previous system by introducing GMDH (Group Method of Data Handling) into the learning system. In the previous research, Fuzzy-Neural network was used for the learning system. Although the previous method shows a good performance, it still has some room to be improved. In this paper, GMDH is employed to improve the performance, because it shows a higher prediction ability in the time series analysis.

2 STRUCTURE HEALTH MONITORING

Recently, health monitoring system has gained attention in Japan because of the great loss due to the Hyogoken-Nambu Earthquake in 1995. In addition, a lot of structures constructed during the high economic growth period have becoming superannuated, so that there arises a big problem that bridge structures cause falling of concrete fragment in Japan. Therefore, the necessity of the health monitoring of structures is taken up as a major subject.

Structure in service is exposed to the risk of deterioration by aging or fatigue and vibration. Conventionally, structural maintenance has been done by routine repair works, however, there are such problems as labor, time, and cost in the daily maintenance work. Then, it is desirable to solve these problems by introducing the health monitoring system. Although the health monitoring system can provide sufficient data in quantity and quality, it also needs a lot of preparation and cost. In order to reduce the labor and cost for the health monitoring, it is necessary to collect data with ease and interpret the data obtained well.

Table 1. Recognition rates.

Learning system	Intact situation	Deterioration situation
GMDH	0.822	0.844
Fuzzy-neuro	0.702	0.835

3 PROPOSED SYSTEM

In this paper, an attempt is made to develop a structural health monitoring system that can diagnosis the deterioration from the displacement of structure. The proposed system compares the intact situation and the observed data. However, since it is difficult to obtain the value of intact situation, the proposed system uses the prediction value instead of the intact situation. The proposed system is composed of prediction part; learn the vibration response and predict the next response, judgment part; detect the deterioration by analyzing the vibration response, and simulation part; analyze the vibration. Inputs to the proposed system are external force and vibration responses; displacement and velocity of the structures. Outputs from the proposed system are probability of deterioration and position of deterioration.

At the prediction part, the vibration response; displacement and velocity of the structure is predicted at the next step from the current responses and external force. In this research, AdaBoost is used to predict the structural responses, and therefore versatile rule can be obtained and the calculation time becomes shorter.

At the judgment part, the deterioration is detected by comparing prediction values and observed values. The proposed system identifies the existence of deterioration using the fuzzy reasoning. Input data of the fuzzy reasoning are errors and error rates. By using the fuzzy reasoning, it is possible to make the calculation time shorter. The position of deterioration is identified by comparing error of each story as well as the detection of deterioration. In this research, the biggest difference of errors between stories shows the deterioration position.

4 NUMERICAL EXAMPLE

A numerical examination is presented by using actual wind velocity data. The proposed system learns those vibration responses in advance. It is considered that no degradation occurs until the 10,000 step though small errors occur. On the other hand, some degradation may occur around the 20,000 step because of the abrupt change of errors. Comparing the error of each story, it is possible to find out the position of degradation. In this case, the proposed system can recognize that the degradation occurs at the third story. Table.1 shows the recognition rates of degradation location.

From Table 1, it is seen that miss-recognition rate is high at the intact situation because the prediction error is rather large. However, the proposed system can diagnosis the deterioration over 80% recognition rate. It can be concluded that the proposed system can recognize the existence of deterioration and its position.

5 CONCLUSIONS

In this paper, an attempt is made to improve the previous system by introducing GMDH into the learning system. By introducing the learning ability, it is unnecessary to prepare any previous knowledge and examination for the underlying structures and environment.

The proposed system consists of prediction part; learn the vibration response and predict the next response by GMDH and judgment part; detect the deterioration by analyzing the vibration response. By comparing the vibration responses at intact state and deteriorated state, it is evident that the vibration responses change when deterioration increases. Thus, the proposed system can recognize existence of deterioration. For a multi-degree-of-freedom models, changes of vibration responses differ by the position of deterioration. Herewith, the proposed system can possibly recognize the position of deterioration. Furthermore, the recognition rate is improved by using GMDH.

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Damage identification method for bridges from a pseudostatic formulation of bridge-vehicle interaction system

C.W. Kim & M. Kawatani
Kobe University, Kobe, Japan

ABSTRACT: This study proposes a damage identification method for bridges based on the traffic-induced vibration data, and shows numerical simulations. An advantage of using the traffic-induced vibration data of bridges in the damage identification may be ease of exciting the bridge. Another important point is that some highway bridges are tested before and during in service such as a moving vehicle test. It means that there exist traffic-induced vibration data available for their health monitoring. Moreover, if vehicle motions (acceleration, etc.) are measured, the data can be used for identifying external loads. The method described in this study adopts a damage identification procedure in time domain considering the physical phenomena behind the bridge-vehicle interaction problem such as time varying coefficients (damping and stiffness matrices) in the system.

The basic idea of the damage identification proposed in this study is to detect damages and quantify the severity of the damage directly from the change in element stiffness using the element stiffness index (ESI) defined as the ratio of damaged flexural rigidity of a member to undamaged one, which indicates the normalized changes in the element stiffness. The mass and damping matrices of a bridge are assumed to be unaffected by the damage.

The validity of the proposed method is demonstrated by a numerical simulation study on a bridge with span length of 40.4 m which is idealized as the beam element with two degrees of freedom (DOFs) at each node. A dump truck with the gross weight of 191 kN is used as the vehicle traveling on the bridge. The dump truck is assumed to the model with 2DOFs, which considers the bounce and pitching motions of the vehicle. Measured roadway profiles on the bridge of the numerical study are considered in the simulation.

Five damage scenarios are adopted as shown in Figure 1. The 1% white noise is added to the calculated responses to simulate the contaminated measurements. The effect of vehicle's traveling speed on the accuracy of the damage identification is also investigated.

Estimated ESI values based on noiseless and polluted vibration data of the bridge are summarized in Table 1 and Table 2, respectively. The Ref. in Table 1 denotes the reference ESI value, whereas the Err is the error between the reference ESI value and estimated one. V1, V2 and V3 in Table 1 (also in Table 2) respectively indicate the vehicle's traveling speeds of 20 km/h, 40 km/h and 60 km/h. The summarized result in Table 1 shows that the proposed method is able to identify severity of damages within the error less than 3%. On the other hand the noise decreases the detection accuracy to the error around 10% as shown in Table 2.

	No.1	No.2	No.3	No.4
Scenario I:	1.00	**0.80**	1.00	1.00
Scenario II:	**0.80**	1.00	1.00	1.00
Scenario III:	1.00	**0.80**	**0.80**	1.00
Scenario IV:	**0.90**	1.00	1.00	**0.80**
Scenario V:	**0.95**	1.00	**0.90**	**0.85**

Figure 1. Damage scenarios.

Table 1. Damage scenarios and identified results (no noise).

		Element											
		No. 1			No. 2			No. 3			No. 4		
Scenario		V1$^{\#}$	V2**	V3^{++}	V1	V2	V3	V1	V2	V3	V1	V2	V3
I	Ref.*	1.000			0.800			1.000			1.000		
	ESI	**0.992**	**0.999**	**0.985**	**0.784**	**0.782**	**0.790**	**0.994**	**0.997**	**0.990**	**1.005**	**0.999**	**1.014**
	Err^{+}(%)	**−0.8**	**−0.1**	**−1.5**	**−2.0**	**−2.3**	**−1.3**	**−0.6**	**−0.3**	**−1.0**	**0.5**	**−0.1**	**1.4**
II	Ref.	0.800			1.000			1.000			1.000		
	ESI	**0.776**	**0.778**	**0.778**	**1.000**	**1.001**	**0.998**	**0.996**	**1.000**	**0.998**	**1.005**	**1.001**	**1.000**
	Err (%)	**−3.0**	**−2.8**	**−2.8**	**0.0**	**0.1**	**−0.2**	**−0.4**	**0.0**	**−0.2**	**0.5**	**0.1**	**0.0**
III	Ref.	1.000			0.800			0.800			1.000		
	ESI	**0.978**	**0.980**	**0.981**	**0.789**	**0.800**	**0.800**	**0.797**	**0.800**	**0.799**	**0.982**	**0.981**	**0.978**
	Err (%)	**−2.2**	**−2.0**	**−1.9**	**−1.4**	**0.0**	**0.0**	**−0.4**	**0.0**	**−0.1**	**−1.8**	**−1.9**	**−2.3**
IV	Ref.	0.900			1.000			1.000			0.800		
	ESI	**0.895**	**0.900**	**0.895**	**0.998**	**0.997**	**0.998**	**0.998**	**1.003**	**0.998**	**0.781**	**0.776**	**0.782**
	Err (%)	**−0.6**	**0.0**	**−0.6**	**−0.2**	**−0.3**	**−0.2**	**−0.2**	**0.3**	**−0.2**	**−2.4**	**−3.0**	**2.3**
V	Ref.	0.950			1.000			0.900			0.850		
	ESI	**0.948**	**0.958**	**0.953**	**0.996**	**0.995**	**0.993**	**0.896**	**0.902**	**0.897**	**0.854**	**0.848**	**0.850**
	Err (%)	**−0.2**	**0.8**	**0.3**	**−0.4**	**−0.5**	**−0.7**	**−0.4**	**0.2**	**−0.3**	**0.5**	**0.2**	**0.0**

*Ref: Reference ESI value; $^{+}$Err: Error between the reference and estimated ESI values; $^{\#}$V1: $v = 20$ km/h; **V2: $v = 40$ km/h; $^{++}$V3: $v = 60$ km/h.

Table 2. Damage scenarios and identified results (1% noise).

		Element											
		No. 1			No. 2			No. 3			No. 4		
Scenario		V1	V2	V3	V1	V2	V3	V1	V2	V3	V1	V2	V3
I	Ref.	1.000			0.800			1.000			1.000		
	ESI	**0.982**	**0.993**	**0.955**	**0.806**	**0.804**	**0.818**	**1.011**	**0.997**	**1.004**	**1.035**	**1.027**	**1.037**
	Err (%)	**−1.8**	**−0.7**	**−4.5**	**0.8**	**0.5**	**2.3**	**1.1**	**−0.3**	**0.4**	**3.5**	**2.7**	**3.7**
II	Ref.	0.800			1.000			1.000			1.000		
	ESI	**0.744**	**0.750**	**0.786**	**1.047**	**1.037**	**1.029**	**1.021**	**1.032**	**1.044**	**1.065**	**1.007**	**1.039**
	Err (%)	**−7.0**	**−6.3**	**−1.8**	**4.7**	**3.7**	**2.9**	**2.1**	**3.2**	**4.4**	**6.5**	**0.7**	**3.9**
III	Ref.	1.000			0.800			0.800			1.000		
	ESI	**0.925**	**1.020**	**0.915**	**0.852**	**0.813**	**0.854**	**0.793**	**0.838**	**0.784**	**1.053**	**0.965**	**1.068**
	Err (%)	**−7.5**	**2.0**	**−8.5**	**6.5**	**1.6**	**6.8**	**−0.9**	**4.8**	**−2.0**	**5.3**	**−3.5**	**6.8**
IV	Ref.	0.900			1.000			1.000			0.800		
	ESI	**0.831**	**0.828**	**0.861**	**1.064**	**1.066**	**1.047**	**0.988**	**0.998**	**1.032**	**0.883**	**0.848**	**0.836**
	Err (%)	**−7.7**	**−8.0**	**−4.3**	**6.4**	**6.6**	**4.7**	**−1.2**	**−0.2**	**3.2**	**10.4**	**6.0**	**4.5**
V	Ref.	0.950			1.000			0.900			0.850		
	ESI	**0.931**	**0.915**	**0.950**	**1.039**	**1.021**	**1.038**	**0.922**	**0.910**	**0.937**	**0.898**	**0.861**	**0.871**
	Err (%)	**−2.0**	**−3.7**	**0.0**	**3.9**	**2.1**	**3.8**	**2.4**	**1.1**	**4.1**	**5.7**	**1.3**	**2.5**

Observations, however, reveal that the damage identification algorithm provides good estimation result, even though the detection accuracy depends on the noise level of the measurement data. An interesting result is that the vibration data obtained under different vehicle speed compared with the speed used in updating baseline model is not greatly affect the accuracy of the damage identification. Further work is needed to consider uncertainties in a real situation because there are many sources of uncertainties such as random noise in experimental data, temperature fluctuations, vehicle properties, modeling uncertainties, and others to make the procedure practically applicable.

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Impact acoustics of concrete structures by applying discrete wavelet transform

Y. Nomura
Graduate School of Science and Technology, Kobe University, Japan

M. Kawatani
Department of Civil Engineering, Kobe University, Japan

M. Hirokane
Faculty of Informatics, Kansai University, Japan

M. Kou
Department of Civil Engineering, Kobe University, Japan

ABSTRACT: This study attempts to develop diagnostic method that is capable of quantitative evaluation on detecting interfacial separation or defects inside the concrete. In this study, Wavelet Transform is applied to analyze sound signals from concrete structures. Discrete Wavelet Transform is applied to detect a possibility of existence of interfacial separations and also to measure the volumes from variations of Wavelet coefficients. Finally, the efficiency and applicability of proposed method is discussed through the several results of examinations.

1 STRUCTURAL MODEL AND ARTIFICIAL DEFECT MODELS

This study employs two types of concrete-steel composite slab as structural models. One is the steel plate rib form; the other is the truss style reinforcement form. The artificial interfacial separation between the steel and the concrete is simulated by styrene foam and rubber sheet. The styrene foam is set to simulate the situation that the steel-concrete interface is invaded by air. Also the rubber sheet simulates the situation that the water intervenes inside the slab. These artificial defect models are embedded in the same depth of steel-concrete interface before casting the concrete.

2 IMPACT ACOUSTIC ANALYSIS BY APPLYING WAVELET TRANSFORM

The analytical results by applying DWT are given in Figures 1(a), (b), (c) and (d). These results are obtained from the truss style reinforcement form. And also, these figures show the sound signals and the results of wavelet coefficients obtained from the first order to the third order by applying DWT. Here, high frequency component at around 12.8 kHz are detected along a time in the decomposition level of first order as shown in C_1. In the decomposition level of the second order C_2 the frequency component at around 6.8 kHz is detected. And also, low frequency component at about 3.4 kHz is detected in the decomposition level of the third order C_3.

Figure 1 (a) shows the results obtained from the intact cases of truss style reinforcement form. Wavelet coefficients obtained from intact part of steel plate rib form shown in Fig.1(a) are found to be scattered in each decomposition levels. In other words, wavelet coefficients in all decomposition levels of the first, the second and the third orders show the rapidly decreasing. Figure 1 (b) indicates that the analysis results when embedding the large styrene foam with the size of 200 mm × 200 mm × 5 mm. In this case, it is found that the wavelet coefficients of the decomposition levels of the second and the third orders show the larger trend in comparison with that of the decomposition level of the first order. As the reason, it is considered that the frequency of

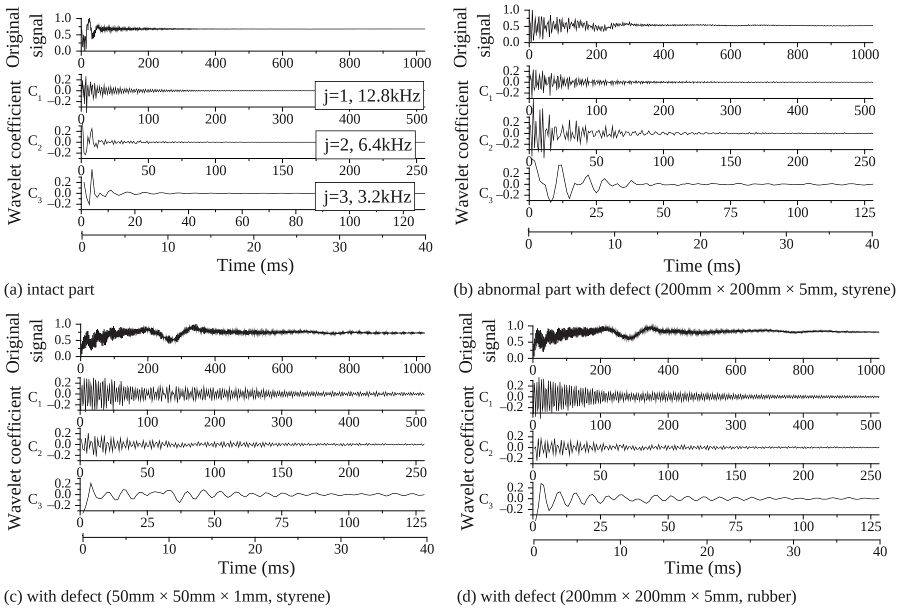

Figure 1. Analysis results of the intact part and the part with defect of the truss style reinforcement form.

P-wave falls off by the influence of the presence of artificial defects. Figures 1(c) and (d) show the analytical results when embedding the small styrene foam with the size of 50 mm × 50 mm × 1 mm and the rubber sheet with the size of 200 mm × 200 mm × 1 mm as the artificial defect models. Both cases indicate the same trend that wavelet coefficient of the decomposition level of the first order is larger than that of the decomposition levels of the second order and the third order. And also, when embedding the rubber sheets as artificial defect models, it was confirmed irrespective of the size that the wavelet coefficient of decomposition level of the first order is the largest than that of the other decomposition levels. From above results, it is understood that the detecting the differences between the normal parts and the abnormal parts having the interfacial separation inside composite slabs is possible by observing the changes of the wavelet coefficients.

3 CONCLUSION

This study improved the impact acoustics by introducing DWT and confirmed the filling situation of the steel-concrete composite slabs for the purpose of the improvement of the accuracy of the daily inspection before applying advanced non-destructive methods. The sound signals obtained from normal and abnormal parts are analyzed by using DWT. As the results, DWT is found to be effective as the quantitative evaluation method. The conclusions are summarized as follows:

- In intact part of composite slabs, it was found that the components of frequency are scattered in the decomposition levels of each order. And also, wavelet coefficients in all decomposition levels of first, second and third orders show the rapidly decreasing.
- When embedding the large defect models into composite slabs, it was confirmed that the components of frequency at about 12.8 kHz fall off more rapidly than that of the other decomposition levels. In other words, the energy of low frequency at about 6.8 kHz is larger than that of high frequency band.
- When embedding the small defect models into composite slabs, the energy of frequency at about 12.8 kHz is larger than that of low frequency at about 6.8 kHz or 3.4 kHz.
- When employing the rubber sheets as defect models, the energy of frequency at about 12.8 kHz is the largest among of the other low frequency band irrespective of the size of rubber sheet and the type of composite slabs.

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Predictive SHM-supported deterioration modelling of reinforced concrete bridges

M. Imran Rafiq, Marios K Chryssanthopoulos & Toula Onoufriou
School of Engineering, University of Surrey, Guildford, Surrey, UK

EXTENDED ABSTRACT: Deterioration, increase in loading demand and change in utilization have induced an unknown level of risk in the use of transport infrastructure systems. Bridges being a vital element of such systems, due to their very nature as well as their exposure to harsh environmental conditions, should be effectively managed for the benefit of the overall transport network. Predicting the future condition and reliability of the bridges is vitally important in this process. Probabilistic models have been developed to estimate and predict the extent of deterioration in, for example, concrete bridges. However, the input parameters of these models are fraught with uncertainties, thus severely limiting their accuracy, particularly over longer time frames. On the other hand, continuous innovations in the sensing and measurement technology have lead to the development of monitoring instruments that can provide continuous (or almost continuous) data regarding the actual structural performance in the time frame. This information cannot be used directly for the prediction of future performance, first because it typically pertains to a small number of specific locations, and secondly because it needs to be combined with a whole host of other knowledge components. Furthermore, uncertainties in the instruments/measurements and in the future behaviour of the structure and its interaction with the environment (e.g. including the effects of deterioration) also hinder the predictive capability of current modelling tools.

The potential benefits of improving performance prediction through the integration of health monitoring systems with probabilistic predictive models, and their implications on the management of deterioration prone structures are presented in this paper through the development of an integrated methodology. It is shown, through application case studies, that the confidence in predicted performance can be significantly increased through the use of SHM-supported modelling of deterioration and the major inspection and maintenance activities can be delayed on the account of increased confidence in the predicted performance. An example of such integration is illustrated in Figure 1 for various cases of sensor outputs including attainment of limiting value as well as

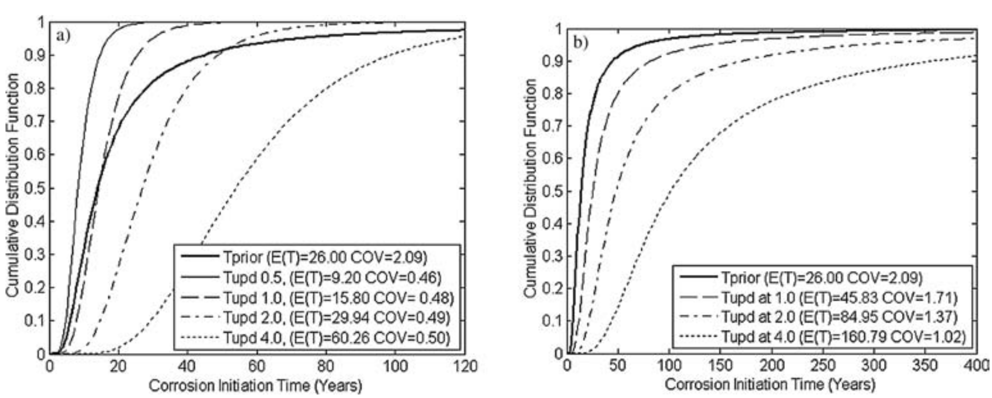

Figure 1. Posterior corrosion initiation time at rebar level. (a) Initiation confirmation (b) Passivity confirmation.

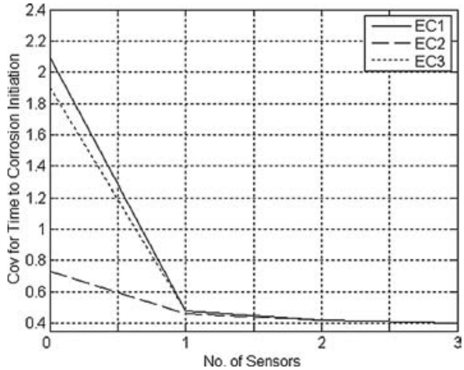

Figure 2. Effects of no. of sensors on uncertainty associated with exposure conditions.

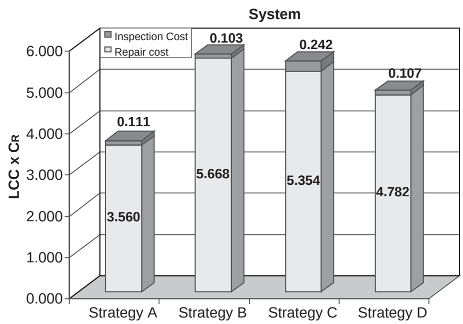

Figure 3. Life cycle cost comparison for the members and the overall system.

confirmation of safety at various points in time during the service life. It is clear that the uncertainty is reduced with the availability of additional information and the level of this reduction depends on the quality and timing of information obtained through sensing equipment.

A sensitivity study of various input parameters has concluded that the range of predicted performance is considerably reduced through the updating methodology presented in this paper. A typical result is shown in Figure 2, which quantifies the influence of the number of sensors on the coefficient of variation for the time to corrosion initiation at rebar revel for various hypothesized exposure conditions. The case with ‘0’ sensor indicates the prior corrosion initiation times. It can be seen that the influence of various models that could be assumed for exposure conditions is minimized by the integration of data obtained through SHM into the predictive models.

Finally a life-cycle cost analysis for various management strategies (with and without the use of SHM) highlighted the safety and cost benefits that can be obtained through the use of SHM-supported predictive models (Figure 3). It is clear from the figure that the LCC is minimized for the case where decisions are aided with predictive models updated through SHM.

It is recognized that the above conclusions are obtained from a limited number of application case studies. Clearly more work is needed in this area including physical tests and field data collection to improve our understanding of the underlying phenomena and to reduce prior uncertainties, especially those related with modelling and measurement (epistemic components).

*Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Development of BMS for a large number of bridges

M. Kaneuji
Civil Engineering Management Division, Kajima Corporation, Tokyo, Japan

H. Asari
Road Management Division, Aomori Prefectural Government, Japan

Y. Takahashi & Y. Ohtani
Civil Engineering Design Division, Kajima Corporation, Tokyo, Japan

H. Ukon
Information Technology Division, Kajima Corporation, Tokyo, Japan

K. Kobayashi
Kyoto University, Kyoto, Japan

1 INTRODUCTION

Bridge Management System has to deal with individual deterioration phenomena, deterioration forecast and effective measures for each deteriorations, which is called “Micro Management”, and at the same time has to deal with long term budget planning to maintain all bridges, which is called “Macro Management”. Since these two managements are quite different in their nature, it has been believed that these two managements should be treated differently, even though they are closely related. The authors have developed a new BMS, which treats both Micro Management and Macro Management in a closely related manner by introducing a “Maintenance Scenario” concept.

2 EQUALIZATION OF LCC

The bridge administrator is usually required to equalize the tax burden of the different generations. Therefore, it is necessary to equalize the annual costs of LCC through a long period such as 50 years. It is not difficult to propose a maintenance plan with the least LCC for individual bridge. But, when you collect all the maintenance plans and sum up their LCC, it is usual to get a non-uniform LCC. If you can get necessary budget each year, the collection of the least LCC plans for each bridge is the best maintenance plan. But, if you are required to submit a maintenance plan with equal budget each year, you need to choose alternative maintenance plans for some bridges. This paper is to introduce a new concept of BMS, which treats Micro Management and Macro Management in closely related manner based on the same database with the introduction of “Maintenance Scenario” concept.

3 MAINTENANCE SCENARIO

The bridge administrators are usually required to equalize annual budget in order to equalize the tax burden of the different generations. When the distribution of the annual maintenance costs over the lifecycle years is not uniform after the summation of LCC of the best maintenance plans for each bridge, some of the bridge maintenance plans need to be changed to have uniform LCC. In order to select bridges those original maintenance plans should be changed to obtain a uniform and the least LCC for the entire bridges, it is necessary to know the LCC of alternative maintenance plans for those bridges. Therefore, the authors have introduced a concept of predetermined Maintenance Scenarios to make the process of equalization and minimization of LCC easier.

(A) Preventive Maintenance Scenario
(B) Early Corrective Maintenance Scenario
(C) Essential Maintenance Scenario
(D) Replacement Scenario.

4 PRIMARY SELECTION OF THE MAINTENANCE SCENARIO

The first step of the primary selection of the maintenance scenarios is the selection of the bridges to be replaced. The second step is to select one or several scenarios from Preventive Maintenance, Early Corrective Maintenance or Essential Maintenance Scenarios for the rest of the bridges, taking various factors, such as condition states, environmental situations, role of the bridges in the road network into account.

5 THE LONG-TERM BUDGETING PLAN

When the LCC is not evenly distributed but concentrated in a particular period, the peak of LCC should be cut off to get a uniform budget plan. The peak cut is obtained by changing the maintenance scenarios of bridges those LCC are concentrated in that peak period (Figure 1).

5.1 *Multi level budgeting*

It is usual to select the maintenance scenarios with the least LCC as the best plan. Therefore, every time you change the maintenance scenario to obtain a uniform budget, you will find the increase in LCC (Figure 2). But, by having a budget plan with multi levels by getting an extra bonus for a particular period as shown in the Figure 3 instead of a uniform budget, you may be able to have less LCC.

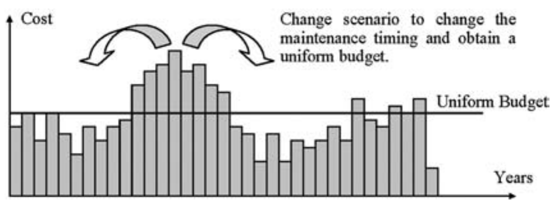

Figure 1. Changing maintenance scenario to obtain a uniform budget.

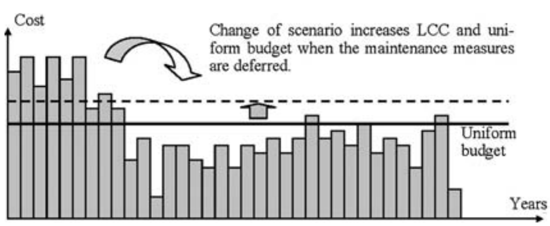

Figure 2. Change of scenario increases uniform budget.

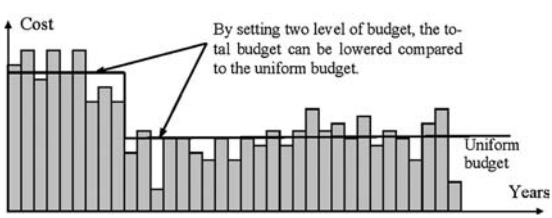

Figure 3. Budget increase is lowered with multi level budgeting.

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Cruz, Frangopol & Neves (eds)

Implementation of bridge management system in Aomori prefectural government, Japan

N. Yamamoto, H. Asari & T. Ishizawa
Road Management Division, Aomori Prefectural government, Japan

M. Kaneuji
Civil Engineering Management Division, Kajima Corporation, Tokyo, Japan

E. Watanabe
Kyoto Univ. & Chairperson, Regional Planning Institute of Osaka, Osaka, Japan

ABSTRACT: Aomori Prefectural Government (Aomori) is located at the northern end of the main island in Japan, and many of the bridges have been deteriorated due to the severe winter weather.

Although the average age of Aomori bridges is around 30's, there are many deteriorated bridges, and it is estimated that the maintenance, repair and rehabilitation cost for those bridges will rise tremendously high in the future. Therefore, Aomori has decided to carry out Bridge Management System (BMS) in Aomori comprising five steps with the aim of reducing such costs in the future.

1 INTRODUCTION

Aomori is located at the northern end of the main island in Japan, and many of the bridges have been deteriorated due to the severe winter weather.

Although the average age of Aomori bridges is around 30's, there are many deteriorated bridges, and it is estimated that the maintenance, repair and rehabilitation cost for those bridges will rise tremendously high in the future. Therefore, Aomori has decided to carry out BMS in order to reduce such costs in the future.

2 AN OVERVIEW OF ABMS

The Aomori's BMS(ABMS) comprises five steps shown in Figure 1.

2.1 *STEP1: Basic Strategy*

We set up a long-term strategy based on the basic policy, and set up the goals for Bridge Condition and States and Budget.

2.2 *STEP2: Individual Strategy*

We set up an Individual Bridge Strategy considering an environmental situation, inspection results and an evaluation of importance and utilized IT to the inspection system and made our own inspection manual for Maintenance Management and Inspection System. As a result, we reduced the cost of inspection by 80%. Also, we established the Maintenance Scenario divided into the Elongated Scenario and the Replacement Scenario. Condition states forecast is established in accordance with environmental conditions in addition to that being conducted according to elements, type of material, deterioration mechanism and specifications. We established deterioration speed and

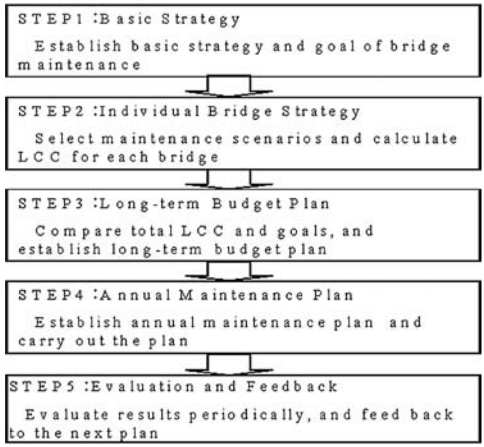

Figure 1. Procedures of ABMS.

confirmed deterioration speeds due to academic experts' knowledge and experiences. Also, in order to calculate LCC, we established "Maintenance Measures Matrix".

2.3 *STEP3: Long-term Budget Plan*

We sum up LCC of all bridges, try to equalize the annual expense, and establish the Long-term Budget Plan. We conducted inspections of all bridges 15 m or longer in total length in both 2004 and 2005 simulated through ABMS based on the inspection results.

2.4 *STEP4: Annual Maintenance Plan*

Based on the results of Step3, we establish an Annual Maintenance Plan regarding what kind of maintenance measures should be taken, on which parts, of which bridge, and when, and execute the plan.

2.5 *STEP5: Evaluation and feedback*

We review the results and feed them back to management system for the purpose of increasing level for more efficient maintenance and management in terms of turning the management cycle around. Maintenance and management is respectively conducted according to each frequency and contents.

3 AN APPROACH FOR POST-INTRODUCTION OF ABMS

We have been conducting education programs for our young in-house engineers who are supposed to directly operate BMS in preparation of an introduction of BMS and have been tacking improving engineering skill of engineers of construction companies in Aomori. Furthermore, we are carrying out joint research projects between college researchers, engineers from private sectors and Aomori in order to improve precision in condition states forecast of BMS which reflects on the results of research on geographical characteristics regarding deterioration and damage of bridges in Aomori.

4 CONCLUSIONS

We have just started with ABMS and have several outstanding issues such as an improvement in precision of condition states forecast. Therefore, we are planning to make continuous and persistent efforts to tackle maintenance and management of bridges through BMS for improving precision of system. Also, we expect our approach will be disseminated to other local governments in Japan and will contribute to improving bridge services across Japan.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Condition evaluation standards and deterioration prediction for BMS

E. Matsumura
New Structural Engineering LTD, Tokyo, Japan

Y. Senoh
Yokogawa Construction LTD. Tokyo, Japan

M. Sato
PS MitsubishiConstruction CO., LTD, Tokyo, Japan

Y. Miyahara
Kawaguchi Metal Industries CO., LTD, Tokyo, Japan

M. Kaneuji
Civil Engineering Management Division, Kajima Corporation, Tokyo, Japan

Masahiro Sakano
Kansai University, Osaka, Japan

1 INTRODUCTION

In order to establish Bridge Management System, the calculation of Life Cycle Cost (LCC) and deterioration prediction are vital, but these engineering technologies have not been established to the level of practical use.

2 OUTLINE OF THE BRIDGE MANAGEMENT SYSTEM

The main procedures of the Bridge Management are divided into 5 steps.

STEP1: Basic Strategy: Establish basic strategy regarding bridge maintenance and set up targets both on Performance of bridges and Budget

STEP2:Individual Bridge Strategy: Based on inspection data, select maintenance scenarios for each bridge and calculate LCC based on deterioration forecast

STEP3: Long-term Budget Plan: Establish long-term budget plan based on LCC simulation and determine the combination of maintenance scenarios.

STEP4: Annual Maintenance Plan: Establish annual maintenance plan based on the budget plan and scenarios.

STEP5: Evaluation and Feedback: Evaluate management result periodically and feed them back to the next planning.

3 CONDITION EVALUATION STANDARDS FOR BMS

The main concept of the Condition Evaluation Standard with five stages are as follows:
[Condition State 5]
The deterioration has not started yet, or has not revealed on surface if it has started under the surface. This condition state corresponds to the “Latent Period ” in deteriorating process.

The maintenance measures taken at this stage are usually called Preventive Maintenance.

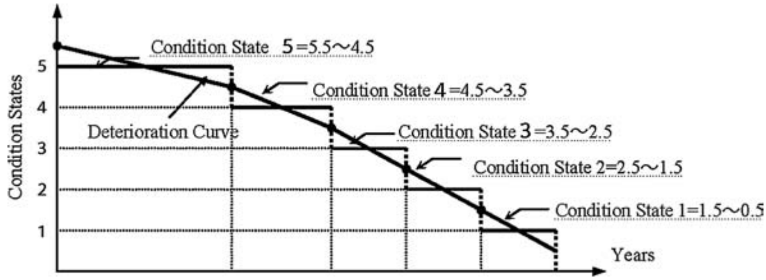

Figure 1. Deterioration Curves.

Table 1. Condition Evaluation Standard for reinforced concrete member suffered from Salt Damage.

Condition State	Definition	Observed State of Condition	Example of Measures
5 (Latent Period)	The period until the chloride concentration at the point of reinforcing bars reaches to the limit value to start corrosion.	No change is observed. (Chloride concentration is smaller than the limit value to start corrosion)	Surface treatment.
4 (Progression Period)	The period that the reinforcing bars start to corrode and the cracking through the cover concrete reaches to the surface.	No change is observed. (Chloride concentration is larger than the limit value and the reinforcing bars start to corrode)	Surface treatment, Cathodic protection, Electro chemical desalination.
3 (Former Acceleration Period)	The period that the cracking is formed through the cover concrete and the speed of corrosion increases.	Cracking due to the corrosion is observed and the cover concrete partially spalls off.	Surface treatment, Surface repair, Cathodic protection, Electro chemical desalination
2 (Latter Acceleration Period)		Many cracks due to the corrosion are observed. The lime exudation or the rust exudation is observed from cracks. Delamination or spalling is observed. The amount of corrosion is large.	Surface repair with reinforcement.
1 (Deterioration Period)	The period that the structural strength of the concrete member severely degrades due to the increase of corrosion.	The width of cracking is large and the large amount of rust exudation is observed. The large delamination or spalling is observed.	FRP reinforcement, Surface repair, Outer-cable reinforcement, Carbon or aramid reinforcement, RC reinforcement, Superstructure replacement.

[Condition State 4]
The deterioration has started and continues to progress. The deterioration may be revealed on the surface. Even if the deterioration is not revealed on the surface, it can be detected by nondestructive methods. This condition state corresponds to the "Progressing Period".
[Condition State 3]
The progress of deteriorations starts to accelerate at this stage.
This condition state corresponds to "Front stage of accelerating period"
[Condition State 2]
The deterioration continues to progress.
This condition state corresponds to "Latter stage of accelerating period".

[Condition State 1]
The deterioration proceeded quite badly to a point where the performance of the structure is no longer secured. This condition state corresponds to “Deteriorating period”.

The structural safety is sometimes in danger. In such cases, emergency measures are necessary. It is often difficult to fully recover their health condition of elements at this condition state by maintenance measures as repairs or rehabilitations. In such cases, replacement of elements or components becomes necessary.

4 DETERIORATION CURVES

The deterioration curve consisted from five linear lines is formed by estimating the length of each Condition State periods in years.

5 THE CONDITION EVALUATION STANDARD FOR CONCRETE MEMBERS

Typical deterioration phenomena of concrete members of bridges are as follows:

(a) Salt damage
(b) Carbonation
(c) Frost damage
(d) Alkali aggregate reaction
(e) Fatigue deterioration of concrete deck

The Table 1 is an example of Condition Evaluation Standard for reinforced concrete member suffered from Salt Damage.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Health monitoring of steel bridges using local vibration excitation

S. Beskhyroun, T. Oshima & S. Mikami
Kitami Institute of Technology, Kitami, Hokkaido, Japan

1 INTRODUCTION

Most vibration-based damage detection theories and practices are formulated based on the assumption that failure or deterioration would primarily affect the stiffness and therefore affect the modal characteristics of the dynamic response of the structure. If this kind of changes can be detected and classified, this measure can be further implemented for a bridge monitoring system to indicate the condition, or damage, or remaining capacity of the structures. It can also be used to evaluate the seismic behavior of the structures. However, conventionally defined modal parameters have been shown to be mildly sensitive in the detection of various types of bridge damages. Furthermore, the modal parameters of conventional modal testing such as frequencies and modal damping are global parameters, which cannot locate the damages. Many damage detection schemes rely on analyzing response measurements from sensors placed on the structure. Research efforts have been made to detect structural damage directly from dynamic response measurements in the time domain, e.g. the random decrement technique, or from frequency response functions (FRF). Also, some damage detection methods have been proposed to detect damage using system identification techniques. In this paper, an algorithm based on changes in Power Spectral Density (PSD) is presented. The algorithm is used to detect damage, locate its position and monitor the increase in damage using only the measured data without the need for any modal identification or numerical models. The method is applied to the experimental data extracted from a steel bridge after inducing some defects to its members. The release of some bolts from one stiffener located on the web of the main girder of the bridge introduced the damage to the bridge. A future goal of a comprehensive bridge management system is to have a self-monitoring bridge where sensors feed measured responses (accelerations, strains, etc.) into a local computer. This computer would in turn apply a damage identification algorithm to this data to determine if the bridge has significantly deteriorated to the point where user safety maybe jeopardized. In such case, the dynamic properties of the structure have to be identified from ambient, traffic-induced vibration. The damage identification method presented in this study requires the excitation forces used for the undamaged and damaged structures to have the same waveform, amplitude and location. Therefore, ambient vibration cannot be used as excitation source in this case. In order to find an excitation source that can be used for continuous health monitoring of the bridges (without interrupting the traffic) and at the same time can provide equal forces, the implementation of piezoelectric actuators as a local excitation source for large structural elements is presented. The advantages of using piezoelectric actuators than using shakers, hammers or ambient vibrations are discussed.

2 RAILWAY STEEL BRIDGE: DESCRIPTION AND EXPERIMENTAL SETUP

The experimental work in this research was performed on a railway steel bridge that is no longer in service. The bridge is removed from its service location several years ago. A man, who is interested in collecting old trains, bought the bridge and kept it in his own land. The bridge is supported now on two wooden blocks. The owner of the bridge has granted us a permission to introduce some damage to the bridge such as releasing some bolts and tightening them again. However, introducing torch cuts to the bridge was not permitted. The bridge consists of two steel plate girders and two steel stringers support the train rails. Loads from the stringers are transferred to the plate girders by floor beams located at various intervals. The multi-layer piezoelectric actuator is used for local excitation. The actuator force amplitude is 200 N. Although this force amplitude is very small compared to shaker forces or ambient vibrations, it was enough to excite the web of the main girder until the position of the farthest accelerometer. Two actuators are used for exciting the web of the main girder in the horizontal direction. The actuators are located at the upper part on the web of the main girder. The excitation forces used for the undamaged and damaged structure are random, equal in amplitude and have the same vibration waveform but the excitation force does not need to be measured. The main advantages of using piezoelectric actuators than using conventional excitation methods such as dynamic shakers, or ambient vibration can be summarized as follows:

– Actuator size is very small and can be handled easily. Moreover, it can be fixed to any structural element and remotely operated for continuous health monitoring of the structure.
– The traffic over the bridge needs not to be interrupted as the case of using dynamic shakers.
– The main advantage of using piezoelectric actuator is that it produces vibration with different frequencies ranging from 0.1 to 400 Hz that is effective in exciting different mode shapes.
– Large number of vibration data can be saved in a short time as the sampling rate in case of using actuator can reach 2 kHz.
– The same excitation force (equal magnitude and the same waveform) can be produced for exciting the undamaged and damaged structure, which is needed for applying damage identification technique studied in this paper.
– Undesired vibrations induced from wind, traffic or any other source can be avoided since the vibration data induced from the actuators can be generated at any desired time.

Eight accelerometers were used to measure the acceleration response in the horizontal direction. One accelerometer is mounted at the geometrical center of gravity of each panel of the main girder. All of the connections of different elements of the bridge are riveted and no damage could be introduced to these connections. Only two angles (look like stiffeners) are bolted to the web of main girder. Therefore, it was decided to remove the bolts one by one from the right angle to introduce damage to the main girder.

3 RESULTS AND CONCLUSIONS

Changes in the PSD magnitude due to the presence of structural damage have been investigated. The experimental results obtained from a steel bridge demonstrate the usefulness of the changes in PSD magnitude as a diagnostic parameter in detecting the damage, locating its position and monitoring the increase in damage. The main advantages of the proposed methods are:

– PSD is calculated from the acceleration response at every channel without measuring the excitation force. However, the excitation forces used for the undamaged and damaged structure have to be random, equal in amplitude and have the same waveform.
– The proposed method encompasses the first three steps of the process of damage detection – existence, localization and monitoring the damage increase being based on only the measured data without the need for any modal identification or numerical models.
– Vibration based damage identification methods sometimes produce false positive readings due to measurement errors, noise and environmental changes. The proposed method has shown better results in identifying the changes in PSD associated with damage from the changes attributed to noise or measurement errors.

In this study, piezoelectric actuators were used as a local excitation source for large structures such as steel bridges. The advantages of using piezoelectric actuators for local excitation than using conventional excitation methods such as dynamic shakers, hammers, or ambient vibration have been discussed.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

A system for field inspection of infrastructure in snowy cold regions using speech recognition

A. Kenmotsu & H. Ushio
NTT Comware Corp., Research and Development Dept., Tokyo, Japan

H. Tsugimura & T. Oyama
Structures Dept., Docon Co., Ltd., Hokkaido, Japan

ABSTRACT: Bridge inspectors make handwritten records during field inspection. The major drawback of handwriting is that it exposes the inspectors to danger when they are making inspections in high places. In cold, snowy regions, sometimes they also have to work under slippery, freezing conditions. The subsequent input of handwritten data into computers is complex and laborious, making it prone to human error. To address these issues, we developed the Field Inspection Recording System (FIRSt). The system compiles audio data and image data to produce bridge inspection reports, thereby eliminating the need for handwriting on paper and reducing the risk that inspectors will meet with accidents. In addition, converting the audio data directly into text data also avoids transcription errors and reduces the workload. The system affords improved safety, accuracy and efficiency of inspection. It has been put to use in bridge inspections.

1 INTRODUCTION

There are many bridges in Japan that are nearing their fifth decade in service. The maintenance/management of such old bridges is becoming a problem. Accurate assessment and precise understanding of current bridge conditions are important, and periodic inspection is particularly important for the maintenance and management of these bridges. Toward taking advantage of onsite data input while securing inspection safety and accuracy, we developed the Field Inspection Recording System (FIRSt).

2 BACKGROUND

Major issues in bridge inspection are as follows: 1) Onsite note-taking for record-keeping is influenced by weather and temperature, 2) Note-taking at high places during inspections can be dangerous, 3) Transfer of data from handwritten inspection records to computer is troublesome and error prone; and 4) Measures against transcription errors are time consuming.

To address these issues, a system that achieves the following has been required: 1) onsite safety during inspection, 2) minimized onsite workload, 3) improved efficiency, particularly that of indoor work such as the drafting of reports, to reduce the overall cost of inspection, and 4) production of final reports that accurately reflect the inspection results.

3 ESTABLISHMENT AND OPERATION OF FIRST

3.1 *System configuration*

The main functions of the FIRSt are the capture of inspection results (speech and photos), speech recognition, and report processing. The system holds down costs by adopting a commercially

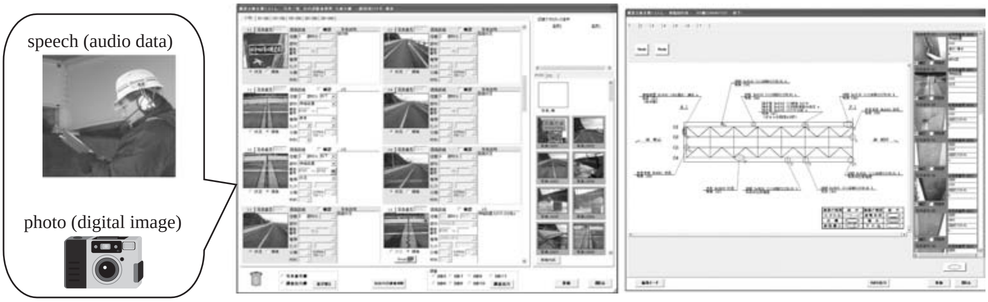

Figure 1. Onscreen image from the system.

Table 1. Items in the speech dictionary.

Member	Member No.	Damage	Damage level
Main girder	0101	Corrosion	a
Cross beam	0102	Cracking	b

available IC recorder. The microphone has noise-canceling capabilities that are useful against onsite wind noise. Hands-free recording fosters inspection safety.

Speech recognition is performed after inspection, using a special speech recognition dictionary that incorporates a glossary of inspection terms that include types of members and damage. An onscreen image from the system is shown in Figure 1. The text converted from the speech appears onscreen in the appropriate field. When the inspector speaks the photo numbers during inspection, speech-recognition text data on damage assessment results are automatically displayed with the photos.

Confirming the inspection results is simplified, because the speech can be replayed. Errors in input, or in linking the members and damages, are marked in color to attract attention. This reduces human error and simplifies confirmation of results.

3.2 *Improving the speech recognition rate*

Several methods were devised to improve the speech recognition rate. First, clear rules for use of words were established to avoid unexpected voice inputs. For example, a predecided set of terms such as “first span, deck, 0203, deck cracking, rank C, photo-2” in a consistent way. Furthermore, multiple dictionaries are used for greater accuracy of recognition. The dictionary creation function allows customized dictionaries to be produced for individual inspectors. These improvements have raised the speech recognition rate to a level (>95%) that is quite suitable for practical use.

4 CONCLUSION

Using this system, about 370 and 800 bridges were inspected in 2004 and 2005, respectively. The goals of safety improvement, accuracy achievement and inspection cost reduction were realized. The voice recording during inspections also provides proof that inspection has been performed. The benefits in cost reduction differed from bridge to bridge, but without changing the time required for inspection, the report processing time was cut to less than 1/3 of that required for the handwriting-based system. The overall cost of inspection can be almost halved.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Integrating bridge health monitoring into bridge management

R. Kiviluoma
WSP Consulting KORTES Ltd, Helsinki, Finland

ABSTRACT: While the usefulness in everyday bridge engineering could be still questioned, bridge health monitoring systems undergo rapid technological development, and in the near future these seem to be ready for low-cost implementations applicable for conventional small bridges. This rises several topical questions, like: how to integrate bridge heath monitoring into the bridge management systems; how to utilise data in improvement of theoretical deterioration models; how to use data in evaluation by bridge engineers; and how standards and legislation may be updated to better take into account benefits of structural monitoring systems. The present paper discusses monitoring technology in perspective of European Commission's large research project "Sustainable Bridges" and integration into bridge management in plans for new procurement methods of road bridge maintenance in Finland.

1 AUTOMATION TECHNOLOGY IN MONITORING SYSTEMS

Automation technology is implemented by small-sized, commercially well available, and standard electrical modules, from which the complete structural monitoring system could be formed. Bottleneck has been the speed of measurements, limited around 10…100 Hz due to both network traffic and electronics. This speed may be not enough for multi channel dynamic (vibration) measurements. In last two years, this problem is removed by Field-Programmable Gate Array (FPGA) technology. A prototype of such monitoring system (Fig. 1) is to be developed and demonstrated by the author in the Sustainable Bridges project.

2 INTEGRATION INTO BRIDGE MANAGEMENT SYSTEMS

Integration of health monitoring into the bridge management systems serves the target of developing the technology towards needs of bridge owners. Optimal allocation of funds to maintenance is

Figure 1. Prototype of structural monitoring system adopting automation technology.

an issue that interests all owners, and bridge management system is the tool for that when assessing owner's bridge stock as whole. Rising question is then could the structural monitoring improve bridge management systems in extend that the integration is reasonable. After initial considerations, the answer is found positive, and this idea is to be further studied in the pilot project for new procurement concept for road bridge maintenance, which takes place in Oulu district, Finland. In the pilot project, bridge management and maintenance of large regional bridge group is outsourced for private consortium for five years. New technologies are studied in respect their potential to bring improved and objective input for decision making.

3 CONCLUSIONS

- Automation technology has recently result in low cost bridge health monitoring systems, which address new possibilities into the bridge management, life-cycle engineering and safety
- technical development of monitoring systems alone is not enough to guarantee successful results and improvements. Thorough cooperation between all parties is needed, including time consuming work to update design standards and legislation. Such cooperation is started, e.g., in the large EC research project Sustainable Bridges
- to give desired results for the owner, bridge engineers should be aware and start exploiting structural monitoring. Beside this, structural monitoring continues to be one of the most valuable modern tool for theoretical research workers
- promising way for storage of bridge monitoring data is put it into centralised asset management database, aside with inspection and inventory data. To this data engineers, contractors, owners and researchers have different views, and analysis may be either instant or delay years from the acquisition phase. Data needs to be correct, well documented and easily transferable to future systems
- integration of bridge health monitoring into the bridge management is characterised by four major steps: 1) technical improvement of monitoring systems, 2) careful plan what to monitor 3) computer engineering in data storage and bridge management software and 4) engineering judgement to use results in form which benefit bridge owner. Obviously, steps 1) and 3) could be satisfactorily solved in few years, while the rest needs experiences, engineering judgement, and research over longer maturing period.

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Innovative developments towards improving bridge seismic safety

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Bayesian updating of bridge fragility curves using sensor data

J.-M. Wong, K. Mackie & B. Stojadinovic
University of California, Berkeley, California, USA

ABSTRACT: Assessment of direct and indirect losses to an urban region due to an earthquake event requires modelling many aspects of the buildings and infrastructure in the region, including a transportation network. Specifically, the propagation of hazard to different spatial locations, the vulnerability of different transportation network components, network traffic behaviour, and the economic losses due to the damages incurred to the network need to be modelled. A critical aspect of such large-scale simulations is the accurate determination of the vulnerability of individual bridges, as these are critical to maintaining flows of goods, people, and services. The vulnerability of bridges can be categorized either in terms of physical damage to the structures, or as estimates of the repair cost required to return the bridge to a previous level of functionality.

There are numerous uncertainties associated with the prediction of bridge vulnerability. Many sources of uncertainty are inherent to the processes being considered and are therefore irreducible. For example, the response of individual bridge structures to different earthquakes is uncertain. However, there are also epistemic uncertainties associated with each model in the transportation network simulation. Such uncertainties can be reduced through more refined models and a better understanding of the constituent processes. For example, the relationships between structural response and earthquake intensity are usually based on nonlinear finite analyses of finite element models of structures. Structural responses obtained from such analyses are sensitive to the choice of model elements, materials, finite element platform, and mathematical theory of the structural response of different components. Therefore, a powerful addition to such simulations is the ability to update estimates of network functionality based on sensor data obtained from instrumented bridges in the network. Specifically, sensor data for instrumented structures in the forms of displacements and accelerations can be utilized to update the distribution of structural response prediction for the earthquake intensity experienced at the site. This is especially constructive when generic bridge fragility curves are used for families of similar bridges, as is commonly done when it is not feasible to develop bridge-specific fragility curves for hundreds of individual bridges in a region.

A Bayesian method is presented for combining measured sensor data and response estimates from analytical models to obtain better estimates of bridge seismic performance and, thus, improve post-earthquake damage assessment of the highway network. Seismic performance assessment is conducted using the PEER probabilistic performance-based earthquake engineering framework. Prior distributions of bridge seismic response measures are obtained using analytical bridge demand models. Empirical damage measures are obtained from a database of experimental results from cyclic column tests. Decision variables are obtained from available repair cost reconnaissance data. Measured data obtained from sensors on an instrumented bridge are used to produce likelihood functions for parameters representing the actual state of the structure postearthquake. A schematic of how the sensor data affects the components of the analysis framework is given in Figure 1. The updated distributions are then used within the PEER framework to obtain a better estimate of the performance of other similar bridges. Such improved estimates can be used, for example, to re-prioritize seismic upgrade work on bridges in a traffic network to increase the safety and reliability of a regional transportation system. Alternatively, the estimates can be used to more accurately determine emergency response strategies post-earthquake.

An example of this method is demonstrated using numerical simulations of a typical reinforced concrete highway overpass bridge in California. The demand model was developed for

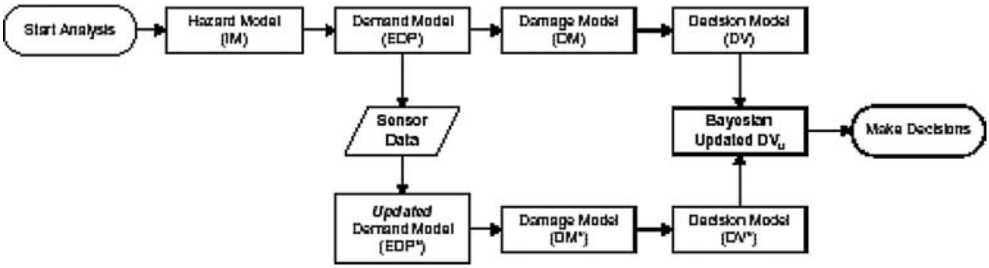

Figure 1. Updating of the demand model and decision variable in PEER PBEE framework using data acquired by sensors during an earthquake event.

Table 1. Changes in bridge column damage and bridge repair cost ratio probabilities caused by updating the analytical models using data measured by sensors during an earthquake event.

	Damage state probability			Repair cost ratio	
Analysis	Spalling	Bar Buckling	Failure	Median	Dispersion
Original	63%	1%	0%	3.91%	0.909
Updated	96%	4%	2%	5.67%	0.707

the example bridge structure using an analytical finite element model implemented in OpenSees (http://opensees.berkeley.edu). The damage model considered only the bridge columns and was limited to values contained in the PEER structural performance database (http://nisee.berkeley.edu/spd/). The decision model used repair cost ratio data obtained from reconnaissance data during the Northridge earthquake are reported in HAZUS (HAZUS 1999). Simulated sensor data are used to update the seismic performance assessment for the example bridge. An extreme event with a 2% in 50 year probability of occurrence is assumed to have occurred in this example scenario. This event induces a maximum drift ratio of 4.0%, measured by sensors. This measured drift ratio is 1.6 times larger than the median predicted by the analytical demand model. The impact of the sensor data on the estimate of bridge column damage and repair cost ratio is shown in Table 1.

The example demonstrates the functionality of the proposed method of incorporating sensor data into the PEER PBEE framework. Several aspects of the approach presented can be further refined. One aspect is in the analysis of the various sources of uncertainty. Another aspect is the evolution of structural response over time and consideration of damage accumulated during the lifetime of a structure. Nevertheless, the changes of the damage and repair cost estimates due to inclusion of the measured data are substantial. They are important and can be used within the PEER PBEE framework to obtain a better estimate of the seismic performance of other similar bridges. Such improved estimates can be used in the immediate post-earthquake emergency response planning, or to, in longer term, re-prioritize seismic upgrade work on bridges in a traffic network to increase the safety and reliability of a regional transportation system.

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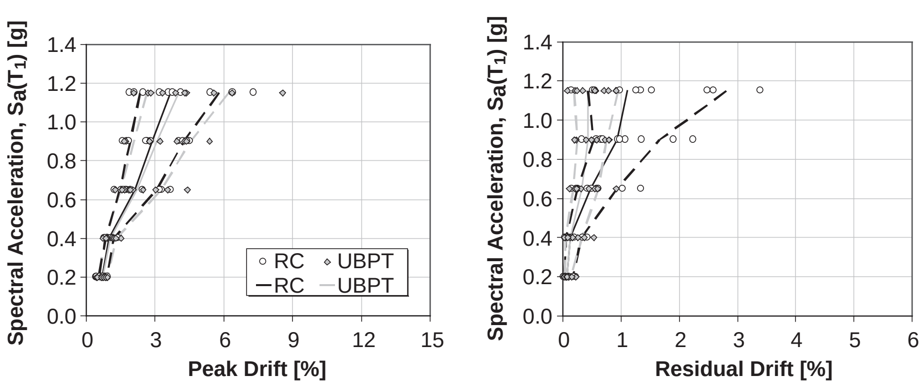
Analytical assessment of the post-earthquake condition of self-centering versus traditional concrete bridge pier systems

W.K. Lee & S.L. Billington
Stanford University, Stanford, California, USA

ABSTRACT: Ensuring adequate post-earthquake operability of highway bridges is crucial for maintaining transportation networks which provide vital services after an earthquake. In highly seismic regions, reinforced concrete bridges are designed to undergo large inelastic displacements during earthquakes, leading to possible large permanent or residual displacements. These residual displacements are an important factor in determining post-earthquake functionality of a highway bridge. To mitigate the problem of residual displacement, methods of providing self-centering to structural reinforced concrete members have been developed for bridges and buildings and has been a subject of recent research both in the US and in Japan. Such systems use vertical, unbonded post-tensioning to facilitate self-centering of the piers (i.e. cause minimal residual displacements) after cyclic loading with low hysteretic energy dissipation. A formalized performance-based earthquake engineering assessment methodology developed by the Pacific Earthquake Engineering Research (PEER) Center in California, USA is being used to assess quantitatively the possible benefits of self-centering systems for structural concrete bridge piers. Potential benefits of self-centering piers include reduced repair costs and reduced bridge closures.

This research focuses on assessing self-centering piers relative to traditional reinforced concrete piers in a case study bridge designed according to the California Department of Transportation Bridge Design Specifications and Seismic Design Criteria, with a geometry and configuration typical of the majority of highway bridges in California. The bridge is assumed to be located near the Hayward fault. The configuration of the self-centering piers consists of adding a single UBPT tendon at the center of each pier in the single-pier bent of the bridge.

A demonstration of the performance-based assessment methodology is presented, including hazard analysis, structural analysis, damage analysis and loss analysis. A suite of near-fault ground motions scaled to various hazard levels was used to perform the dynamic structural analysis. A nonlinear model representing the bridge with self-centering columns was subjected to these ground motions to evaluate the seismic behavior. Results of the analysis are shown in the figure below.



Given the resulting structural response, the predicted damage was determined through fragility curves. Damage in the columns is represented by spalling of cover concrete, buckling of longitudinal reinforcing, and severe residual displacements that require replacement of the column. Finally, expected losses are computed given the predicted levels of damage in the bridge. Losses are measured in terms of repair costs and downtime of the bridge. The bridge with self-centering piers is found to perform very well, with similar peak displacement demands to the conventional bridge, and with a 64% reduction in residual displacement after severe earthquakes. While repair costs for the bridge are found to be similar to those of a bridge with conventional reinforced concrete columns, the considerable reduction in residual displacements greatly reduces the risk of significant downtime for the bridge after a seismic event. The use of PEER performance-based earthquake engineering assessment methodology has been demonstrated as a valuable tool for quantitatively assessing and comparing novel, enhanced-performance materials or systems, which is not possible using current design and assessment guidelines.

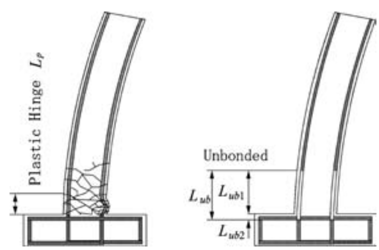
Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Seismic performance of unbonded columns and isolator built-in columns based on cyclic loading tests

K. Kawashima & G. Watanabe
Department of Civil Engineering, Tokyo Institute of Technology, Japan

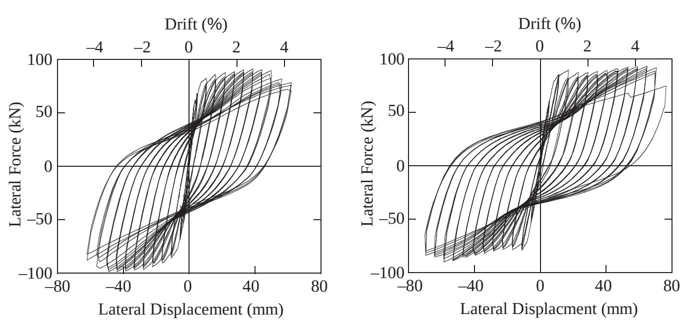
ABSTRACT: The bond between longitudinal bars and the concrete results in concentration of damage at a specific localized region. One of the measures to mitigate such a concentration of damage at the plastic hinge is to unbond the longitudinal bars from the concrete. By appropriately unbonding the longitudinal bars at an interval with length L_{ub} as shown in Figure 1, the deterioration of longitudinal bars is mitigated resulted from reduced strain at the interval L_{ub}.

A series of cyclic loading test was conducted on 1.45 m tall single columns with a square section of a width equal to 400 mm. The concrete strength was 24 MP, the longitudinal reinforcement ratio was 0.95%, and the volumetric tie reinforcement ratio was 0.77%. An important feature of the unbonded column is that it responses in a rocking mode. Since the longitudinal bars are unbonded at an interval of L_{ub}, the deformation of longitudinal bars in tension results in the dominant rocking mode of the column. As a result of the small flexural deformation of the column, the flexural failure at the plastic hinge of the column is limited. Figure 2 compares the lateral force vs. lateral displacement hysteresis between the standard and unbonded columns. The restoring force of the



(a) Standard Column (b) Unbonded Column

Figure 1. Unbonding of longitudinal bars.



(1) Standard Column (2) Unbonded Column

Figure 2. Lateral force vs. lateral displacement hystereses.

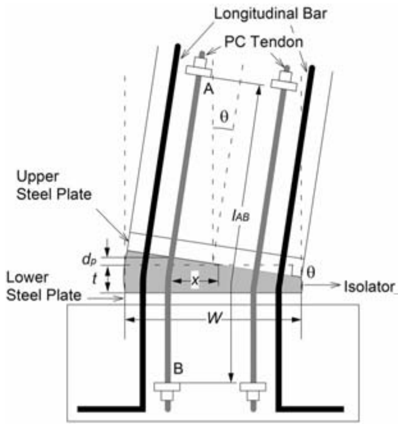


Figure 3. Isolator built-in column.

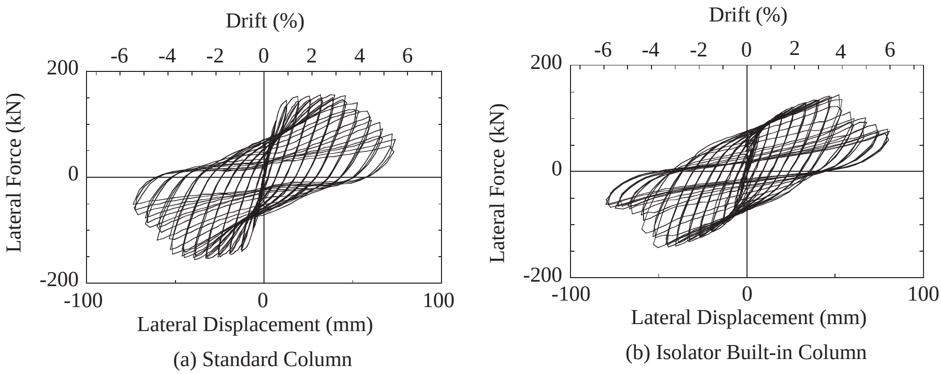


Figure 4. Lateral force vs. lateral displacement hystereses.

standard column starts to deteriorate at 9 δ_y (=3.9% drift), while the restoring force is stable until $11\delta_y$ (=4.8% drift) in the unbonded column. Based on the studies, it is considered that the unbonding is an effective means to increase the ductility capacity of columns by properly choosing the unbond length L_{ub}.

Since the hysteretic behavior of a reinforced concrete column occurs only in the plastic hinge, it is interesting to replace the concrete in the plastic hinge by an appropriate material that provides enough deformation and energy dissipation so that the flexural deformation in the rest of a column is mitigated. The material has to be appropriately softer than the reinforced concrete in order to reduce the flexural deformation of the column. One material studied is the high damping rubber that is used for standard high damping rubber bearings for seismic isolation. If one sets a high damping laminated rubber unit at the bottom of a column, the column deforms as shown in Figure 3 under a lateral seismic force. The longitudinal bars are continuous through the laminated rubber unit. Prestressed tendons are effective to prevent sudden deterioration of the restoring force and to mitigate residual drift. Since such a column is virtually equivalent to a built-in high damping rubber isolator, it is called here an *isolator built-in column*.

A series of seismic loading tests was conducted on 11 columns to verify the performance of the isolator built-in columns. Model columns were constructed 1350 mm tall (effective height) with a 400 mm by 400 mm rectangular section. 30 mm and 60 mm thick damping rubber units were used with an initial shear modulus of 1.2 MPa. The longitudinal reinforcement ratio was 1.58%, and the volumetric tie reinforcement ratio was 0.79%. Four PC tendons with a diameter of 9.2 mm were provided at the corners. The columns were laterally loaded under displacement control subjected to a constant axial force of 240 kN.

Figure 4 compares the lateral force vs. lateral displacement relations of the two columns. A remarkable change of the shape of the hysteresis loops is seen. The lateral force is virtually the same at the post-yield zone in the standard column, while it increases as the lateral displacement increases in the isolator built-in column. An important difference of the isolator built-in column is the smaller initial stiffness due to the soft deformation of the rubber unit. However, since the stiffness of the standard column deteriorates due to progress of failure, the difference of lateral stiffness between two columns becomes small over 2.5% drift.

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Seismic performance of reinforced concrete bridge columns encased in fiber composite tube

Z. Zhu & A. Mirmiran
Florida International University, Miami, FL, USA

M. Saiidi
University of Nevada, Reno, NV, USA

ABSTRACT: Recent experiments in the US, Japan and China have shown that replacement of transverse steel reinforcement in a reinforced concrete (RC) bridge column with an external fiber reinforced polymer (FRP) tube can significantly enhance ductility and energy dissipation capacity of the column. This paper reports on a comparative analytical study to examine the effect of column encasement in FRP tube on its seismic performance at the sectional, member, and system levels. A multi-span bridge in the State of Washington was chosen for this comparison. A non-linear finite element model of the bridge was assembled with and without FRP tube. Since FRP has a lower rupture strain than steel spiral, and because of its linear elastic response to failure, the tube-encased column section shows less ductility but higher strength than its RC counterpart. On the other hand, at the member level, the tube-encased column distinctly outperforms its RC counterpart with almost twice the base shear and over three times the lateral drift. This phenomenon was attributed to the effective role of FRP tube in extending the plastic hinge zone of the column well beyond its typical range in conventional RC columns. The earthquake spectrum analysis indicates that the bridge with tube-encased RC columns is capable of withstanding a major earthquake with 2500-year return period, while the conventional bridge suffers irreparable damage. A seismic simulation of the entire bridge under a major historical earthquake confirms these findings.

Fiber reinforced polymer (FRP) composites have in recent years been used in a variety of bridge applications. By far, the most prevalent application of FRP has been the repair and retrofit of bridge girders and pier columns, where the reduced construction time and labor offset the higher initial cost of FRP materials. New bridge construction, on the other hand, has seen much less influence from FRP industry, mostly due to the higher cost of materials. There is an additional concern in seismic regions, since FRP materials are known to be brittle and linear elastic to failure. To counteract both the economical and technical concerns, many investigators have proposed the combined use of FRP with concrete. One such application is the in the form of encasing bridge concrete column with FRP, i.e., concrete-filled FRP tubes (CFFT).

The tube provides light-weight permanent form as well as life-long protective jacket for concrete. Previous studies have shown that the entire reinforcement in the column could be replaced by the fiber composite tube without affecting the load carrying capacity of the column. As a result, the concept has been applied in a number of piling applications in the US. One example is the precast piles on Route 40 bridge in Virginia.

On the other hand, early studies had ruled out the effectiveness of carbon fiber tube as a replacement of internal steel in concrete columns. Their tests on quarter scale models of carbon CFFT columns showed no ductility for the embedded FRP tube with no internal steel, leading to the design recommendation that steel starter bars are needed to connect CFFT to its footing. They also suggested a small gap in the order of 50 mm from the bottom of the tube to the top of the footing to avoid over-strengthening of the section at the column base.

Recently, it was shown that with proper design of fiber orientation in the tube, it is possible to achieve considerable ductility in concrete columns with glass FRP tubes as a replacement of all internal steel. Extending their earlier work, the authors have suggested the use of glass FRP tubes in columns as a replacement of transverse steel while maintaining the longitudinal reinforcement in the plastic hinge zones.

The objective of this study is to simulate the effect of FRP encasement for RC columns subjected to a major earthquake record. This study is based on the component tests carried out earlier by the authors on single RC columns with and without FRP tube encasement.

A comparative study was carried out on the seismic performance of RC pier columns with and without FRP tube. The moment-curvature response of the column section with FRP tube has higher ultimate strength but less ductility as compared to its RC counterpart. On the other hand, the push-over analysis shows the CFFT frame to have much more deformation capacity than its RC counterpart, simply because of the effectiveness of the FRP tube in extending the plastic hinge zone of the column along its height, thereby engaging a larger portion of the column in the energy dissipation process. Simulated under a major earthquake record, the CFFT substructure is shown to sustain only minor damages, while its RC counterpart will suffer severe damage at the verge of collapse.

This study shows the benefits of the FRP tube in replacing the lateral steel reinforcement of RC columns in seismic regions. The encouraging results can be used for the next generation of highway bridges in earthquake prone areas.

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Analysis of reinforced concrete bridge columns with shape memory alloy and engineered cementitious composites under cyclic loads

M. Saiidi, M. Zadeh & M. O'Brien
University of Nevada, Reno, Nevada, USA

ABSTRACT: Current seismic design practice for reinforced concrete columns focuses on yielding steel when subjected to large lateral loads in order to dissipate energy. This causes large permanent displacements that make the structure susceptible to severe damage and possible collapse. Damage to plastic hinge is an accepted practice to allow for energy dissipation. However, the consequence of this damage can interfere with disaster recovery operation and can have major economic impact on the community. Column plastic hinges that can dissipate energy without suffering from severe damage and permanent deformation would alleviate these problems. This study focused on incorporating two innovative materials into 0.2 scale bridge column models in order to create more ductile columns that experience substantially reduced damage and permanent deformation while dissipating the earthquake energy.

Shape memory alloys (SMA) are able to undergo large levels of strain (up to 8%) and still recover their shape through either heating (shape memory effect) or stress removal (superelastic effect). The behavior of SMA is a function of temperature, stress and strain. In this study, the super-elastic behavior of NiTi (NiTi: 55.9% Nickel, 44.1% Titanium), one of the most often used SMA, was explored.

Another issue that arises during earthquakes is the brittleness of conventional concrete and its low capacity in tension. The replacement of conventional concrete by Engineered Cementitious Composites (ECC) at the column plastic hinge could improve the column behavior under earthquake loads. ECC is a fiber-reinforced cement-based composite engineered for high tensile ductility, compressive and tensile strength. ECC has the ability to reach ultimate tensile strain of approximately 3% to 5% because of its unique cracking mechanism in which many closely spaced microcracks form. In compression, ECC has a lower modulus of elasticity (due to the lack of large aggregates), and a higher stress capacity as compared to conventional concrete. This study investigates the use of ECC with 2% synthetic poly-vinyl-alcohol (PVA) fiber content by volume.

The combination of the low deformability of ECC with the superelastic characteristics of NiTi offers great potential of increasing ductility, and decreasing damage and residual displacements of concrete members. The possible use of NiTi and ECC has been analytically investigated in this study. Three 0.2 scale reinforced concrete bridge columns were modeled (all with 10 inch (254 mm) diameter and 45 inches (1143 mm) height). The first column, which served as a reference, incorporated conventional concrete and steel reinforcement. It is referred to as RSC, for "round steel conventional concrete." The second column incorporated conventional concrete and NiTi, for longitudinal reinforcement in the plastic hinge region. This column was called RNC for "round NiTi conventional concrete." The third column was similar to RNC, but the conventional concrete was replaced with ECC in the plastic hinge region. This column is referred to as RNE for "round NiTi engineered cementitious composites."

NiTi and ECC were only used in the plastic hinge zone of RNC and RNE. The NiTi rods were 14 inches (355.6 mm), extending 10 inches (254 mm) above and 4 inches (101.6 mm) below the top of the footing to the middle of the couplers connecting to steel rebar. RNE also included ECC to evaluate the composite performance of ECC with NiTi. The height of the ECC was chosen to be 20 inches (508 mm); 10 inches (254 mm) above the NiTi-steel connection to avoid weakened capacity at the material interface.

The computer program “Open System for Earthquake Engineering Simulation” (OpenSees) was used to perform numerical analysis of the columns. Three programs were created to evaluate the columns behavior: moment curvature, pushover, and cyclic loading. The programs were developed based on the actual column specifications and material properties.

The results of the analysis show that RNE has the highest capacity when compared to RSC and RNC. The reason for this is the high tensile and compressive strength of ECC. There is a sharp peak point in the moment-curvature and force-displacement graphs of RNE indicating high tension capacity of ECC before cracking. RSC requires higher moments and forces to reach the same curvatures and displacements when compared to RNC. This is due to the higher modulus of elasticity and higher strain hardening slope of steel as compared to NiTi. Steel enters strain hardening much more rapidly, so higher stress is necessary to reach the same strain levels when compared to NiTi, which plateaus at almost constant stress but increasing strain

The program used in the analysis for static-cyclic loading was based on user-defined peak displacement points. The loading plan was determined using drift ratio (displacement at the top of the column versus the total column height). The behavior of RNE showed that smaller increments at the beginning of the cycling are necessary to capture the sharp peak. Therefore, the cycles were specified as one cycle at 0.25% drift, two cycles at 0.5%, 1% and 1.5% drift, then 1% increments starting at 2% drift until failure. To keep consistency between the three test specimens the same load plan was used for all columns.

The cyclic loading programs showed that NiTi provides the column with recentering capabilities, and thus limited residual displacement. Also, in RNE it was shown that the strength of ECC, especially in tension, helps the column resist higher lateral forces with less strength degradation, as compared to the conventional concrete columns.

Energy dissipation calculations (based on the area bound by the hysteresis curves) show that RNE can dissipate more energy than the other two columns, especially at higher drift levels. The combination of ECC and NiTi in RNE contributes to the high energy dissipation compared to lower energy dissipation in RNC. This also indicates that less damage is sustained by RNE, including damage to the longitudinal reinforcement and the concrete in plastic hinge zone, as expected.

Although the analytical results showed that the combination of NiTi and ECC in bridge column plastic hinges could potentially make bridges serviceable by decreasing residual displacement, increasing ductility, and capacity to resist high forces, even after undergoing large deformations, actual laboratory testing is necessary to verify the analytical results.

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Seismic upgrade of column-bent cap connections of Alaska bridges

M.C. Lubiewski, P.F. Silva & G.D. Chen
Department of Civil, Architectural, and Environmental Engineering,
University of Missouri-Rolla, Rolla, MO, USA

ABSTRACT: Typical bridges in the state of Alaska are characterized by columns with excessive reinforcement ratios encased in a steel shell that is partially embedded in the bent cap; the bent caps are inadequately designed for flexure, shear, and joint shear for the feasible moments that can be imposed upon them during a seismic event. A research program was undertaken to develop a method to improve the seismic response of such typical bridges using conventional retrofit techniques.

Three 80% scale specimens were built to model the deficiencies of as-built bridges in the state of Alaska. Based on current capacity design practice, it was decided that the retrofit should be designed to allow the bent cap to stay in its elastic range while forcing a plastic hinge to form in the column. To accomplish this, the reinforcement ratio of the column was reduced at the joint, the bent cap was enlarged, and additional flexural, shear, and joint shear reinforcement were added to the bent cap. In addition, the steel shell was cut leaving a gap at the interface of the column and bent cap.

The first specimen failed due to splitting of the bent cap and pullout of the column longitudinal reinforcement. The transverse reinforcement was installed from each side to a depth of half the beam which allowed the crack to form. For the second specimen, the transverse reinforcement was continuous across the beam. However, the second specimen's load vs. deformation response was very similar to the first specimen. The third specimen was post tensioned in the transverse direction and has yet to be tested. Discussions of the results, along with recommendations for improving the retrofit program are discussed in detail in the paper.

Soft computing in bridge engineering

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Application of soft computing techniques to safety management during bridge construction

A. Miyamoto
Yamaguchi University, Ubee, Japan

ABSTRACT: Bridge construction has become complex and is being conducted on ever larger scales, and the technology of bridge construction is advancing rapidly. Currently, problems in the safety management during bridge construction are very important. In our country, Japan, the number of worker killed on construction sites steadily decreased in the past, however the rate has been constant for the past 15 years. Therefore, the causes of serious accidents on bridge construction sites have been analysed. The results of our analysis reveal that serious accidents are caused by a combination of lesser accident factors. It is reasonable to suppose that serious accidents can be prevented if potential factors are eliminated. Skilled construction workers are able to avoid potential accident factors because of their experience and knowledge. However, the number of skilled workers has decreased and their valuable knowledge and experience is seriously needed. The aims of this paper are not only to develop an expert system for supporting safety management in the bridge construction but also to apply 3-D computer graphics (CG) to simulate and also to make better understanding the accidents during bridge construction for integrating into the expert system.

The proposed expert system uses, as knowledge, hierarchized factors of serious accidents, such as falling down and collapse, and the probabilities of those factors causing accidents to predict accident risks from the actual conditions of construction sites and present dangerous spots. Because factors of accidents differ among construction methods, an expert system for safety management has to be constructed for each construction method. The expert systems developed in this study are systems for the launching method and the large block erection on carriage which are relatively special. The launching method has often seen accidents, and the system for the method was constructed based on the presumed factors of accidents and the knowledge of experts. The large block erection on carriage is relatively new and no accident has been reported yet. It is, however, technically advanced and will often be used in future, then it was considered significant to construct a system for the method. In cooperation with experts, presumed factors of accidents were extracted for the construction of the systems. Again, the pieces of knowledge were expressed as *IF-THEN* production rules. The input and output pictures were designed to make the systems practical. The systems were verified by using actual data on accidents and a questionnaire survey of experts as to the effectiveness of the systems was made. And also, the systems were constructed so that they can be used to check risks at construction sites.

The safety management system designed to prevent serious accidents occurring using the launching method for steel bridge construction presents an estimated danger level by assessing the various accident factors that result in serious accidents. In order to clarify the relationship between the cause and affect for serious accidents, a hierarchical classification, based on the work procedures for the launching method is created. Analysis examples using data from previous construction accidents were performed to determine the suitability of this expert system. The results illustrate the systems efficiency as a safety management system, however, further analysis and research of the accident factors would clarify the causes and improve its effectiveness. And as an additional function of the system, in order to better understanding the accidents during bridge construction works, 3-D images of some kinds of accidents was created on the system as an animation of accident examples.

The results of this study are summarized as follows:

(1) The safety control system has developed to prevent serious accidents in the launching method for steel bridge construction, as an expert system.
(2) The safety control system will be able to make advice the possibility of accident by the estimated dangerous level with 3-D images.
(3) A few examples assigned in site condition data on the actual construction accidents are performed for demonstrate the usefulness of this expert system (see Figure 1).
(4) The Virtual Bridge Construction System, integrating the "Bridge Construction Sequence Simulator" and "Safe Bridge Construction Simulator" and designed for use on construction sites, was developed.
(5) The animation created by 3-D CG allows construction workers to easily understand bridge construction sequence which used to be prepared based mainly on two-dimensional drawings produced by designers (see Figure 2).

Figure 1. Input screen for the check item.

Figure 2. Example of 3D-CG for steel bridge construction.

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Imaging-based surface quality assessment of weathering steel bridge based on wavelet transform and support vector machine

B.F. Yan
Institute of Bridge Engineering, College of Civil Engineering, Hunan University, China

S. Goto
Ube Machinery Corporation, Ube, Yamaguchi, Japan

A. Miyamoto
Department of Computer Science & System Engineering, Yamaguchi University, Japan

ABSTRACT: The use of weathering steel is increasingly important to prevent further corrosion of steel due to its special characteristics of forming a stable and protective oxide layer termed rust patina in suitable environments. As a result, the weathering steel having natural corrosion resistance is becoming an appropriate choice for structural components of new steel bridges since 1964.

The monitoring and condition assessment for weathering steel bridges are becoming important aspects on appropriate material selection in planning and design stages, corrosion prediction and further maintenance planning during their service life. The popular NDT methods such as ion transfer resistance method, electrochemical potential method, X-ray diffraction method, spectroscopic identification techniques are employed to measure the corrosion behaviors from the point of view of corrosion loss, and chemic, electric and magnetic properties of the rust patina. Note that human vision has played an indispensable role in extracting effective and directive information from the external appearance. Since the atmospheric environment primarily causes the corrosion of the exterior of the weathering steel, the state of the surface rust is sufficient to exhibit the corrosion damage. Japan Association of Steel Bridge Construction and Japan Iron and Steel Federation proposed a standard criterion to evaluate the state of weathering steel based on visual inspection and a supplemental rust thickness measurement, and categorized the surface condition into 5 levels ranging from 5 to 1. It is obvious that the expert visual inspection and evaluation are subject to the impacts of subjective decision-making such as experience, ages, and even temper. Moreover, conventional visual analysis approaches are time-consuming work, having high degree of variability, failing to provide quantitative information. Recently, digital image recognition has been used experimentally in the automated system for bridge inspection, pavement management, sewer pipeline inspection, material engineering. The critical issues of this approach contain imaging-based feature extraction and pattern classification algorithm. For imaging-based feature extraction, wavelet transform (WT) has shown better performance in many cases due to its outstanding capabilities in space-frequency decomposition at each resolution. Another crucial issue is the pattern classification, numerous classifiers using the extracted features have been employed, including nearest neighbor classifier, Bayes classifier, probabilistic neural network, learning vector quantization. Recently, the support vector machine (SVM) is becoming a successful method for pattern recognition.

In the cases of weathering steels, it is generally believed that the surface corrosion characteristics of them under different controlling factors can be described by morphology, i.e., color, texture and shape. If we are able to construct a relationship between morphological attributes of the training sample images and the corresponding qualified condition state, it is desirable to make a pattern classification on morphological attributes of the new test image.

This paper presents an imaging-based method for condition rating of surface quality of the corroded weathering steel bridges using WT and SVM. A schematic of the imaging-based condition

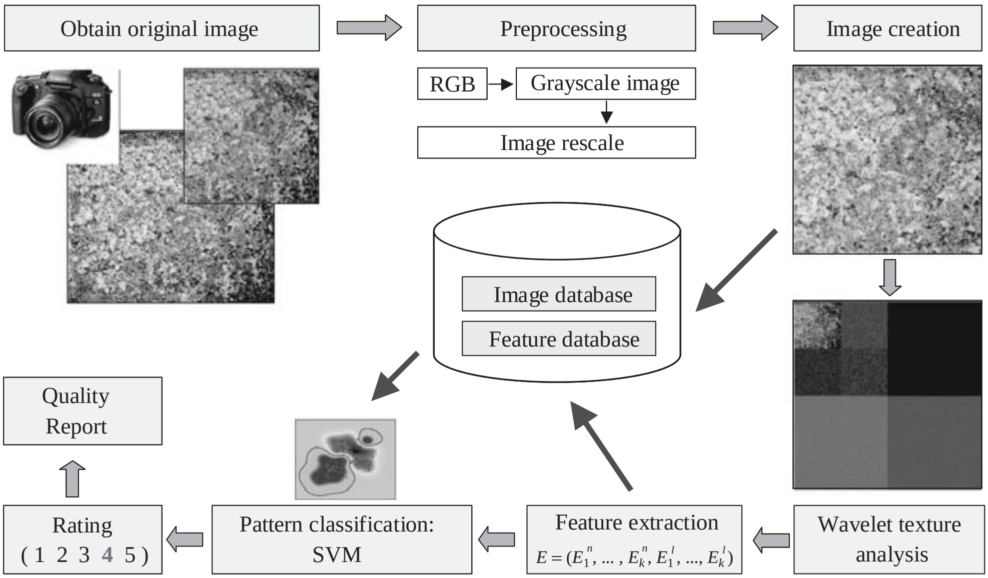

Figure 1. Imaging-based condition rating of weathering steel using WT and SVM.

rating for weathering steel using wavelet texture analysis and SVM is shown in Figure 1. The surface image data of weathering steel acquired by digital camera were preprocessed to create standard rescaled grayscale image for wavelet texture analysis, from which both local and normalized global energy distribution of each detail sub-image are extracted as the feature vectors. Then, the SVMs were implemented to classify a set of wavelet texture features into various corrosion levels related to condition state of the surface images of weathering steel. The quality assessment was finally reported.

The systematic investigation on a representative set of 530 images indicates that, by carefully select the decomposition level and texture feature vector, the averaged classification accuracy can attain to 82% in the case of using half samples for training and half for testing. The limitation of this imaging based pattern classification method is that the morphological characteristics of surface image such as color, texture, shape can cover most information of the surface quality but not all. Moreover, the critical feature of thickness of rust patina can not be fully indicated by morphology. The further study will placed emphases on selection of new feature vector and comparisons with many other pattern classification algorithms.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Development of standardized semantic model for structural calculation documents of bridges and XML schema matching technique

S.-H. Lee, B.-G. Kim, D.-H. Kim & Y.-S. Jeong
School of Civil & Environmental Engineering, Yonsei University, Seoul, Korea

ABSTRACT: The structural calculation document of bridge is the important information set to be maintained because it contains core information which can be reused for the diagnosis of structural safety and the damage assessment during the lifetime of bridges. In general, most of the structural calculation documents are managed through the paper-based documents or the electronic files generated by software such as word processor, spreadsheet program, etc. Such management way, especially in development of a decision support system, has a limitation in reusing the structural calculation information. Therefore, in the digitalization of structural calculation documents, a scheme which assures the flexible use of the document in the heterogeneous computing environment should be established. In the recent studies, the extensible markup language (XML) is widely used for the interoperability of document information. For example, Zhu et al. (2001) proposed a framework for the construction process based on XML-based technology; Tserng & Lin (2003) developed XML schema for scheduling based on the aecXML framework; Zhilian et al. (2004) provided an in-formation exchange method in the construction project based on XML technology and they use it to develop a decision support system for the construction phase (Zhilian et al. 2005); and Lee et al. (2004) developed an XML-based framework for bridge design documents. In general, the document schema is used to operate the XML document and the data schema can be seen as a semantic model for the document. Therefore, as Song et al. (2002) pointed out, it is generally true that the standardization of the document data requires a large degree of agreement upon format, contents and responsibilities. However there are few researches that are reported in the literatures for the efficient standardization of the document model.

This study provides a new formal standardization methodology of the structural calculation document for bridge maintenance and a standardized semantic model of the structural calculation document for general types of the steel bridges by using the developed methodology. Application modules capable of checking document data and storing them into a standardized database are also developed.

The standardization process of the structural calculation document is composed of three stages. In the first stage, sample documents are collected, and specific document contents in the collected documents are indexed with templates defined in this study to make the document contents and their structures recognizable in the application program. In this study, the text file format is used as a neutral file format of the collected document contents because it can be exported easily from the most of the commercial programs.

The second stage that deals with construction of temporary document schema can be divided into three steps. In the first step, the indexed document is read by the application program, and a document tree is made on the computer memory. Thereafter, the document tree which is a new collected document schema can be compared with the temporary document schema by using the schema matching techniques presented by Yi et al. (2005), and then the new content object is added to temporary document schema. This second part is repeated until all of the collected documents are compared.

As a final stage, the common elements and unbounded elements are extracted by determining the occurrence of the temporary document elements, and the standardized document schema is exported in the XSD format.

(a) XML generator (b) document viewer (c) integrated information viewer

Figure 1. Operation example of the standardized semantic model for structural calculation document.

For a case study, the 41 kinds of structural calculation documents subjected to steel plate girder bridges and steel box girder bridges are collected, and all of the bridges are in the first and second classes determined by "Special Act on Safety Control for Infrastructure" in Korea. The standardized semantic model was composed of 5 major parts: design conditions, design of bridge deck, design of structural members, design of utilities, and usability check.

Figure 1 illustrates web-based application examples of the standardized semantic model developed. The XML generator shown in Figure 1(a) is capable of translating the original document into XML document. After the translation is complete, the structural calculation document can be stored in the XML database and it can be retrieved on the web browser as shown in Figure 1(b). The Relay Module supports the integrated operation of the STEP-based bridge information (Lee & Jeong, in press) and the XML-based document information. Figure 1(c) shows an example of the integrated document viewer displaying the section information of a girder in segment 'NM1' of the Hannam bridge. The lower section of Figure 1(c) represents a result of data consistency check. Considering the fact that different users prepare the document information and the CAD information in general situations, this is a useful function to guarantee the data consistency during the long-term lifetime of bridges.

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*Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Application of PSO algorithm to damage identification for concrete bridges

M. Beppu
National Defense Academy, Yokosuka, Japan

H. Emoto & A. Miyamoto
Yamaguchi University, Ube, Japan

ABSTRACT: This study proposed a method to identify damage of concrete bridges using Particle Swarm Optimization (PSO) algorithm and FEM. In the truss structure problem, the feasibility study on the location and severity of damage using this integrated method was conducted. Then, damage identification for reinforced concrete beam was performed to verify the effectiveness of this novel method.

Recently, maintenance technology for infrastructure such as concrete bridge has been notable in Japan because these structures built in economical development period are getting decrepit. In the maintenance of structures, damage assessment is very significant. In practice, damage assessment has been carried out by visual-inspection. However, it may be difficult to examine the damage severity of structures objectively because of the effect of uncertainty in the process of inspection and evaluation. By now, lots of studies for damage assessment have been conducted. For example, health-monitoring methods and damage identification algorithms have been developed. In the case of damage identification methods, inverse analysis based on filter theory was proposed (Yoshida et al., 2002). Miyamoto et al. proposed a system identification method to determine the damage parameters utilizing dynamic structural sensitivity on natural frequency (Miyamoto et al., 1992). These methods termed as inverse analyses identify the location and extent of damage by means of comparison of measured data between normal and disorder condition of structures.

Solution methods of inverse analysis are categorized into two types of direct and back formulation method. In general direct formulation method, appropriate constants are introduced into numerical analysis like FEM, next the difference between measured and calculated value will be examined. This procedure will be iterated until the difference converges to tolerance value. Hence the number of calculation tends to increase. On the other hand, the problem will be solved by substitution of measured data into formulated equation and unknown parameters will be determined mathematically. In this study, damage identification method using Finite Element Analysis (FEM) and Particle Swarm Optimization algorithms (PSO) (J. Kennedy et al., 1995; R. C. Eberhart et al., 1996), which is one of artificial intelligence tool, is proposed (see Figure. 1). Figure. 1 shows procedure of PSO and calculated error value. The damage level rate is identified by PSO algorithm. This method as one of direct formulation methods appears to be versatile and easy to use. Firstly, an investigation on novel stochastic search algorithm termed PSO is conducted to verify its solution characteristics and effectiveness. Then, a PSO is proposed and demonstrated on a few simulated examples of a simple truss structure to explore its performance on the multiple optimal solution problems. Finally, an application of this strategy to RC truss model is implemented and its effectiveness is verified.

In this study, we make an attempt to use PSO for detecting damage level and its location. First of all, feasibility study of damage identification on truss structure was conducted based on FEM and PSO. Next, application to RC beam was carried out to clarify the effectiveness of this novel method.

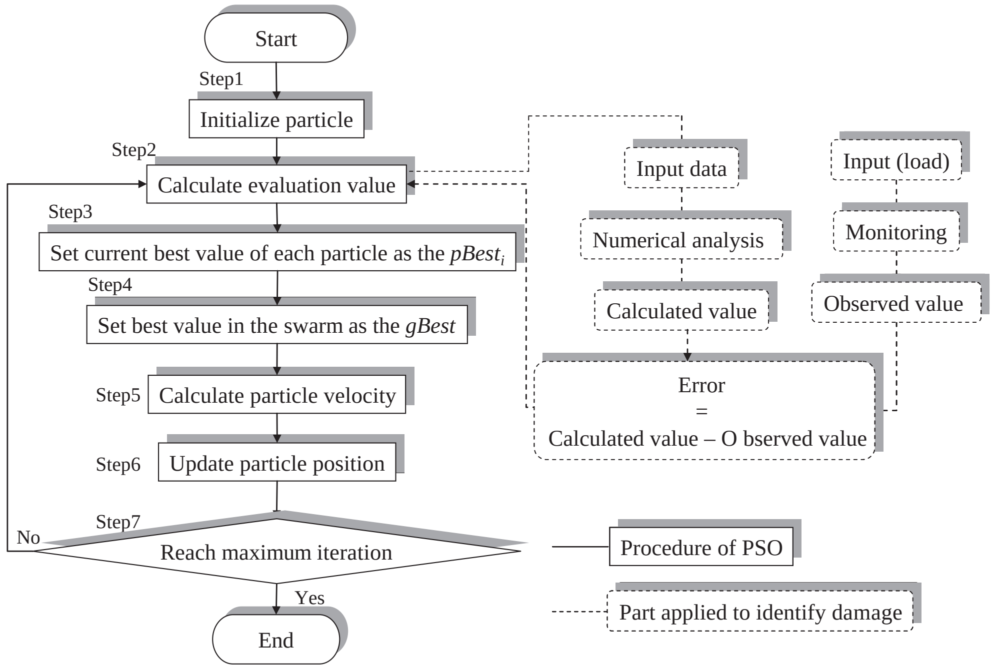

Figure 1. Procedure of damage identification using PSO.

The conclusions are summarized as follows:

(1) When the prior information for damage location is known, the damage in truss is identified with 99% accuracy in the case of identifying one elastic modulus (Problem 1), whereas the probability of identification is down to 70% in the case of unknowing two elastic modulus (Problem 2).
(2) Damage location in truss model was identified at 693 times among 1,000 trial times. When the load point was increased to two, the number of successful calculation has improved to 732 times among 1,000 trail times.
(3) It was found that when the iteration number reaches to 50, the damage location in RC beam was completely identified about 100% in 1,000 trials.
(4) We have seen that the identification accuracy of the element near the load point has improved when damaged level and its location in RC beam were identified.

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Monitoring of early age shrinkage using image analysis and it's use in the repair of bridges

K.C.G. Ong & Myint-Lay Kyaw
Department of Civil Engineering, National University of Singapore, Singapore

ABSTRACT: The very early age shrinkage of concrete has significant effects on the behavior and durability of concrete in service, especially in the case of composite specimens comprising freshly cast low W/C cementitious materials on a hardened concrete substrate. Such specimens are commonly encountered in the repair of bridges in the form of cementitious overlays, replacement of spalled cover concrete and enlargement of concrete bridge members. A new method of monitoring early age shrinkage strains beginning 30 minutes after adding water to the mix is presented herein. Where applicable, shrinkage strains monitored using this technique were compared with readings obtained using the laser sensor technique. Based on the results obtained it was found that shrinkage strains monitored using image analysis compares well with and as expected, are higher than that measured using laser sensors. This new technique is capable of monitoring differential shrinkage in the composite specimens tested.

1 INTRODUCTION

Resurfacing and overlays are commonly used on concrete bridge decks and existing concrete pavement. The main purposes of overlays include, increasing the load carrying capacity of old pavements and repairing the damage/deteriorated surfaces of concrete pavement. By increasing its overall thickness the flexural stiffness of the pavement is increased.

In the construction of thin overlays the early age shrinkage of overlay is expected to have a particularly adverse effect as the overlay is unable to shrink freely due to restraints afforded by the substrate which has little or no shrinkage tendency to which it is bonded. Tensile stress induced in the overlay by the restraint may cause cracking when the induced tensile stress exceeds the tensile strength of the overlay. Debonding of the overlay due to this differential shrinkage with depth may also occur. Furthermore the water absorbed by substrate in contact with a freshly cast cementitious overlay may cause an expansion of the substrate and that would aggravate the stress differential developed in the overlay as well. Thus a comprehensive knowledge of all types of shrinkage, both early age and long term shrinkage is essential.

In this paper, a technique to monitor very early age shrinkage of composite specimens comprising a new cementitious repair layer and an old concrete substrate is introduced. This new method makes use of image analysis (Ong & Kyaw 2005) for the monitoring of very early age shrinkage of concrete especially during the first 24 hours after casting. Using that method expansion of the substrate concrete and shrinkage of the repair concrete due to movement of water from fresh repair concrete layer to the substrate were monitored. Three types of moisture condition were employed to investigate the effect of moisture condition on moisture absorption and expansion. The effect of the restraint by the concrete substrate on early age shrinkage of the repair material was demonstrated.

2 EXPERIMENTAL INVESTIGATION

2.1 *Material and specimen preparation*

Two types of concrete mixes were used; Mix-S with W/C ratio 0.5 was used in casting the substrate concrete and Mix-R with W/C ratio 0.45 was used as the cementitious overlay. Two types of specimens were tested in this study viz. monolithic specimens and composite specimens. In the case of composite specimen, the concrete substrate were prepared with three different moisture conditions, oven dry (OD), air dry (AD) and saturated surface dry (SD).

2.2 *Very early age shrinkage of Mix-R*

Very early age shrinkage of monolithic with Mix-R specimens was monitored using image analysis technique and laser technique. The results showed that the shrinkage of Mix-R was very high before final setting of concrete and it may be noted that as much as 80% of the total very early age shrinkage monitored would not be captured if the initial setting time was chosen as the initial starting time to monitor shrinkage strains. And the result also showed that the readings monitored with image analysis were higher than that monitored using laser sensors on the same specimen.

2.3 *Differential shrinkage of composite specimens*

The result shows that the shrinkage strain reading of overlay in the composite specimen is much lower than the free shrinkage strain reading in monolithic specimen. With the water uptake by the substrate, the repair concrete tends to stiffen earlier and the overlay could not shrink as the free shrinkage. The result also showed that the differential shrinkage in the repair concrete, from a maximum shrinkage strains at a place away from interface to zero at the interface.

During the duration 30 minutes to 90 minutes after adding water to Mix-R the shrinkage strain in the overlay increase abruptly in the composite specimen with OD substrate. In the specimen with AD and SD substrate, shrinkage strains were still registering increases up to 2.5 hours and 9 hours respectively. OD and AD specimens registered much higher initial values of shrinkage than SD specimens. The shrinkage readings reaching after 90 days in the three types of composite specimens are comparable.

The result also showed that the expansions of all three composite specimens tested were about 50 microstrains higher than in the free expansion of monolithic specimen at the age of 24 hours after casting. This was probably due to the curvature that could be caused due to the differential shrinkage and expansion in the composite specimens.

3 CONCLUSIONS

Image analysis was used to monitor the early age shrinkage of a typical concrete mix (Mix-R) used in overlays. Both unrestrained (monolithic) specimen and restrained (composite) early age shrinkage of the mix was monitored. High shrinkage strains (2265 μm/m) was monitored in the monolithic specimens during the first 24 hours starting 30 minutes after adding water to the mix. If monitoring of shrinkage starts after initial setting of the mix the shrinkage monitored was 430 μm/m after 24 hour and 830 μm/m after 90 days respectively. In the case of composite specimens, the present test result showed that the early age shrinkage of a 50 mm thick overlay was lower than monolithic 100 mm thick specimens.

The restraint shrinkage of composite specimen is significant lower than free shrinkage of the Mix-R. The effect of moisture condition of the concrete substrate prior to overlay was evaluated by using three different moisture conditions. The rate of very age shrinkage of over was influenced by the moisture condition of the substrate. Results show that the rate of shrinkage strains in the overlay in the case of OD and AD specimens was higher than SD specimens. Shrinkage strains of specimens with all three types of substrate were comparable.

Based on the test result, a restraint force was generated at very early age about 90 minutes after adding water to the mix of overlay in the case of OD specimens. As a result, overlay was not allow to shrink freely, and causes differential shrinkage in the overlay; no or less shrinkage at a place very close to interface and highest at the place far away from the interface. The differential shrinkage during the first week after casting is more significant that at later age and that shrinkage could cause cracking or debonding in the composite specimens.

In this investigation, only one type of repair material and 50 mm thick substrate was employed. The influence of other repair materials and difference proportion of thickness is recommended for further study.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Development of an internet para-stressing system for intelligent bridge

T. Morisaki
Dept. of Mechanical & Electrical Eng. Collage of Technology, Syunan, Japan

M. Motoshita
Kyowa Sekkei Co.,Ltd, Osaka, Japan

A. Miyamoto
Dept. of Computer & Systems Eng., Yamaguchi Univ., Ube, Japan

ABSTRACT: Because Japan has been invested a large amount of money in social infrastructure business, the enhancement level of domestic social infrastructure facilities is extremely high. On the other hand, it implies increasing maintenance cost. By 2030, more than the half of bridges will become 50 years old or more. Therefore, it is feared that the bridge and road making business will be jeopardize (at risk) by costs of the administrative, maintenance and update (rebuilding) of old ones. In addition, since a social situation of recent years changes from high growth rate into the lower ones, the worldwide concern changes from the constructing new bridges to maintenance, management, repair and reinforcement of existing bridges. As the time passes, the number of old bridges will increase and there will be limitations in monitoring the bridge behavior which presently relies on human labor. In consequence, it is difficult to maintain a required measurement level. Moreover, a lot of accidents have been reported because of overlooking important damages because such parts could not be examined by surveyors. In order to solving above-mentioned problem, monitoring system is required to not only measure the minor behavior of a bridge, which is carried out by sensors installed in different positions difficult to measure by surveyors, but also collect and manage measurement data to effectively provide the measurement result for a bridge manager. Such a monitoring system should essentially be a remote measurement system which has automated and the real-time measurement functions. Additionally, the system is required in order to realizing or ensuring the effective operation of Bridge management system.

A cable-stayed bridge is not only an excellent spectacle, but also is an economical design because the bending moment of the main girder affected by the load can be adjusted by the amount of pre-tension applied to the cables. This "load" includes the dead load, i.e. bridge's own weight, and the live load such as vehicles passing on the bridge. However, in the case of light traffic on the bridge, there is a problem that such pre-tension becomes inversely an extra load imposing to the bridge, when it had been assumed that large live loads would get on the bridge. To solve this, it is required to equip the bridge with an actuator. Therefore, the system can automatically control the tension for counterbalancing or distributing local live load using that actuator. Such system is called "Para-Stressing system" or "effect system". Forty years ago, such idea was advocated first by Freyssinet and Zetlin. However, this research has not progressed greatly afterwards because at that time the demand to reduce architectural maintenance cost and architectural materials for the global environment was less than present. Recently, the social environment has changed greatly since the demand has become high and the not only cost of indispensable computers for constructing the system has been decreased but also high-speed Internet networks has been grew popular. Consequently, the bridge with installed system becomes a real possibility and the system has come to be researched zealously[1−2].

In this research, based on the above-mentioned background, we propose a bridge monitoring System that enables an automated, remote, and real-time measurement while the inspected result can be referred by officials from different place at the same time using Internet. It consists of a Stand-alone Monitoring System (SMS) and a Web-based Internet Monitoring System (IMS). Furthermore, we apply this monitoring system to Para-Stressing Bridge System (PSBS) consisting of a sensor, actuator and processor function and PSBS is verified by experimental results with the model of a cable-stayed bridge.

SMSs are set up on the sites of target bridges for the monitoring. The purpose is to continuously measure the external force and watch the behavior of the bridge affected by the force, via sensors installed in the bridge. The temperature in the vicinity, the stress and displacement of objects are measured concretely. In fact, SMS creates and preserves the graphs and tables to provide easy-to-use visual information based on the obtained data. In addition, SMS has the function of controlling actuators installed on the bridge. Para-Stressing Bridge System (PSBS) consisted of the several functions and SMS. PSBS is an intelligent bridge by integrating the bridge monitoring information collected from SMS and fed into the system to control the bridge performance automatically. PSBS generates the external forces (or moments) to counterbalance the applied heavy load or to distribute it locally. Therefore the demand for material of the bridge will be reduced while the allowable load is maintained. In addition, the initial cost of construction is expected to be reduced due to a recent great decrease of cost of sensors and the computers which are required to make an intelligent bridge.

Since from the viewpoint of managing the bridge efficiently, it is difficult to say whether SMS, which always measures the state of the bridge, is a practicable monitoring system, Internet Monitoring System (IMS) is added to the proposed system. IMS, as a web-based system, is capable of addressing the remote monitoring by introducing measuring information derived from SMS into the system through Internet or Intranet connected via PHS or LAN. Moreover, IMS has the key functions for bridge maintenance planning managers, such as data management system, condition assessment and decision-making. Although life of many bridges can be prolonged by implementing a maintenance process, few of them can permanently remain functional while this procedure is going on. On the other hand cost problems related to rebuild, maintenance and architectural materials (with regard to the problems of global environment) are becoming more and more important.

The bridge designer assumes a big load applies on the bridge for safety. However, during the life (service period) of a bridge, it is rare that such a load be applied. Such a design unnecessarily increases the amount of building material, and thus cannot meet the demand for the decrease of the construction cost. Moreover, with the problems of time and cost in conventional maintenance methods, which depend on human resources, it is necessary to supervise the generation and progression of abnormalities on a real time bases. PSBS generates the external moments to counterbalance the applied big load or to distribute it locally. Therefore the demand for material of the bridge will be reduced while the allowable load is maintained. In addition, the initial cost of construction is expected to be reduced due to a recent great decrease of cost of sensors and the computers required to make an intelligent bridge. For the purpose of verifying its validity to actual bridge structure, an attempt is made to study the possibility of controlling the actual structural performance by adjusting the cable forces on a two-span continuous cable-stayed bridge model.

Bridge monitoring system via information technology is capable of providing more accurate knowledge of bridge performance characteristics comparing to traditional strategies. This paper describes not only an integrated internet monitoring system that consists of a Stand-alone Monitoring System (SMS) and a Web-based Internet Monitoring System (IMS) for bridge maintenance but also discussed its application to PSBS as an intelligent structure.

The experimental results demonstrate that the implemented monitoring system supplies detailed and accurate information about bridge behavior for further evaluation and diagnosis, and it also opens up new prospects for future application of web-based remote system to actual in-service bridges under field conditions. As mention above, this research introduces the next generation of bridges, in which a lot of problems of the previous ones are solved.

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Optimal intervention strategies for multiple bridges during catch-up periods using age equivalents

B.T. Adey & R. Hajdin
Infrastructure Management Consultants, Zürich, Switzerland

E. Brühwiler
Division of Maintenance and Safety, Swiss Federal Institute of Technology (EPFL), Lausanne, Switzerland

1 INTRODUCTION

Many infrastructure owners now find themselves in a situation where it is no longer possible to spend only the predicted minimal long term resources to maintain their bridges. This has occurred due to reductions in funding to levels below the required expenditures for optimal long term management of bridges that have lasted multiple years and that have not been followed by periods of time with the additional funding required to catch-up. This normally happens due to the inability of infrastructure managers to provide solid estimates of the financial needs required and proof that their suggestions are optimal.

Once this happens even the resumption of spending at the initial level is insufficient and further bridge deterioration results. Consequently ever greater amounts of additional funding are required for the eventual *catch-up period*. The term catch-up period is defined herein to mean the period of time where the condition of deteriorated infrastructure is improved to the point that minimal long term costs can be achieved. During this catch-up period, however, it is still in the infrastructure owner's interest to follow an optimal intervention strategy taking into consideration workload and budget constraints.

The optimization of expenditures during catch-up periods requires that decisions be made based on bridge condition and intervention costs. Since intervention costs are linked directly with bridge condition, a stumbling block to the determination of these optimal strategies has been the qualitative descriptors of bridge condition, which cannot be used in optimization algorithms. This problem, however, can be overcome by using *age equivalents* to quantify bridge condition. Intervention costs can then be linked directly to condition through age equivalents.

In this article it is illustrated how age equivalents, coupled with binary programming methods, can be used to take into consideration the different conditions of bridges and the associated interventions, in the determination of optimal intervention strategies during catch-up periods. The use of age equivalents in the determination of optimal intervention strategies during catch-up periods is illustrated for a fictive park of bridges, consisting of 160 metal bridges whose initial condition can be classified into four different groups.

2 AGE EQUIVALENTS

The age equivalent of a bridge is the age that, the same bridge, newly constructed, without a rehabilitation intervention, i.e. only routine maintenance, such as cleaning of the drains, will have when it has the specified condition. For example, the protective coat of paint on a newly constructed metal bridge will be used up by the time the bridge is approximately 25 years old, but there will be no general deterioration of the structure. Therefore if an existing bridge, regardless of its date of construction or the interventions that have been performed on it previously, has the protective painting used up but there is not yet a general deterioration of the structure the bridge is considered to have an age equivalent of 25 years. In essence this means x condition states are introduced for each bridge, where x is the expected number of years that the bridge will remain in service if no interventions are done.

3 OBJECTIVE FUNCTION

The objective function is the minimization of all costs during the catch-up period while ensuring that each bridge has one and only one planned intervention and the annual permissible budget is not exceeded. Only *planned* interventions, i.e. those that can be accurately predicted based on the gradual change in bridge condition, are discussed in this paper. *Unplanned* interventions, i.e. the ones that are urgently required due to extreme events, such as floods or earthquakes, are not taken into consideration.

4 DISCUSSION

This algorithm deals with one time interventions during catch-up periods. The determination of optimal long term intervention strategies should be based on the minimisation of life-cycle costs. Strictly speaking this algorithm only works when all bridges are past their optimal time of intervention, although it may be used for approximations on very large networks when the number of protected structures is relatively small. A protected structure is one where the protective systems, such as paint, still function as intended.

This algorithm can only yield meaningful results if the appropriate costs for the appropriate interventions are determined. Although some initial work has already been performed in this direction (Sobjano et al., 2002, Saito et al., 1991 and Adams et al., 1998), the determination of more precise costs is essential for improved future predictions.

5 CONCLUSION

The use of age equivalents and binary programming can allow for the determination of optimal intervention strategies during catch-up periods.

The principal advantages of equating condition to age equivalents are:

1. Condition can be degraded every year by exactly one year for all different conditions.
2. Intervention costs, which change depending on bridge condition, can be linked directly to condition.
3. Bridge history, which is often lacking, can be neglected.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Bayesian regression modeling of concrete carbonation depth for inclusion in J-BMS

A.Tarighat
Shahid Rajaee University, Tehran, Iran

A. Miyamoto
Yamaguchi University, Ube, Japan

ABSTRACT: Deterioration modeling of reinforced concrete structures is the heart of almost any Bridge Management Systems. There are many models for prediction of concrete deteriorations depth, but almost all of them can only predict the output without any sense for uncertainty. Concrete deterioration models should be updated after inclusion of new data to existing databases. Since the collected data is uncertain one of the best modeling methods can be based on the Bayesian Decision Theory. This theory introduces an attractive method through combination of old even poor quality data or expert information based on experience with the new high quality data. In this paper a Bayesian regression model is introduced for concrete carbonation depth prediction as an example. It is found that this method of modeling with its framework can be a good candidate as an appropriate predictor for better decision making, maintenance and repair programming in Japanese Bridge Management System.

1 INTRODUCTION

Recently controlling and extension of service life of structures are becoming more important. This importance mainly depends on technical necessities and economical condition. On one side the number of concrete structures suffering from deterioration phenomena increases and on the other side there are many economical and budgeting limitations.

As an important element of a Maintenance and Repair Management System, prediction models are crucial for one or more of the following analyses: maintenance planning, budgeting, life-cycle analysis, multi-year optimization of maintenance works program, and improved design methods.

2 BAYESIAN REGRESSION METHOD

Predictive models require a large number of observations in order to provide a good coverage of all factors included over a reasonable reference space. Since the deterioration models are fundamentally a function of time, data needs to be available over a reasonable time span (10–20 years) for concrete structures or bridges of BMS being observed.

Bayesian regression method is different from the more conventional regression approaches. In essence, given the observed data, once the prior distribution and the statistical model are specified, Bayes' theorem is applied to derive posterior distributions of the parameters in the model or functions of these parameters (Gelman 1995). Given certain constrained assumptions, Bayesian regression develops a statistically optimal posterior multivariate regression model based on a defined prior and field model (Samprit 1991).

An emerging technique for concrete deterioration modeling is to incorporate expert opinion or old, poor quality and noisy data as "prior" knowledge in the new observed and controlled data. Bayesian statistics were developed specifically to cope with small and noisy sample data by providing a structured way to introduce prior information into the regression analysis. The main reason for selection of this modeling method in this paper is that the Bayesian approach allows the development of better models than be possible with data alone. Figure 1 shows the main idea of Bayesian regression modeling method.

3 MODEL DEFINITION

In this paper it is decided to propose a model to predict the concrete carbonation depth. This model is only for investigation about Bayesian regression method and might be used as a good candidate in JBMS to develop better scheduling of maintenance and repair activities.

4 DISCUSSION

Bayesian regression method is used to find the posterior model that is the prediction model in future. It is as follows:

$$d_c = 3.6731 + 11.3682 \times \frac{w}{c + csf} + 7.3898 \times \frac{csf}{c + csf} - 0.0994 \times RH\% + 0.357 \times t \tag{9}$$

4.1 *Sensitivity analysis*

A sensitivity analysis was carried out to compare the Data, Prior and Posterior models. The effect of changing the values for each independent variable is shown in Figure 8. As the line plot shows, predictions from the Prior regression are lower than the Data and Posterior functions. According to the plot, a change in the time variable has the greatest effect on concrete carbonation depth. As it is expected increase in the water to binders ratio increases the concrete carbonation depth. Increasing the condensed silica fume to binders ratio has little effect on predicted depth. In the case of relative humidity percentage, concrete carbonation depth in high ranges of RH% is low. It is a finding that supports some research facts.

It is obvious that even though the number of data is low the power of prediction is relatively high and for better predictions it is necessary to collect more precise data under controlled program of testing. From Table 4 it can be concluded that as the time increases the predictions are not accurate and variability increases. Therefore for better estimation of uncertainty and variability of concrete carbonation depth more data from old concrete structures is needed to enhance the model predictability.

5 CONCLUSIONS

The main objective of this paper is to develop deterioration models for Bridge Management Systems. The most important point of this study is the application of Bayesian regression employing expert opinion or old poor quality data to augment data-models. Based on this study some important conclusions can be derived as below:

1. It is found that the Bayesian regression approach is more meaningful than other methods of modeling approaches through the introduction prior knowledge to the modeling process.
2. As the most important result from this study, contrary to data mining methods by Bayesian regression modeling approach it is possible to include some prior knowledge or rule type information to model and enhance its predictability power and/or find the strength of the prior knowledge with further findings via the more data collected in future.
3. Since the Bayesian regression method is versatile and the prepared software for it is user-friendly it is strongly recommended that instead of construction of a model with many parameters and measurements for collecting data, it is better to have several simple models for different area with special environmental conditions.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Long-term monitoring of concrete bridges by direct combination of experimental and mechanical analysis

K. Brandes
Institute of applied Research in Civil Engineering (IaFB), Berlin, Germany

W. Daum & F. Buchhardt
Federal Institute for Materials Research and Testing (BAM), Berlin, Germany

ABSTRACT: The more and more common task of bridge monitoring demands a mathematically based method for the symbiosis of all the measured data in view of the structural system.

The aim of the newly developed method was the combination of all measured data in one physical (mechanical) system which is based on the structural system of the bridge. Of great importance was the easy understanding of the method, and this was the prior reason for starting from the *Finite Element Method.* The different steps of solving the problem are:

- mechanical modelling (e.g. beam bending theory for simplicity)
- discretization (finite elements)
- mathematical modelling by applying Hermitian Polynomials (of third order) as shape functions
- the Gaussian least squares method as related variational principle which allow to add the constraints from the structural system as side equations with Lagrangian Multipliers
- of basic importance is the separation of the sum of least squares of the different physical (mechanical) quantities because they have no common metric. Only after the regression formulation for the different quantities, the coupling with respect to the mechanical system can be performed.

As a consequence of an only limited number of measuring points and measurement data, the number of parameters which have to be established should be as small as possible.

In this paper, the treatment of the problem will be limited to bridges as one-dimensional structures. Additionally, only the long-term settlement and heaving of the supporting points will be taken into account which simplifies the treatment of the problem. To establish a further simplification, a period of time of about two years will be considered and the position at the beginning and the end of that period is compared. The temperature at the first day and that of the last day are nearly the same. No deformation due to temperature changes had to be taken into account. For the complete problem, the formulation is quite more complex and will be avoided in this presentation.

At the bridge, vertical displacements as well as strains at several points have been measured, and their interrelation will be evaluated.

The new method starts from two basic principles:

- the Method of Finite Elements for discretizing the structure and
- the Variational Principle of Least Squares as introduced by Gauss at the beginning of the 19th century extended by constraint conditions.

The continuous bridge beam is divided into beam elements of span length which represent different domains. For each of the domains, a set of functions with parameters is introduced to approach the deformation. For different physical quantities (e.g. displacements and strains), different functions have to be chosen which should satisfy the mechanical conditions of the theory of beam bending.

The best fitting of the measured values of the deformation is reached by applying the *least squares* method. First, *error equations* are formulated for each measurement point for any domain

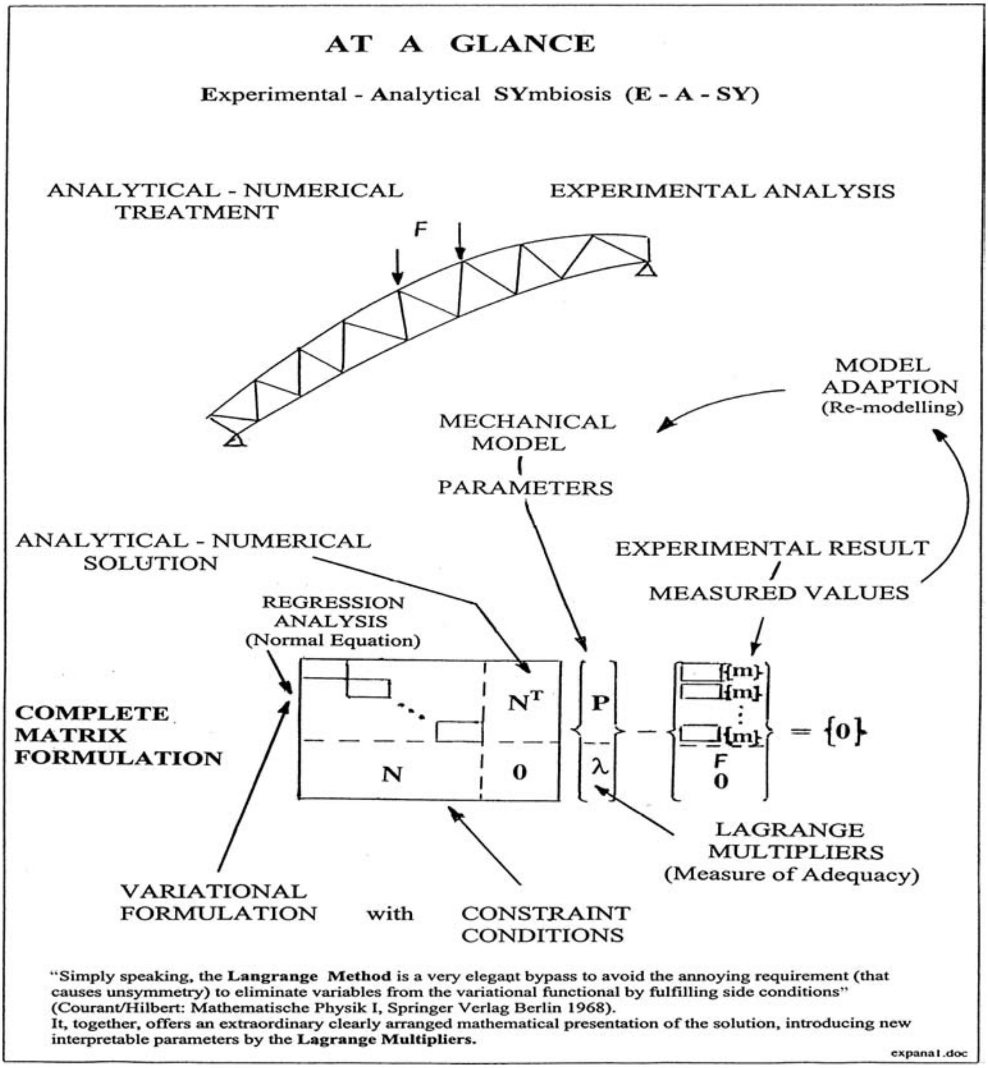

Figure 1. Brief scheme of the method of combining experimental evaluation and physical/numerical analysis.

and for any physical quantity. The interrelation of the (local) parameters in any domain to global parameters is given by the mechanical system. They are formulated as *constraint equations* which have to be multiplied by *Lagrangian Multipliers* according to the well known Lagrangian Multiplier method of Variational Calculus.

In the second step, the *error equations* will be Gauss transformed resulting in the *normal equation* of regression analysis which are accompanied by the constraint equations. The number of parameters is extended by the Lagrangian Multipliers which give some hints to the quality of the adequacy of mechanical model and measurement. The resulting matrix equation is the largest one possible in stating the problem, however, is very simple to survey (so called Complete Matrix Formulation).

For the bridge under consideration, some results will be presented and interpreted. The figure will give a first idea of the interrelation used in the treatment.

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Development of a web-based database system for management of existing bridges in the Yamaguchi prefecture, Japan

K. Kawamura
Yamaguchi University, Ube, Yamaguchi, Japan

D.M. Frangopol
University of Colorado, Boulder, Colorado, USA

A. Miyamoto
Yamaguchi University, Ube, Yamaguchi, Japan

ABSTRACT: Bridge management based on recent theoretical developments has become a major social concern in Japan. The Japanese central and local governments are deeply concerned about the management of existing bridges. This is because the number of deteriorated bridges is increasing due to increasing volume of traffic, increasing weight of road vehicles, and structural aging. However, the budget limitations of local governments for bridge maintenance actions and new construction are much more severe than those of the central government. Therefore, local governments are deeply concerned about the management of their bridges.

The Yamaguchi prefectural government is one of the leading prefectures interested in developing a practical management method and a computerized system for road infrastructure (Miyamoto et al. 2000). The BMS for the Yamaguchi prefecture is referred to as the Japanese Bridge Management System (*J-BMS*). The system consists of three main subsystems: performance evaluation system based on two indicators: durability and load carrying capability (Kawamura et al. 2003, 2004a), maintenance planning system (Miyamoto et al. 2001, Kawamura et al. 2004b), and bridge management database system.

Figure 1 shows the *J-BMS* framework. The performance evaluation system and maintenance planning system enable performance evaluation, deterioration prediction, and maintenance planning for main girders and slabs of reinforced concrete bridges. The database system stores data such as technical specifications, traffic volume, inspection results, and maintenance plans of bridges. Then, the database system provides some input data to the other subsystems.

The performance evaluation system (Kawamura et al. 2003, 2004a) evaluates the performance of bridge elements using the inspection data and technical specifications which are stored in the database (see ① and ② in Figure 1). The main outputs of the evaluation are soundness scores for two performance indicators: load carrying capability and durability, which are given on a scale varying from 0 to100. These two indicators are used as measures to consider the necessity for maintenance. Specifically, load carrying capability is used as an index to estimate the necessity of strengthening, and then durability is used as an index to estimate the necessity of repair. The system categorizes the soundness into one of the following five categories: "safe," "mild deterioration," "moderate deterioration," "severe deterioration," and "unsafe." A detailed inspection is performed on bridges that are classified into the categories "unsafe" or "severe deterioration" by the system. An appropriate maintenance method of repair or strengthening is identified using results of detailed inspection including non-destructive testing.

Based on the results of the performance evaluation system, the maintenance planning system (Miyamoto et al. 2000, 2001) estimates the present degree of deterioration. The remaining service life of the bridge is predicted using a deterioration prediction function (see ③ in Figure 1). If the remaining service life, calculated by the maintenance planning system, does not exceed the

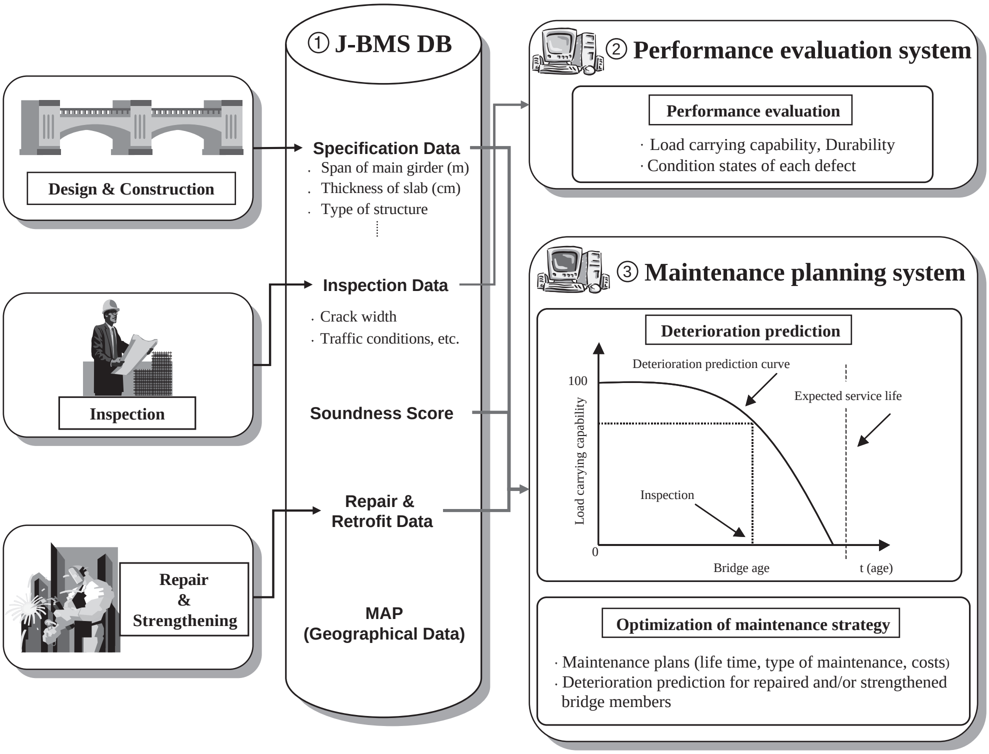

Figure 1. J-BMS framework.

expected service life of the bridge, a maintenance strategy is generated by considering the costs and effects of repairs and strengthening.

The strategy includes various maintenance plans based on cost minimization or benefit maximization.

After briefly reviewing the outline of each subsystem of *J-BMS*, this paper presents the web-based bridge management database system for the Yamaguchi prefecture, Japan. Several screen shots of the computer program are also presented in order to demonstrate how this system performs.

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Loads and testing

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AASHTO-LRFD live load distribution: Limitations and applicability

Z.G. Yousif
Earth Tech Canada Inc, Calgary, Alberta, Canada

R.A. Hindi
Bradley University, Peoria, Illinois, USA

ABSTRACT: The AASHTO-LRFD (2004) was initially calibrated by trial design to provide a high and uniform level of safety in new bridges. The safety level is expressed by a reliability index (β). AASHTO-LRFD provides a uniform reliability index (β) of 3.5 for different type and configurations of bridges. This reliability index ($\beta = 3.5$) ensures that only 2 out of 10,000 design elements or components will have the sum of the factored loads greater than the factored resistance during the design life time of the bridge.

The current AASHTO standards (1996) (LFD) do not provide a uniform safety level and the reliability index β can be as low as 2.0 or as high as 4.5. If it is equal to 2.0, 4 out of 100 design elements and components would probably be overloaded and would experience a problem during the design life time of the bridge. Based on that, the requirement for the AASHTO-LRFD specifications was necessary.

The major change in the AASHTO-LRFD is the distribution of vehicular wheel load on highway bridges which is considered to be a key response quantity in determining the bridge component size and detail, consequently, strength and serviceability, which are considered as a vital issue concerning the safety and economy of highway bridges. Therefore, it is of critical importance in designing new bridges and in evaluating existing bridges.

Since1930s, the AASHTO simple S/D formula has been used for live load distribution factors in most common cases to calculate the bending moment and shear in bridge design. Where S is the girder spacing and D is a constant that depends on the type of the bridge superstructure and the number of the design lanes loaded. This formula allows the designer to simply calculate the part of live load to be transferred to the girders without any consideration for the bridge deck, girder stiffness and span. Furthermore, some bridge designers apply the above mentioned formula even to more complicated bridges such as skewed, curved, continuous and large spans with wide and different girder spacing, eventhough, the formula is developed for simple bridges with typical geometry. Therefore, these bridges will be constructed either in a conservative way which involves the unnecessary additional cost or unconservative way which is related to bridge service life and safety.

In 1993, the National Cooperative Highway Research Program (NCHRP) developed new live – load distribution factors "Project 12–26 – Distribution of live load on highway bridges". Additional parameters were included in the new formulas to obtain more accurate distribution factors, such as bridge span (L), slab thickness (t_s), girder spacing (S) and the longitudinal stiffness parameter (K_g). The first edition of AASHTO-LRFD (1994) specification was based on this study.

In bridge industry, the precast prestressed concrete I girders are the most common type of bridges due to the durability of concrete, flexibility in construction and the speed of construction. Therefore, a very high percentage of bridges in the United stated are cast in place concrete deck with precast prestressed concrete girders (beam-and-slab bridges). The girders are mainly AASHTO-PCI concrete girders (Types I to VI).

Although it has been concluded that AASHTO-LRFD is less conservative than the standard AASHTO (1996), it has also been shown that the AASHTO-LRFD is conservative for specific bridge parameters and geometries compared to several refined methods of analysis.

The current AASHTO-LRFD (2004) imposes limits in terms of range of applicability on the use of its live load distribution factors for design of highway bridges. These limits are specified in terms of bridge span, slab thickness, girder spacing, and longitudinal stiffness. In order to ensure the safety of highway bridges, these limits have to be studied and verified.

This paper summarizes the live load distribution factors for simple span highway bridges using the 2004 AASHTO-LRFD and finite element analysis. The objective of this study is to verify the applicability of the limitations specified by AASHTO-LRFD in order to ensure the safety of highway bridges. This study focuses on bridge with deck on concrete girders (AASHTO-PCI Types I to VI). The research covers wide range of several parameters; span, deck thickness, and girder spacing for each girder type in order to study the effect of their limitations as specified by AASHTO-LRFD. Several three dimensional linear elastic models were built using SAP2000 and compared to obtain the most accurate method to model the girders, deck and their connections. The bridge deck was modeled using quadrilateral shell elements and the concrete girders were modeled using space frame elements. HL-93 (HS20) truck plus the uniform lane load as specified by AASHTO-LRFD were used in the analysis. This research showed a reasonable agreement between AASHTO-LRFD and the finite element analysis. This study also showed that the AASHTO-LRFD is on the conservative side in terms of the limitation range specified for the span length, deck thickness and girder spacing.

Based on the results of this study, the following findings were concluded:

1. For exterior girder of bridges subjected to one lane loaded, the LRFD/FEA ratio is 1.56 and the major difference was for spans longer than 31 m.
2. For exterior girder of bridges subjected to one lane loaded, the increase in longitudinal stiffness (K_g) reduced the LRFD/FEA ratio to an average of about 1.18 in long spans and 1.10 for short spans.
3. For exterior girder of bridges subjected to multiple lane loaded, the AASHTO-LRFD results were within a range of less than 10% compared to the FEA results for most of bridges spanning between 31–73 m. While for bridge spans ranged between 6–31 m, the LRFD results were conservative with a range of 10–30%.
4. For interior girder of bridges subjected to one lane loaded, the following was observed:
   - LRFD and FEA produced similar results for slab thickness and girder spacing ranging between 110–190 mm and 1100–2400 mm, respectively.
   - LRFD overestimated the distribution factors by 10–30% for slab thickness and girder spacing ranging between 190–300 mm and 2200–4900 mm, respectively.
   - The increase in longitudinal stiffness (K_g) reduced the difference significantly for short spans and had less impact on long spans.
5. For interior girder of bridges subjected to two lanes loaded:
   - LRFD and FEA produced similar results for slab thickness and girder spacing ranging between 190–240 mm & 2200–2940 mm, respectively.
   - For spans less than 31 m, the difference between LRFD and FEA is 5–18%.
   - For spans ranging 31–73 m, the difference between LRFD and FEA reached 15–30%.
6. The FEA results demonstrated that not necessarily the distribution factors obtained from two lanes loaded will govern the design compared to three lanes loaded as proven for span larger than 31 m with slab thickness and girder spacing ranging 190–240 mm and 2200–2940 mm, respectively; therefore, EFA is recommended for bridges subjected to three lanes loaded.

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Numerical model for bridge-vehicle interaction and traffic-induced vibration investigation

H. Moghimi
NAMVARAN consulting engineers, Tehran, Iran

1 INTRODUCTION

The undesirable perceptible vibration of the composite steel girder bridge under live loads is one of the most important problems in the bridge industry especially with current trend towards longer span and lighter deck system which combined with limited damping due to nature of the system. Because of their structural form, being wide but shallow in depth, bridges are predominantly susceptible to vertical vibration. Although the strength design and the deflection control criteria of these bridges are done fairly well in accordance with the bridge design codes, the vehicles movement on the bridge may still cause vibrations that would seem undesirable from the viewpoint of the pedestrians walking or standing on the footpath. Essentially, the bridge vibration under the effect of traffic movement should be limited to satisfy two basic purposes. First, reducing ratio of dynamic stresses to static stresses (this feature be considered in static design by dynamic load factor or dynamic load allowance). The second reason is that too much vibration may cause fear to those who use the bridge and undermine confidence in the structure.

This investigation has presented comprehensive numerical methods to study bridge-vehicle interaction. 3D finite element models are developed for trucks, road surface (roughness) and composite girder bridge. Truck parameters include the body (mass), suspensions and tiers and its random variables include the total weight, and speed. A bridge is treated as actual 3D composite steel girder bridge with simply supported span and with its elastomer support at each pier. Deformations are assumed to within the elastic range. Dynamic modulus of elasticity of concrete is used for RC slab of composite steel girder bridge.

Parametric study is done to identify the effect of various parameters on the dynamic load and consequently on the vibration of composite girder bridge, such as vehicle speed, stiffness of neoprene, effect of aspect ratio (ratio of height to length) of steel girder of composite deck, type of vehicle, integrity of deck slab at pier and initial bounce of the vehicle due to road surface roughness. The results of each parametric study have been expressed in form of human perceptibility curves (graph of perceptible vibration acceleration versus vibration frequency) and then the results are compared with base curve for acceptable human response to vibration of bridge under frequent form of loading.

2 CONCLUSION AND RECOMMENDED DESIGN CRITERIA

The finite element analysis results indicate that the vibration is correlated with the velocity of the truck. The bridge response is significantly influenced by vehicle speed, i.e. as the vehicle speed increase, dynamic deflection and acceleration of deck increase while vibration frequency has been little affected and it means grater level of vibration perception. Another important parameter on the deck vibration is ratio of vehicle weight to total weight of super structure. It has been found that when the ratio increases above ten percent, vibration acceleration will be highly increased, while the response frequency remains constant.

The vibration of sidewalk is affected by change in vehicle passage lane. It is found that eccentric vehicle loading increases the amount of deck vibration in footway depending upon the distance between the bridge center line and the line of vehicle loading. In the meantime, the torsion and transverse bending deformation of the deck increase. This effect is insignificant under the cumulative response of initial bounce and vehicle velocity.

Also the simulated vibrations indicate that considering some features in composite girder bridge design have major advantages on reducing perceptible vibration. The stiffness of neoprene has a considerable effect on the perceptible vibration of the composite bridge. If the area of the neoprene is selected based on high permissible stress or thickness of the elostomer support is chosen greater than required value, a serious increase in the desk vibration is induced.

The aspect ratio (height to length ratio) of steel girder has a direct effect on the deck system rigidity and as a result, on perceptible vibration in deck system. As a general statement, the higher natural frequency of bridge deck, the less perceptible vibration on the sidewalk. Therefore using box section or I section with greater aspect ratio than 1/20 is a rational strategy for mitigation of vibration in the footway.

Reduction of deck joints to minimum required value and continuous construction of the RC slab at pier of simple span composite steel girder bridge has important advantage in vibration perceptibility problem. The results indicate that this practice leads to great decrease in vibration acceleration, especially at the beginning of the span, and moderate increase in vibration frequency. Then lower vibration will induce in the footway. In other word, RC slab integrity in simply supported span causes the vibration of the beginning of the span damp in large extent.

It has been found that the last item has another positive effect on bridge vibration problem. Due to poorly backfilled of pavement adjacent to deck joint, after a short time, it acts as a bump and when the vehicle is passing on them, a large deflection with much initial bounce, which is correlated with speed and weight of vehicle, will generate in the vehicle. Although the initial bounce of vehicle are damped by vehicle's suspension system within a short time, these initial bounce can still produce high initial vibration in the deck, particularly in the beginning of span where vehicles still have considerable kinetic energy. It is important to note that the increase in the vibration of the deck does not correlate with the initial bounce of the vehicle and it is vehicle dependent. In general, the initial bounce which causes resonance in the vehicle will cause highest vibration acceleration in the bridge deck system as well. Furthermore, instead of simple span bridges, continuous spans with non-periodic spacing can lead to more efficient structures with reduced vibration response to vehicle passage.

Another objective of present study is investigation of efficiency of code advised limitations on deflection to control perceptible vibrations of bridge deck. The investigation indicate that deflection limitation of design code is not a safe way for vibration control of composite steel girder bridge and some checking criteria to control deck vibrations has been proposed. It is found that aspect ratio of the steel girder equal or grater than 1/20 represents reasonable vibration performance in composite girder bridge under movement of highway load. Development of Ellingwood criterion for vibration control of commercial environment is represented as a second criterion to prevent unpleasant footway vibration. Field observation has been done on a bridge located on the downstream of Karkheh River dam, which has significant vibrations under moving truckloads and it validate the proposed interaction model and numerical results.

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The probability of extreme load effects in bridges subject to dynamic vehicle-bridge interaction

P.H. Rattigan, E.J. Obrien, A. González & N.K. Harris
University College Dublin, Earlsfort Terrace, Dublin, Ireland

ABSTRACT: The ability of a bridge structure to perform adequately under the action of extreme loading events is a primary concern both in the design and maintenance of the infrastructure network. Thus it is of importance that the total effect exerted by heavy traffic flow is suitably modeled and analysed.

Codes for the design and assessment of bridges provide traffic load models that give conservative traffic load effects. These notional traffic load models incorporate an allowance for Dynamic Amplification Factor (DAF) that is not generally related to worst static case scenarios and as such over-compensates for the effects of dynamics. These effects can be considerably reduced through the use of Weigh-In-Motion (WIM) data – actual traffic data recorded on live carriageways – and statistical modeling of the recorded traffic.

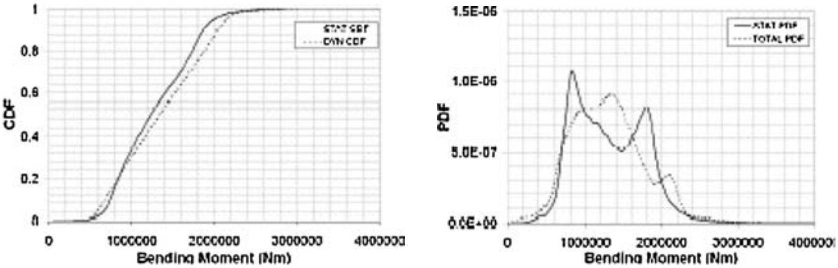
A convolution or integration approach applied to theoretical models may be used to obtain a probability distribution for total bending moment. This integration approach has the advantage of including all possibilities, and is not dependant on Monte Carlo simulation which can result in considerably variable results. The distributions of the random variables included in the problem are modeled based on measured WIM data, and applied initially to obtain a worst static load case. The variable distributions are then applied to a dynamic model in order to obtain a worst possible total (dynamic + static) load case. Comparison between the total and the static results yields a value for assessment/design DAF, outlined in Equation 1.

$$DAF_{Characteristic} = \frac{M_{Characteristic\_Total}}{M_{Characteristic\_Static}} \tag{1}$$

The procedure is initially applied to 25 m span bridges subjected to the crossing of a single 5-axle vehicle. To allow the problem to be illustrated graphically, an example is considered with only two design variables, truck velocity and truck gross vehicle weight (GVW). A convolution of all possible solutions is proposed, formed by all possible locations of the truck on the bridge, all truck crossing velocities and all possible truck weights. The principle can be readily extended to more complicated problems with more variables or bridges with more lanes etc. The truck length, axle spacings, weight distribution etc., are kept constant for the example problem with the only two design variables being vehicle velocity and GVW.

Based on WIM data representative of European traffic, the integration method is used to generate a distribution for static bending moment, and from this distribution a critical value can be obtained. From this worst static effect, a region of design space is now defined and isolated for analysis using a more time consuming and computationally expensive dynamic model. This method allows for the critical total (dynamic + static) load effect to be obtained with an acceptably low probability of occurrence. As the 1 in 1000 year GVW (which is proportional to static bending moment) has previously been obtained it is obvious that a truck of less than the 1 in 1000 year static GVW value can only contribute to critical events if the trucks associated DAF is significantly large to raise the total bending moment sufficiently.

It is found that the highest DAF values in the chosen design range will occur at low GVW values and hence are not important in analysis of the worst total bending moment. By using the critical



(a) (b)

Figure 1. (a) CDFs of static and total bending moment (b) PDFs of static and total bending moment.

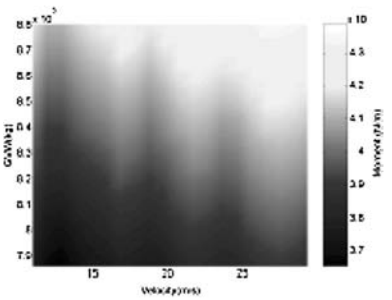


Figure 2. Variation in total bending moment with velocity and GVW for refined design space. Static and total CDF tails.

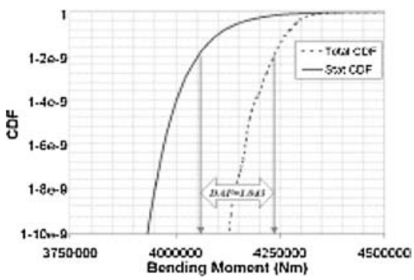


Figure 3. Static and total CDF tails.

static load effect to identify a critical GVW value, and combining this GVW value with a maximum permissible and a minimum permissible DAF value it is possible to define a range in which a more refined dynamic analysis is performed. Figure 2 presents the total bending moment response for all velocity/GVW combinations in the refined design space. By examining the probabilities of occurrence in this critical design space the tails of static and total CDFs are obtained and can be seen in Figure 3. Characteristic values for critical static and critical total bending moment are obtained and application of Equation 1 yields a value for characteristic DAF.

This paper presents a method whereby the use of measured WIM data can be efficiently used in conjunction with dynamic bridge-truck interaction models to develop a characteristic critical total load effect for the chosen bridge, as well as a relevant and accurate value for bridgespecific DAF.

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NCHRP project 20-07/task 122: Load rating by load and resistance factor evaluation method

D.R. Mertz
University of Delaware, Center for Innovative Bridge Engineering

ABSTRACT: The Draft Manual for the Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges was developed under NCHRP Project 12-46. The manual, in particular the section on load rating, has been extensively reviewed by Technical Committee T-18, Bridge Management Evaluation and Rehabilitation. As a result of this review, several changes were made to Section 6, Load and Resistance Factor Rating (LRFR). The revised version was adopted as a Guide Manual at the May 2002 Meeting of the AASHTO Subcommittee on Bridges and Structures (SCOBS).

The LRFR methodology is based upon calibrated load factors using the principles of structural reliability. Further, the LRFR procedures were subjected to trial analyses as part of the development. Nonetheless, there exists a need to further demonstrate that the method gives valid, consistent results for all major bridge types and span ranges, and to document and explain differences between LRFR and current load factor ratings (LFR) results. Improved validation and comparisons are necessary for LRFR to be accepted by state DOT's before they will be willing to accept LRFR for bridge sufficiency evaluation, load posting and overload permit approval analysis.

The objective of this project is to provide explicit comparisons between the ratings produced by the LRFR methods of the Guide Manual and LFR ratings from the latest edition of the AASHTO *Manual for Condition Evaluation of Bridges*. The comparisons are based upon flexural-strength ratings. For girder-type bridges, the rating comparisons further concentrate on the interior girder.

A matrix of 145 example bridges was developed by the project panel to provide an appropriate cross-section of bridge types to be utilized in the rating comparison. The data matrix was to be extracted from a subset of those bridges already coded into the Virtis database. Any omissions were requested from other State DOTs that were compiling Virtis data for their bridge inventory. NYSDOT had an extensive number of bridge systems coded into the Virtis database, and 97 of the possible 145 example bridges were available. WYDOT identified 20 candidate bridges coded as girder lines in BRASS-GIRDER™.

Globally, the sampling of example bridges suggests that the design-load inventory rating factors by LRFR are generally greater than the corresponding inventory rating factors by LFR, while the design-load operating rating factors are lower. Only in the case of reinforced concrete slabs are the LRFR factors less than the LFR factors for both the inventory and operating ratings.

The design-load levels are different for LRFR and LFR: The design-load level for LFR is HS20 weighing 36 tons, while the HL-93 design load for LRFR is a notional load. (In other words, it does not "look" like a simple truck with a specified tonnage.) However, in the original development of the HL-93 load model, a truck-type live-load model, the HTL57, which produced similar moments and shears, was proposed. The HTL57 was longer than a traditional HS20 but weighed 57 tons. There is no simple relationship to relate the LRFR design-load rating factor to the LRFR design-load rating in equivalent tons, but a simple approximation would be to multiply by the 57 tons of the HTL57. (This approximation is more appropriate for longer spans where the configuration of the truck is less significant than the weight.) Thus, an LRFR design-load rating factor of 36/57 or 0.63 could be simplistically considered equivalent to an LFR rating factor of 1.00 for a longer

bridge. A better comparison of ratings would be to compare an equivalent HL-93 rating in tons to the HS20 rating in tons.

The reliability of the example bridges was established through a Monte Carlo simulation that compares two distributions of values – in this case, load and resistance – and determines a random value of resistance minus load for a given design criterion, in this case, the Strength I limit state for flexure. The resultant value is independent of the design methodology employed in the design of the bridge, as a probable resistance is compared to a probable load without regard to the design methodology.

For each sample bridge, 1,000,000 Monte Carlo simulations were made. When the Y_i value of step 9 is negative, the simulation has resulted in a "failure" of the limit state (not necessarily a structural failure, but a failure to satisfy the design or rating criteria). For relatively safe bridges ($\beta > 4$), a significant number of failures (greater than 10) will not occur in 1,000,000 simulations.

Of the 74 bridges in the database, 26 demonstrated a failure rate of more than 10 failures out of 1,000,000 simulations. The other 48 bridges yielded no significant number of failures of the design criteria in 1,000,000 simulations. These bridges were not investigated further. With rating factors in excess of about 1.5, the assumptions inherent in the design and rating procedures become suspect. Suffice it to say that such bridges are safe enough (with LFR or LRFR). Bridges with ratings near the design point are more telling.

The conclusions and recommendations from this study are narrowly based upon the scope. Only flexural strength ratings were made, and these ratings were made by the BRASS™ programs. The author assumes that the ratings as produced by BRASS™ are correct. The investigation of reliability made using Monte Carlo simulation suggests this to be true. Based upon the results of this investigation, in general, LRFR rating factors are equal to or greater than LFR ratings factors except for reinforced-concrete slab bridges. These types of slab bridges may represent a problem in terms of LRFR rating. As was demonstrated, the author believes that the lower slab bridge ratings are technically appropriate. The effect of these low slab-bridge ratings on operating a bridge system must be assessed by the owners.

This limited study suggests that LRFR is technically sound, with the LRFR rating factors in good correlation with the failure rates. In other words, LRFR rating factors lower than one demonstrated relatively high failure rates. LFR ratings did not correlate well. In fact, many bridges with LFR rating factors above one demonstrated unacceptably high failure rates. This is not to say that the continued use of LFR rating is necessarily unsafe, just irrational.

Questions about LRFR versus LFR for force effects other than moment and limit states other than strength are not answered. Nonetheless, the author recommends adoption of the LRFR methodology for rating bridges. Assuming the LRFR calibration process is sound, comparable results should result for other more extensive studies. The service limit states that are uncalibrated and optional in LRFR need additional thought.

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Investigating truck load effects using bridge weigh-in-motion system

E.-S. Hwang, J.J. Lee & S.H. Jang
Kyunghee University, Yongin, Korea

I.Y. Paik
Kyungwon University, Songnam, Korea

1 INTRODUCTION

Statistical data of traveling vehicle should include the information about total weight, number of axles, axle weight, axle distance, vehicle distance, vehicle velocity and traffic pattern, etc. In order to obtain the information, static measurement method, in which the traveling vehicle should stop to be weighed, could be used. But the traffic is severely interrupted in crowded and speedy highways and such information as vehicle velocity, vehicle distance and traffic pattern cannot be obtained by static method. Bridge weigh-in-motion (BWIM) system can overcome the shortcomings of the static method by collecting data while vehicles are in motion.

2 BRIDGE WEIGH-IN-MOTION (BWIM) SYSTEM

Hardware of the BWIM system is composed of piezo sensor, loop sensor, strain gage, signal transforming and acquisition device. The piezo sensor and the loop sensor are embedded in the pavement and measure vehicle velocity, number of axles and axle distances. The electrical strain gages are attached at the bottom surfaces of the middle of the steel girders to measure the strain when the girders are loaded by traveling vehicles. Weight of the vehicle is calculated by using the strain gage signal. Axle weight is distributed based on the piezo sensor signal.

3 MEASUREMENT AND ANALYSIS OF DATA

PC is installed inside of the steel box girder of Maebong Bridge equipped with BWIM system. Figure 2 shows the signal from piezo sensor and strain gage of the two lanes. Weight of the vehicle is calculated by using the strain gage signal. Axle weight is distributed based on the piezo sensor signal. Analysis is conducted on the data collected during 14 days in April and August. Total number of vehicles passing the bridge during the period is about 250,000 and about 71,000 is heavy vehicle.

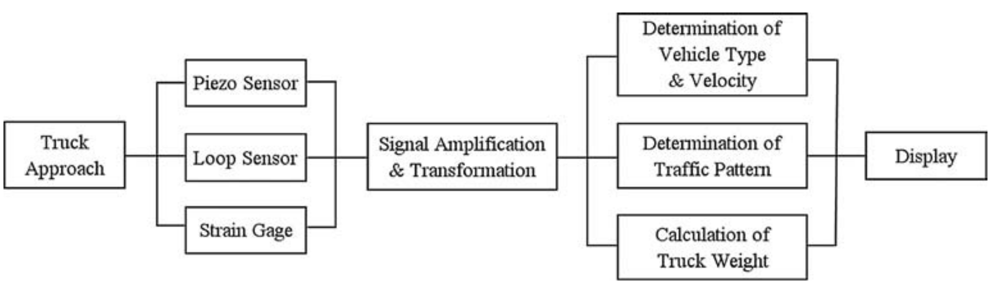

Figure 1. Schematic diagram of BWIM system.

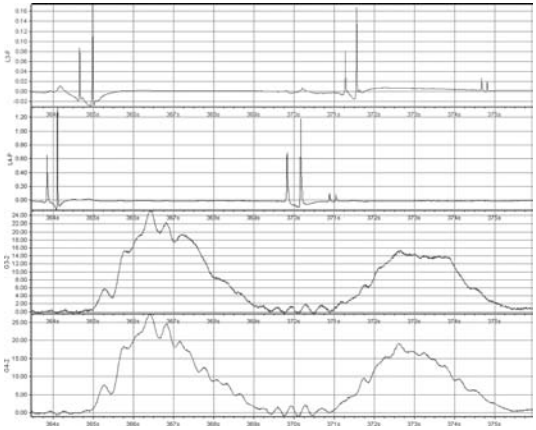

Figure 2. Piezo sensor and strain gage signal of two lanes.

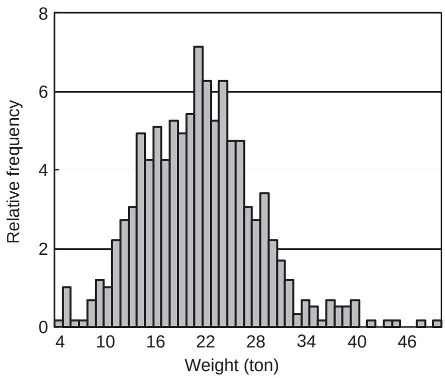

Figure 3. Relative frequency diagram of type 50 vehicle weight.

Traffic Pattern	Relative Frequency	Traffic Pattern	Relative Frequency
LANE 1 / LANE 2	78.4%	LANE 1 / LANE 2	1.4%
LANE 1 / LANE 2	15.0%	LANE 1 / LANE 2	1.8%
LANE 1 / LANE 2	3.3%	LANE 1 / LANE 2	0.1%

Figure 4. Relative frequency of traffic pattern.

Average daily traffic of heavy vehicle is about 5,000. Data of 7 hours and 12 minutes are analyzed for the weight calculation and relative frequency of the vehicle weight for the vehicle type 50 which represents 3 axle trucks is plotted in Fig. 3. Most of the vehicle weight is between 10 ton and 30 ton for this type of vehicle but there is significant number of trucks which weighs over 40 ton.

Total number of 71,000 heavy vehicle data is used for the traffic pattern analysis and the result is plotted in Fig. 4. Single vehicle traffic over the bridge at any time is 78.4% of the total and single vehicle with additional vehicle following in the next lane comprises 15%.

Maximum probable weight of heavy vehicle during a design period of a bridge could be estimated by plotting inverse normal distribution plot and extrapolation for the expected years.

4 CONCLUSION

In order to collect data to model the representative truck for a new bridge design code, BWIM system is installed to a highway bridge. By processing and analyzing the measured data, traffic pattern on a bridge, weight distribution between axles and the distances between axles of the vehicles could be determined and the configuration of the currently operating truck could be figured out. Based on the field measured data and the analysis of the data, the weight and configuration can be predicted for the truck which constitutes the main load for the design of short to medium span girder bridges.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Design temperature load models for concrete slab bridges

E.-S. Hwang & J.J. Lee
Kyunghee University, Yongin, Kyeonggi-do, Korea

The purpose of this paper is to propose new temperature load model for concrete slab bridges through experimental study and analysis. For the experimental study, long-term measurements of temperature distributions along the height of slab have been performed on a rahmen type slab bridge. Temperatures are measured on a section of a newly constructed slab bridge having the slab thickness of 90 cm. 10 thermistors were installed on each of two locations at midspan. Measurements have been carried out every hour from September 2001 to June 2005.

From the measured temperature data at each sensor, average temperature and linear vertical temperature difference of the section can be calculated and plotted in Figures 1 and 2.

To estimate the maximum values of thermal loading parameters during the bridge lifetime, extreme analysis is carried out. 160 extreme values are plotted on the Gumbel probability paper and distribution types of all four parameters are determined as Weibull distribution type. Then optimal parameter values of Weibull distribution and the extreme values of thermal loading parameters corresponding to 100 years lifetime are estimated as shown in Table 1.

The average temperature for the concrete slab bridge is proposed as the variation from −10°C to 36°C. This value is similar to design values for the bearing movement of the current Korea bridge design code and for Eurocode.

To develop the load model of the vertical temperature gradient, vertical temperature distributions of several days showing large linear vertical temperature difference are plotted. It is assumed that the temperature is constant from the bottom of the slab to 0.45 m height. Also it is found out that the temperature distributions from 0.45 m height to the top of the slab can be fitted to the second-order polynomial function as shown in Equation (1)

$$T(y) = 21 \times \left(\frac{0.45 - y}{0.45} \right)^2 \tag{1}$$

where, $T(y)$: vertical temperature gradient function
y: distance from the top of the slab ($0 \leq y \leq 0.45$ m)

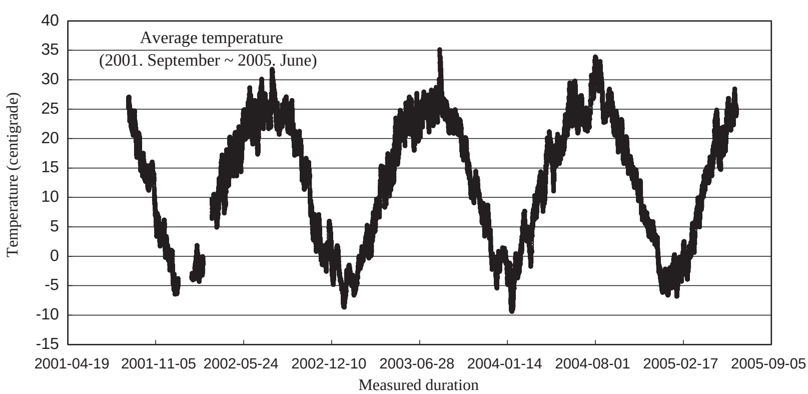

Figure 1. Variations of average temperature during the measured period.

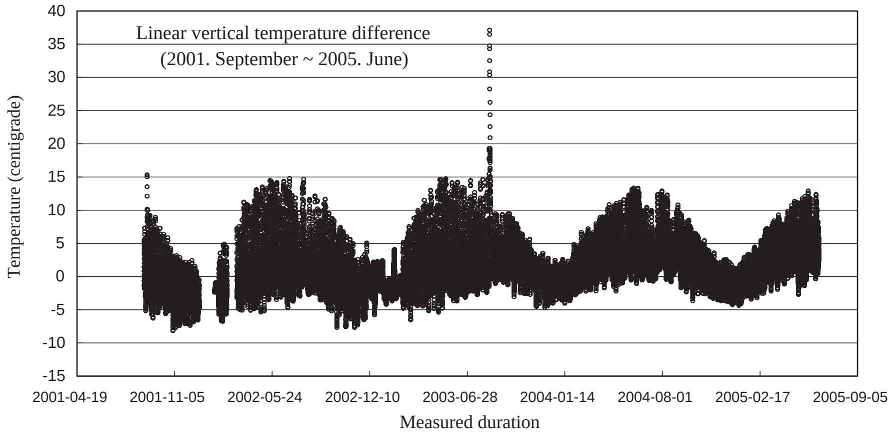

Figure 2. Variations of linear vertical temperature difference during the measured period.

Table 1. Weibull Distribution parameters and extreme values corresponding to 100 years return periods.

Thermal loading parameters	Distribution parameters λ	γ	δ	Extreme values (centigrade)
Maximum average temperature	35.9	3.4232	18.81	35.55
Minimum average temperature	−9.90	2.9578	11.91	−9.78
Maximum linear vertical temperature difference	15.9	2.9022	18.30	15.74
Minimum linear vertical temperature difference	−7.90	3.5297	8.29	−7.73

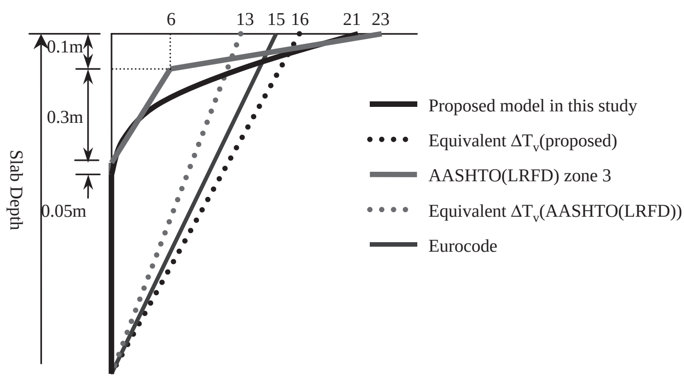

Figure 3. Comparison of proposed load model for vertical temperature gradient with various codes.

Figure 3 shows the proposed model, equivalent linear temperature difference, and provisions of AASHTO(LRFD) and Eurocode. To improve the validity of the proposed model, theoretical studies should be done for different slab thicknesses.

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Prediction and influence of future traffic demands on Croatian highway bridges

A. Mandić & J. Radić
Faculty of Civil Engineering, University of Zagreb, Zagreb, Croatia

ABSTRACT: Bridges are crucial points of the road traffic network. Great number of Croatian bridges like in other countries is designed using old codes. To reach the same level of safety, serviceability and durability of bridges on all traffic corridors, it is necessary to adopt temporary standards in their design. Croatian codes for bridge loading will be based on European codes with national load specifications. In this paper, the reliability of bridges, designed using previous codes, present Croatian and foreign codes is estimated.

Development of high quality roads, highways and so called semi-highways, started in seventies of the 20th century and gather momentum in the last decade. Road traffic develops rapidly. To establish efficient and rational road and bridge management system a counting of traffic started in the year 1973. Namely, road traffic information is necessary for investigations, planning, design, building, reconstruction and maintenance of roads and bridge structures on those roads.

Since the year 1949 Croatian bridges were designed using PTP-5 today so called – old code. In seventies of the 20th century, it was established that this code is out of date so designers started to use German code DIN 1072. Still, old code was valid since the year 1991 when Croatian code, valid until now, inured.

Time variability of actions and an innovative approach to safety, serviceability and durability of structures are leading to the necessity for modernizing the Codes for bridge design. Croatian roads will soon have to accept traffic of European communications. These European traffic situations are represented by a series of load models in the European standard EN 1991-2, Road traffic actions.

For the realistic traffic simulation, data of the year average daily traffic at the competent location with the largest number of heavy vehicles are used. The traffic is composed of cars and seven observed heavy vehicles types set in four categories with corresponding frequencies in transport. Due to inappreciable influence of cars, they are excluded from the load modeling. First category – buses and trucks <7.0 t is presented with the model 1, trucks ≥ 7.0 t are presented with two models – model 2 with the overall weight of 150 kN and model 3 with the overall weight of 250 kN. Tug trucks are presented with two vehicle models – 4 and 5, and trucks with trailers with model 6.

Random arrangement of 1000 vehicles in the 13,000 m long motorcade is used. Vehicles are modeled with axle load as concentrated forces at familiar axial indent. A traffic density is assumed with the variable vehicle distance r.

Comparisons of the influences due to standardized load models and realistic traffic are done for the highway bridge. Maximal flexural beam moments per meter of the bridge width are compared (Figure 2). Bridges designed with old Croatian code will need strengthening due to maximal traffic on Croatian roads for the serviceability limit state. Bridges designed with valid Croatian code satisfy serviceability limit state for the present-day traffic. Adoption of European models for traffic load of road bridges will increase their serviceability limit state. Old bridges need reparation to come at serviceability level according new European standards.

It is possible to use correction factors of European load models to reduce unnecessary expense in designing new and strengthening old bridges. These factors will have to be adjusted with the ones proposed in neighboring countries to obtain the unique traffic system.

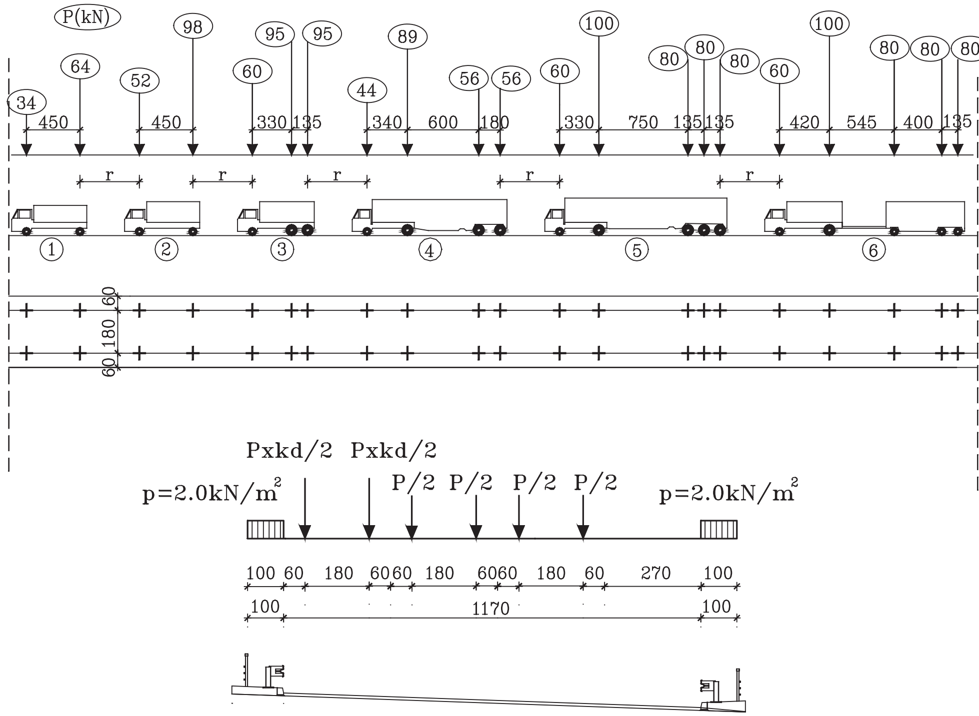

Figure 1. Heavy vehicles schemes for the realistic traffic simulation.

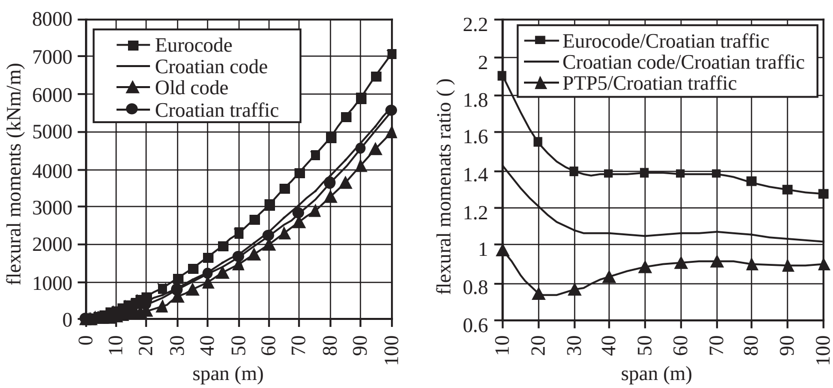

Figure 2. *Maximal* beam flexural moments due to standardized traffic load models, realistic traffic simulation and their comparisons.

Smart bridge technology

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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A low power wireless sensor network for structural health monitoring

J. Meyer, R. Bischoff & G. Feltrin
Swiss Federal Laboratories for Materials Testing and Research (Empa), Duebendorf, Switzerland

O. Saukh
University of Stuttgart, Stuttgart, Germany

1 INTRODUCTION

This paper presents a prototype monitoring system for civil structures composed of a wireless sensor network for data acquisition, a database holding the collected data and various tools allowing the user to access and evaluate the aggregated data and to configure the different components of the monitoring system. A prototype system for monitoring stay cable forces has been implemented on the scaled cable stayed pedestrian bridge at Empa.

2 METHOD

Traditional monitoring systems show a star like topology with a data logger unit as center. Various sensors deployed over a structure are connected via long cable runs to that logging unit. These systems are very time consuming to install and therefore cost-intensive. The monitoring system presented in this paper aims to solve this problems by introducing wireless data communication techniques, i.e. the data acquisition is performed by a wireless sensor network installed on the structure. This sensor network is made up of many tiny intercommunicating computers equipped with one or several sensors.

Various software tools and user interfaces have been developed and implemented which allow the user to assess the condition of the structure. These software components implement data visualization, condition representation and are responsible for the long term data storage.

The present monitoring system enables the user or system operator to observe, control and configure the components installed on the structure remotely. Figure 1 gives an overview of the monitoring system.

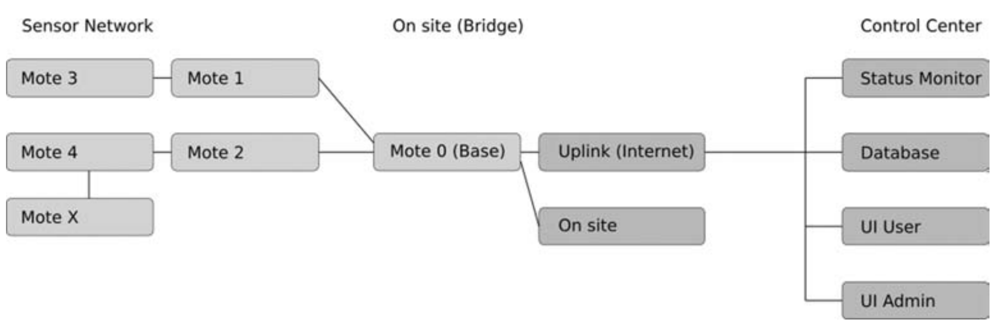

Figure 1. Overview of the present monitoring system on the laboratory bridge at Empa showing the wireless sensor network (left), the control components (right) and the link in between.

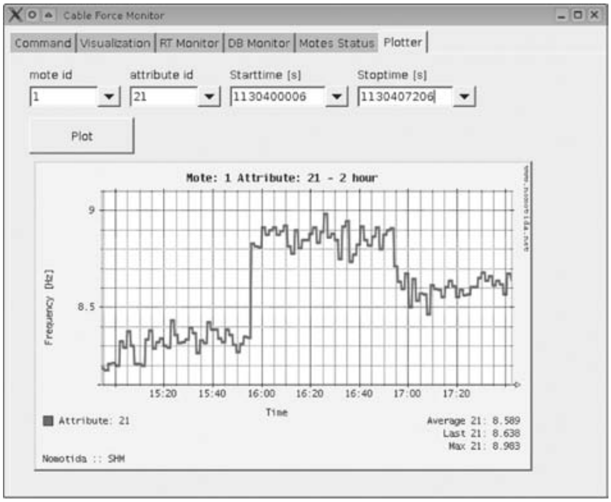

Figure 2. Screenshot of the graphical user interface. The graph window shows a plot of the monitored natural frequency of one bridge cable stay.

A potential application of wireless sensor networks is cable tension force monitoring of stay cable bridges based on vibration measurements and natural frequency estimation. A simple autoregressive model to estimate natural frequencies, which requires significantly less resources than usual frequency spectra or output only system identification algorithms, has been implemented and tested.

3 RESULTS

The presented prototype monitoring system has been successfully implemented and tested on the scaled laboratory bridge at Empa. The prototype shows that monitoring based on wireless sensor networks is feasible and that appropriate algorithms and strategies can be implemented which fit the limited memory and computation resources of the motes and provide reasonably accurate results. The cable stay force monitoring prototype yielded results with an accuracy of about 1%. Figure 2 shows a plot of the natural frequency of one stay cable in the graph window of the graphical use interface. Various software tools have been introduced which allow the monitoring system operator to access the aggregated data and assess the condition of the structure in a suitable way.

ACKNOWLEDGMENTS

Part of this work was performed within the FP6 Integrated Project "Sustainable Bridges: Assessment for Future Traffic Demands and Longer Lives". The authors acknowledge the European Commission and the Swiss Federal Office for Education and Science for their financial support.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Global smart bridge monitoring system

S. Sumitro
Smart Structures, Rantoul, IL, USA

M.H. Hodge
SIMS, Inc., Lisle, IL, USA

ABSTRACT: Deterioration mechanisms of a bridge due to numerous loading condition and various unanticipated loads highly complex. The most prevalent deterioration factors are not due to catastrophic events such as seismic disturbance and typhoons, but rather are due to natural phenomena such as aging, overload, fatigue, weather and settlement. These factors, which are hidden inside 'the black box', contribute to the long-term degradation of civil infrastructures. A reliable Structural Health Monitoring System (SHMS) should be derived to monitor the effect of these parameters on the integrity of such structures as distortion, deformation, stress change, crack propagation and corrosion progression. The long-term monitoring of structural deformational response will indicate structural degradation as a warning paradigm before catastrophic failure. Therefore, a smart bridge monitoring system that applies FODS (Fiber Optic Displacement Sensor) has been developed to monitor the global deformational behavior of bridge structural elements, such as, pier tilt, bearing anomalies, deck deflection and rotation due to live loads, environmental loads and long term pre-stress effects. By investigating field monitored results, it is confirmed that this real time monitoring system represents a reliable and economical approach to providing an early warning system for bridges.

The current bridge monitoring approach of periodic inspections followed by a flurry of in-depth sensor interrogations when some serious deterioration is suspected is not the most cost-effective approach to infrastructure maintenance. The human analogy might be a hypothetical medical system wherein an Emergency Medical Technician is expected to refer a suddenly stricken patient directly to the correct medical sub-specialist, without the benefit of a triaging emergency room staff or even the longer term history of the general practitioner (Hodge and Kausel 2001, Sumitro et al. 2001).

The SMART FODS (Fiber Optic Displacement Sensor) system is designe*d for lifetime monitoring of the entire structure*; keeping a finger on the pulse of the "patient" in real time. SMART FODS interrogates each sensor up to five thousand times per second for the life of the structure, and reports sensor data back to a remote computer base for current and long-term analysis, and it is directed primarily towards bridges. By suitably modifying the actuation of this very precise displacement sensor, SMART FODS is able to track bridge deck deflection and vibration, expansion joint travel, concrete and rebar corrosion, pothole development, pier scour and tilt as shown in Figure 1.

All of these data are received within microseconds, which means that appropriate computer algorithm manipulations can be carried out to correlate one sensor with other sensors in real time. A complete 24-7-365 history of bridge function will be available, showing annual fluctuations and irreversible sets, the effects of rush hour live loads, and differential thermal expansion effects created by the change in the sun's declination. This internal verification feature automatically enhances confidence in the system's predictive ability and alerts the user to any anomalous behavior.

The sensors consist of the body with an optical connector socket, and a slide as shown in Figure 2. Each sensor body is attached to one section of a critical structural joint. The slide is attached to the opposing portion of the critical joint, and an extension rod is provided to accommodate varying structural dimensions.

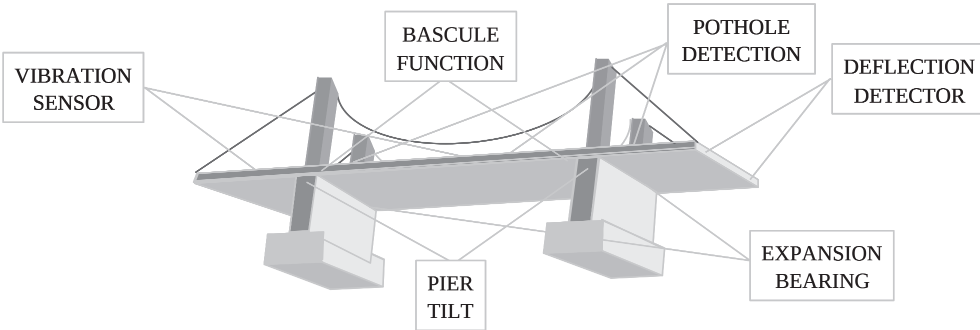

Figure 1. The essence of the SMART FODS system.

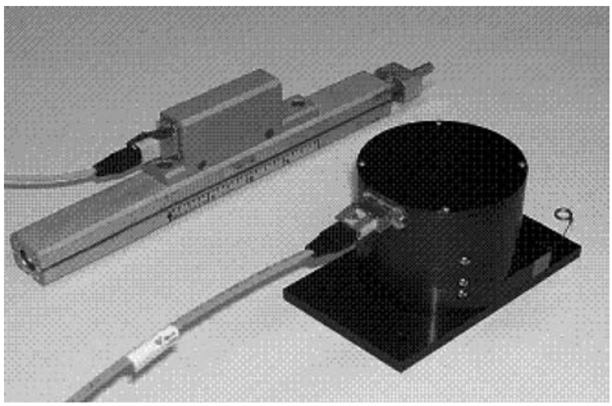

Figure 2. Optically encoded linear and rotary SMART FODS.

SMART FODS monitoring system is a network of up to fifty digital displacement sensors which are addressed through a permanently deployed optical cable-modem system. The displacement sensors are polled continuously by thousands of light pulses every second in a real time base. These pulses read the status of each of these displacement encoders and track individual movements less than one fifth of a human hair.

By investigating field monitored results, it is confirmed that SMART FODS technology is apparently suitable for developing a long-term health monitoring system for grasping deterioration of structural members of bridges.

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*Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Design approach and full implementation of intelligent SHM systems for bridges

H. Li & J.P. Ou
School of Civil Engineering, Harbin Institute of Technology, Harbin, China

ABSTRACT: Chinese economy develops rapidly in recent years, so the traffic industry gains a large-scale development, especially the long-span bridges being in the ascendant. During the service time they are inevitable to suffer from environmental corrosion, material aging, fatigue and the coupling effects with long-term loads and extreme loads. The induced damage accumulates and performance degenerates due to the above factors would inevitably reduce the resisting capacity of bridges against the disaster actions, even result in collapse with the structural failure under extreme loads. Due to those reasons structural health monitoring (SHM) has more and more attracted great research and development interests of scientists and engineers in the whole world because of the ability to ensure the safety and study the damage evolving characteristics of the structures (Ou, 1996; Ou, 2003).

The SHM system includes sensor module, data acquisition module, signal transmission module, module of damage identification, model updating and safety assessment, and data management module. The modules mentioned above involve different hardware and software, so the system integration technique is needed to integrate the independent module into a SHM system that can synergistically work.

In the past decade, most of researches focus on developing innovative smart sensors, methods of damage detection and model updating. Very few researches on the design approach of integrated SHM system have been carried out. With the increasing demand on implementation of SHM system on bridges, it comes into being an urgent task to propose the design approaches of SHM system.

In this paper, the general design principles of SHM system are proposed. And then design method of each independent module is studied. Finally, the full implementation of structural SHM system for a long-span cable-stayed bridge is introduced.

1 GENERAL DESIGN PRINCIPLES OF SHM SYSTEM

The general design principles of SHM system for cable-stayed bridge mainly include three aspects: monitoring content, the grade of monitoring system and objectives.

2 LOCATION OPTIMIZATION AND SELECTION PRINCIPLES OF SENSORS

2.1 *Location optimization of sensor*

When SHM is designed, the location of sensor is firstly determined. The design method of location of sensors varies with the variables measured by sensors.

2.2 *Criteria of choosing the right type of sensor*

In order to fulfill the purpose of SHM, the following primary criteria for selecting the right type of sensors are considered: advance, sensitivity, precision, proper operating environmental conditions, low cost, high reliability, high durability, high stability, easy installation, replaceability, rehabilitation and expansibility.

3 DESIGN METHODS OF DATA ACQUISITION, TRANSMISSION AND MANAGEMENT SYSTEMS

Design of data acquisition system includes scheme of the system, instrument, software, sampling rate and operational strategies used in the system. Generally, the higher grade of the SHM system is, the higher performance of the data acquisition system is.

In an SHM system, data transmission technology plays the role to transmit data from sensor to industrial computer, from industrial computer to server and from server to user. Wired or wireless transmission technology can be chosen to transmit data from sensors to industrial computer according to the types of sensors.

Data measured by SHM system for cable-stayed bridge should be stored in order and effectively managed. Database provides a tool to manage huge amount of data.

4 METHODS OF DAMAGE DETECTION AND LOCALIZATION, MODEL UPDATING AND SAFETY ASSESSMENT

The purpose of SHM system is to detect and localize damage, research damage accumulation and evolving, assess health status and safety level.

5 INTEGRATION TECHNIQUE OF SHM SYSTEM FOR CABLE-STAYED BRIDGES

As described early, different instrument and software are used in each module of SHM system for cable-stayed bridges. Integration technique can assemble all software and instrument into a whole SHM system and make the system automatically and harmonically operate.

6 DECISION-MAKING BASED ON MEASUREMENT OF DATA

The purpose and objectives of SHM system for cable-stayed bridge are to diagnose health status, assess safety, comprehend the feature of load and response, research damage accumulation and evolvement of bridge.

7 FULL IMPLEMENTATION OF SHM SYSTEM FOR A CABLE-STAYED BRIDGE

The Shandong Binzhou Yellow River Highway Bridge is a cable-stayed bridge with three towers. An SHM system has been installed on this bridge two years ago, in which includes optic fiber Bragg-grating strain and temperature sensors, accelerometers, anemoscopes and GPSs. All the system is designed by using the proposed method above. The system has been running at website and can be operated via Internet.

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Monitoring of PC structure with distributed sensing techniques

Z.S. Wu & C.Q. Yang
Department of Urban & Civil Engineering, Ibaraki University, Hitachi, Ibaraki, Japan

K. Kishida
Neubrex Co., Ltd, Kobe, Hyogo, Japan

ABSTRACT: This paper summaries our recent development of smart health monitoring approaches for prestressed concrete (PC) structures based on different distributed sensing techniques. The current distributed sensors are Brillouin scattering-based fiber optic sensors (FOS) and HCFRP (hybrid carbon fiber reinforced polymer) sensors. The HCFRP sensor is a novel type of composite sensor, which consists of three types of carbon tows: high strength (HS), high modulus (HM) and middle modulus (MM) carbon tows. Both types of sensors are characterized by a broad-based and distributed sensing function. The HCFRP sensors are bonded on PC tendon, steel reinforcing bar, and embedded in tensile and compressive concrete sides with epoxy resins and putties. The FOS are embedded in the tensile and compressive concrete sides where the HCFRP sensors are embedded as well. The distributed sensors are arranged to detect the initiation and propagation of cracks, yielding of steel reinforcements and corrosion of PC tendons. The experimental investigations demonstrate that the initiation and location of cracks, yielding of steel reinforcements, corrosion of PC tendons and structural health of PC structures can be effectively detected and monitored with such kinds of distributed sensing systems.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Damage detection of truss structures

Z.D. Duan, G.R. Yan & J.P. Ou
School of Civil Engineering, Harbin Institute of Technology, Harbin, China

1 INTRODUCTION

Techniques for damage detection based on structural deflection flexibility have received considerable attention due to the following unique features of the flexibility matrix. One is that the flexibility matrix can be estimated from a truncated set of lower frequency modes with sufficient accuracy. And the other is that flexibility matrix at the measurement sensor coordinates can be extracted from the matrices of system realization. This paper is dedicated to develop a new flexibility for damage detection of truss structure. The main idea is that members in truss structure are dominated by axial forces, and axial stress or strain could be a better index than deflection for damage detection. An Axial Strain Flexibility (ASF) is proposed. The remainder of this paper is organized as follows. First the ASF is defined and ASF matrix is formulated in terms of translational modes. Second, two damage detection techniques are applied to ASF matrix, and performance of the damage detection techniques with ASF matrix is validated by a numerical example. Finally the paper is concluded.

2 AN AXIAL STRAIN FLEXIBILITY OF TRUSS STRUCTURE

A flexibility matrix for truss structure, which is called Axial Strain Flexibility (ASF), is defined, and formulation of ASF by translational modes is deduced. The element in ASF is defined that ASF_{ji} is the axial strain of the jth member resulted from a pair of forces with equal amplitude, but opposite directions applied at two nodes of and along the axis of the ith member. The magnitude of the equal forces are the reciprocal of the length of the ith member. The ASF can be formulated in terms of translation modes as follows

$$\mathrm{ASF} = \sum_{r=1}^{n} \frac{1}{\omega_r^2} S_r S_r^T$$

where S_r is the rth axial strain mode shape

$$S_r = \begin{pmatrix} c_1 \dfrac{(\varphi_{2a-1,r} - \varphi_{2b-1,r})}{l_1} + s_1 \dfrac{(\varphi_{2a,r} - \varphi_{2b,r})}{l_1} \\ \\ c_j \dfrac{(\varphi_{2o-1,r} - \varphi_{2p-1,r})}{l_j} + s_j \dfrac{(\varphi_{2o,r} - \varphi_{2p,r})}{l_j} \\ \\ c_n \dfrac{(\varphi_{2w-1,r} - \varphi_{2x-1,r})}{l_n} + s_n \dfrac{(\varphi_{2w,r} - \varphi_{2x,r})}{l_n} \end{pmatrix}$$

and ω_r is the rth circular modal frequency; φ_{ir} and φ_{jr} are the ith and jth entries of the rth mode shape respectively; n is the number of measured modes.

3 DAMAGE DETECTION BY ASF

Two damage detection methods based on ASF are investigated, namely direct ASF difference method and Damage Locating Vector method.

3.1 *Direct ASF Difference Method*

The ASFs for the intact and damaged structures, denoted as ASF^u and ASF^d, respectively, are first constructed by the procedures presented in the previous section. Then the changes in ASFs are computed as

$$\Delta ASF = ASF^d - ASF^u$$

The percentage of changes in the diagonal entries of ΔASF is designated as the damage indicator of each element. The elements with large percentage of change are identified as damaged members.

3.2 *Damage Locating Vector approach based on ASF*

The Damage Locating Vectors (DLV) method, developed by Bernal [16], is an approach for damage localization using changes in measured flexibilities for the pre- and post-damage states. The fundamental idea of the DLV approach is that the vectors that span the null-space of change in flexibility (from the pre to post damage states), induce no stress in the damaged elements when they are treated as static loads on the undamaged structure. The ASFs are employed instead of deflection flexibility to implement the DLV method. To obtain the DLVs, a Singular Value Decomposition (SVD) is performed on ΔASF as

$$\Delta ASF = USV^T = \begin{bmatrix} U_1 & U_0 \end{bmatrix} \begin{bmatrix} s_1 & 0 \\ 0 & 0 \end{bmatrix} \begin{bmatrix} V_1^T \\ V_0^T \end{bmatrix} \tag{14}$$

The DLVs are applied on the members of the intact truss in the form of a pair of axial forces with equal magnitude and opposite direction to calculate a generalized internal force of each member. The elements with small generalized internal forces are potentially damaged.

4 NUMERICAL EXAMPLES

By numerical simulation of modal test, the performance of the proposed damage detection procedures with ASF is investigated. A 14-bay planar truss structure with simply supports as shown is considered in this example. Damages are simulated as reduction in cross-section areas of different members by 10%.

Structural damping is assumed to be Rayleigh damping with critical damping ratio of 1% for each mode. The truss is excited by limited-band random white noise applied at all nodes. Simulated acceleration responses are computed by Newmark-Beta integration. 5% of the RMS noise is added to the structural responses.

The example shows that: (1) ASF is more viable for damage detection than deflection flexibility; (2) sensors can be placed on the part that you are interested in, and damages within this part can be detected with ASF.

5 CONCLUSIONS

An Axial Strain Flexibility (ASF) is proposed for damage detection of truss structures, and formulation to synthesize ASF by identified translational modes is deduced. The ASF matrix is diagonal for statically determined truss structures, and is dominated by diagonal elements for statically under-determined structures. Two damage detection techniques, i.e., direct difference flexibility method and DLV method, are investigated using the proposed ASF matrix. A numerical study shows that generally ASF matrix is better than deflection matrix for damage detection of truss structure. Some uniqueness or characteristics are observed for the ASF based damage detection techniques: 1) they can both locate and identify severity of damages; 2) damages can be detected in parts of a structure where the sensors are placed; 3) they are robust to measurement noises.

ACKNOWLEDGEMENTS

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State-of-the-art and state-of-the-practice and guidelines of bridge health monitoring in the mainland of China

J.P. Ou & H. Li
School of Civil Engineering, Harbin Institute of Technology, Harbin, China

ABSTRACT: Structural health monitoring (SHM) systems more and more become a technique for ensuring the health and safety of bridges and also an important approach for research of the damage accumulation or even disaster evolving characteristics of bridges, and attract prodigious research interests and active development interests of research community, engineers and public sector since a great number of bridges are planning and constructing each year in mainland China (Ou, 2003).

SHM system includes sensor module, data acquisition and process system, signal transmission system, damage detection, model updating, safety evaluation, data management, integrated system, decision-making techniques of health status of structure based on measured data. In the past decade, Chinese researchers have made great progress and fruit achievements in research and application of SHM. In this paper, the state-of-the-art and state-of-the-practice of SHM for bridges in the mainland of China are summarized.In this paper, some advance smart sensors developed by Chinese researchers, especially optical fiber Bragg-grating (OFBG) sensor, OFBG-based smart self-sensing products are introduced. Wireless strain sensors, wireless accelerometers and sensor networks are presented. The implementations of SHM systems for bridge are summarized. And then two SHM systems for two cable-stayed bridges are presented in detail as examples to demonstrate the development of SHM in the mainland of China. Based on the research and application, guidelines of SHM for bridges are urgent to publish. Recent progress on this topic is also introduced in this paper.

1 ADVANCED SMART SENSORS

Various optical fiber Bragg grating (FBG) strain sensors have been developed in author's group. The performance of the sensor, including sensing properties, corrosion resistance and fatigue resistance, has been systematically studied.

2 WIRELESS SENSORS AND SENSOR NETWORKS

A good SHM system requires the sensor to have the following merits: cheap; durable; easy and simple to install and maintain; wireless; no battery replacement needed for operation; smart, which means individual or a set of individual sensors can process sensed data and directly outputs the information regarding the health or damage status of the structure. To improve these disadvantages and reach the aims, a task effort has been made towards to developing wireless sensors, wireless sensor networks and wireless monitoring systems in mainland China.

3 IMPLEMENTATION OF SHM SYSTEMS FOR BRIDGES

Many long span bridges have been constructed or are planning to be constructed in China, such as the Hangzhou Bay Bridge with total length of 36 km, the Eastern Ocean Bridge with the total length of 32 km, the Qingdao Bay Bridge with total length of 26 km, the Sutong Bridge with the main span of 1080 m that ranks No. 1 over the world, and etc.

Public sector concerns the safety status of the bridges, and thus promotes the applications of SHM technology. Most of long span bridges are implemented with SHM systems in the mainland of China as well as Hong Kong. The information of SHM systems for the bridges in the mainland of China and Hong Kong in details can be found in Ou (2005).

Two SHM systems for two cable-stayed bridges are presented in detail.

- The SHM system for the Shandong Binzhou Yellow River Bridge. The Binzhou Yellow River Highway Bridge is a cable-stayed bridge with three towers in Shandong, China It has a total length of 768 m consisting of two three hundred-meter main spans and two eighty four-meter side spans.

  Following modules are included in this system: sensor module, data acquisition and processing module, signal transmitting module, structural analysis module including damage detection, model updating and safety evaluation, and database module.
- The SHM system for the Harbin Songhua River Cable-stayed Bridge. The Harbin Songhua River Cable-stayed Bridge is a cable-stayed bridge with a main span of 365 m. A SHM system is implemented in this bridge before it opened to traffic on August 26, 2004. The system continuously collects data for 7 days each time and twice time each year. After collecting data, structural analysis and health diagnosis based on measured data are conducted offline in lab, not in-situ. The SHM system consists of sensor module, data acquisition module, database (same as that in last example) and module of structural analysis and decision-making based on measured data.

4 DESIGN METHODS AND GUIDELINES OF SHM SYSTEMS FOR BRIDGES

The design for an SHM system for bridges included following steps: conceptual; design, location, type and amount of sensors; hardware, software and strategies of data acquisition system; data transfer system; structural analysis; data interpretation; data management; system maintenance and upgrade.

The design methods of SHM system for long-span bridges have been proposed by authors and can be found in Li and Ou (2006). Additionally, the guideline of SHM systems for long-span bridges will be published by the Ministry of Transportation in the mainland of China in two years.

5 CONCLUSIONS

This paper introduces the recent advances on SHM for bridges in the mainland of China and the advance in the design guideline of SHM systems for long-span bridges. The good performance of the smart FBG sensor and FBG-based smart products, and wireless sensors provide a solid foundation to apply SHM in practical engineering. A great number of SHM systems implemented in bridges in the mainland of China offer a new pathway to research the performance and deterioration of bridges in long-term service.

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Structural control of seismically induced pounding of elevated bridges by using magnetorheological dampers

A.X. Guo, L.L. Cui & H. Li
School of Civil Engineering, Harbin Institute of Technology, Harbin, China

ABSTRACT: Pounding between the adjacent members of the elevated bridges is an important factor that affects the safety of the bridges under modern to strong earthquakes due to the differences in dynamic characteristic of the adjacent structures and spatial variability of the ground motions. The exchange of the velocity during pounding can induced abrupt acceleration change in the adjacent frames of the elevated bridges, and the impact of the girders or the slabs of the highway bridges can generate huge additional forces to the members resulting in the spalling of concrete and the damage of the piers and abutments, even the unseat of the bridge girders.

The aim of this study is to investigate the possibility of using MR dampers to suppress the collision between the adjacent segments of the elevated bridge in seismic events. Finite element model of an elevated bridge is established, and the dynamic responses of the uncontrolled structures with different gap size of the expansion joints are investigated firstly to analyze the effects of the collision of the deck. Then, dynamic analyses for structure with MR dampers are also conducted to investigate the performance and effectiveness of the MR dampers for suppressing the collision of the bridge. Semiactive control strategy of the MR dampers is designed and used to manipulate the actuators. The simulation results show that the semiactive control system by using MR dampers can effectively reduce the structural responses if the control system is appropriately designed.

Finite element model of an elevated bridge is established by using the ANSYS code. In this model, the decks of this bridge are assumed to be linear in this analysis because the inertia force generated in the decks due to earthquake mainly acts in the longitudinal direction. For the columns of the bridge, the nonlinear properties are modeled by using two plastic hinges at the two ends. The collision of the adjacent segments of the bridge is modeled using the Combin40 element integrated in the ANSYS code.

The analysis results show that collision between the adjacent segments of the elevated bridge can induce huge impact forces, which will resulting the spalling of the concrete at the expansion joints. The shear forces and the moments are also not significantly changed with different gap size of the expansion joints. The results indicate that the variation of the gap size cannot affect the shear forces and the moments significantly, and the huge impact forces generated in the superstructure are not transmitted to the bridge columns. This result is similar as that obtained, which indicated that although the impact forces are large in magnitude they are quite short in duration compared with the longitudinal period of the bridge segments.

The semiactive control system is designed based on the assumption that no collision will occur at the expansion joint with the installation of the devices. To simplify the control system design, the structural is also assumed to be linear with providing damping by the MR dampers. According to the above assumption, the governing equation of motion of the bridge can be given as

$$M\ddot{X} + C\dot{X} + KX = -MD_s\ddot{x}_g + B_sU_s \tag{1}$$

where M, C and K are the mass, damping and stiffness matrices, respectively; X is the displacement vector of the superstructure; $\dot{X}$ and $\ddot{X}$ are the corresponding velocity and acceleration vectors, respectively; D_s is the excitation influence vector and F is the excitation vector of the ground, and

B_s is the MR location vectors; U_s is the force generated by the MR damper. B_sU_s can be expressed by equation as follow

$$B_sU_s = -C_d\dot{X} + B_sF_{dy} \tag{2}$$

where the first term at the right is the viscous damping force generated by the MR damper, and the second term is the coulomb force. Then the governing equation of motion of the bridge can be given as.

$$M\ddot{X} + (C + C_d)\dot{X} + KX = -MD_s\ddot{x}_g + B_sF_{dy} \tag{3}$$

The control strategy of the semiactive control system is designed to determine the control force of the MR damper by tracing the optimal control force obtained by using the LQG control algorithm. According to the control strategy of the control system, the control force of the MR dampers can be obtained as follow: (1) An optimal control algorithm of the system designed firstly by using the LQG control algorithm; (2) The determined control force is assumed to be the optimal semiactive control forced, and the control force should be provided by the MR dampers as much as possible; (3) Decide the control electricity and voltage of the dampers.

According to the above control strategy, the performance of the semiactive control system will achieve the best if the MR damper can accurately provide the same optimal control force determined by using the LQG algorithm. However, due to the shortcoming of the semiactive control system it cannot achieve at all time. So, Hrovat optimal controller algorithm is adopted to trace the optimal control force, and it can be expressed as

$$u_d = \begin{cases} c_d\dot{x} + F_{dy\max}\,\mathrm{sgn}(\dot{x}) & (u\dot{x} < 0 \text{且} |u| > u_{d\max}) \\ |u|\,\mathrm{sgn}(\dot{x}) & (u\dot{x} < 0 \text{且} |u| < u_{d\max}) \\ c_d\dot{x} + F_{dy\min}\,\mathrm{sgn}(\dot{x}) & (u\dot{x} \geq 0) \end{cases} \tag{4}$$

in which $u_d = c_d|\dot{x}| + F_{dy\max}$ is the maximal control force generated by MR damper according to the optimal control force.

A semiactive control system for eliminating the pounding of the elevated bridge is designed according the above method. Numerical simulation of the elevated bridge with gap size of 5 cm of the expansion joint is conducted by using the semiactive control system. The analysis result show that the shear force decrease from 1.29×10^6 N to 1.00×10^6 N at the base of the middle segment piers of the elevated bridge, and the moment also decrease from 2.84×10^7 Nm to 2.22×10^7 Nm with the installation of the MR dampers. To validate the linear assumption of the control system design, the time history of the control force obtained in the simulation is saved. The nonlinear analysis of the elevated bridge is also conducted by using the ANSYS code, in which the saved control force time history is applied as axial force in the expansion joint in the analysis. The simulation results show the assumption is acceptable and accurate in the practice of engineering.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Development of bridge management system for expressway bridges in Japan

K. Yokoyama, S. Sakai, N. Inaba, A. Homma & N. Ogata
Expressway Research Institute, Central Nippon Expressway Company Limited, Japan

ABSTRACT: The Japan Highway Public Corporation (JH) that has been divided into three private expressway companies in October 2005 has been actively trying to reduce the maintenance costs for its express-highway networks. As the starting point of its series of efforts, JH has been trying to computerize all the processes of maintenance activities. JH proposed a total bridge management system (BMS) to support its decision-making process for well-planned maintenance by determining bridge condition, predicting future deterioration, and selecting optimal timing and method for repair and/or reinforcement. The original version of its BMS, called JH-BMS, was developed in 2003. Since then, some study results have been achieved to improve the quality of original JH-BMS. This paper discusses study results toward the development of JH-BMS, such as bridge soundness evaluation and deterioration prediction based on the inspection results, and optimum maintenance strategy to minimize life cycle cost. Future directions of research activities are also discussed. JH-BMS is designed not only as a tool for proper budget allocation but also for supporting engineering decision-making process in the practical bridge maintenance field. JH-BMS is a decision-making support system on planning of bridge repair or reinforcement. The system uses a bridge maintenance database including the bridge specifications and inspection data in order to evaluate the soundness of bridge elements, predict deterioration, select optimal timing and method of repair or reinforcement and calculate the estimated costs. In JH-BMS, bridge conditions are evaluated by individual elements. Current condition of individual bridge elements is evaluated based on the inspection results, environmental condition and expected future traffic volume. And then, future deterioration of the elements is predicted at the time of inspection depending on the deterioration mechanism. The basic approach of JH-BMS is to calibrate the existing deterioration prediction equations (formulas) developed in Japan, based on the field inspection data. In these equations, specific mechanism of deterioration of each structure such as chloride attack or fatigue expected under severe environmental condition is taken into consideration. Historical inspection data will be used in JH-BMS to improve the accuracy of soundness evaluation and deterioration prediction of individual bridge elements. The results of accurate deterioration prediction enable the proposal of well-planned bridge maintenance strategies including preventive maintenances actions.

JH-BMS offers the features as follows:

(1) The soundness of individual bridge elements is determined at the time of inspection based on the inspection results, applied specifications and environmental conditions.
(2) Deterioration of elements expected at a given point in time in the future is predicted based on their current conditions, identified deterioration mechanism, structural characteristics, and environmental condition at the time of inspection.
(3) The effect and unit cost of each repair or reinforcement method are determined depending on the deterioration mechanism. The method of repair or reinforcement and its application times are selected to optimize the life-cycle maintenance cost.
(4) The maintenance cost required throughout the design service life is calculated for each bridge. Calculating sub-total costs by routes, areas under jurisdiction or other classifications are possible.

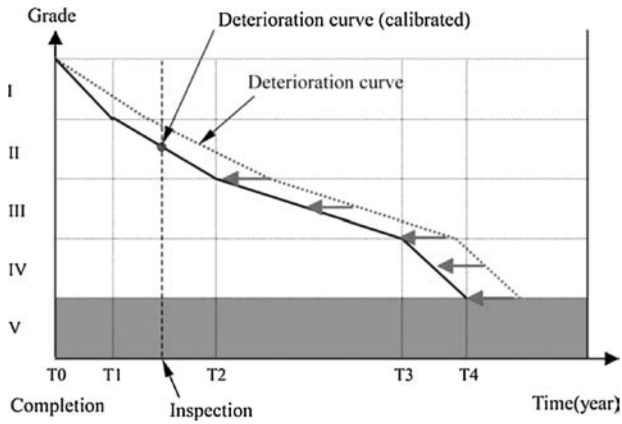

Figure 1. Calibration of deterioration curve.

The soundness of an element at a given point in time is determined using inspection data. Deterioration of the future condition states can be predicted based on the prediction formula combined with the inspection data. For example, if chloride attack causes deterioration, data such as the chloride ion content at the depth of reinforcement, area of corroded reinforcement and cracking data are obtained through inspection, and used for evaluating element soundness. If sufficient data is not available, future condition of the element against chloride attack is evaluated based on the numerical calculation by plugging the parameters into the prediction equations. Future deterioration is predicted based on the calculation results according to the deterioration mechanism or actual progress of deterioration identified using historical inspection record. Right now, inspection is performed mainly by visual inspection and cannot provide all the necessary data to evaluate element soundness. In the future, inspection items and methods should be improved in order to provide quantitative data as well as the visual inspection results required for soundness evaluation and deterioration prediction. For carbonation and chloride attack, the depth of carbonation and chloride ion concentration is calculated using the Japan Society of Civil Engineers (JSCE) formulas[2]. These formulas are calibrated using the inspection results. For the fatigue of RC slabs, the soundness index is calculated using Matsui's formula[3]. Deterioration prediction curves are calibrated based on the results of mainly visual inspections and of detailed investigations (carbonation depth, chloride amount, etc.) (see Figure 1).

JH-BMS is a decision-making support system based on the bridge maintenance database. The database is composed of applied specifications and historical inspection data to evaluate the soundness of bridge elements, predict deterioration, select optimal timing and method of repair or reinforcement and estimate the life-cycle repair or reinforcement cost. Deterioration predicttion curves are calibrated according to the results of mainly visual inspection and of detailed investigations. It is still important to include engineers' know-how as decision-making criteria even if JH-BMS is in practice in the future. JH-BMS is designed to enable engineers to calibrate system output based on their judgment. The system can also be an effective tool to objectively prove the validity of conventional bridge maintenance strategies based on engineers' judgment. It is hoped that the technologies of this study field would be improved and those technologies should be used for more advanced JH-BMS.

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*Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Acceleration response energy method for damage identification of bridge structures

Z.S. Wu & Z.-D. Xu
*Urban and Civil Engineering Department, Ibaraki University, Hitachi, Japan;
Civil Engineering College, Southeast University, Nanjing, P.R.China*

ABSTRACT: The potential economic and life-safety implications of early damage identification in civil, aerospace and mechanical engineering systems have motivated a significant amount of research in structural health monitoring. It is hopeful to detect damage in a fast and reliable way in order to increase the safety and security, extend the working life and reduce the maintenance cost of long-span bridges. Recently, more attentions are paid for dynamical damage identification technique, such as frequency-based technique, mode-based technique. While little attention has been paid to energy damage identification technique based on acceleration responses, which is important in real-world application with wide use of accelerometer. Moreover, accurate mode shapes solution is required for the few existing energy damage identification technique.

In order to erect damage identification technique for real-world application with wide use of accelerometer and avoid complex mode shape solution, a damage identification strategy based on acceleration responses energy is proposed based on the relationship between frequency response function (FRF) of acceleration responses and mode shapes in this paper. The basic idea of this strategy is determining the relationship between FRF of acceleration responses and mode shapes for the second-order dynamic system in the frequency domain firstly, and then deriving the expression of power spectral density and energy of acceleration responses. Numerical analysis on a long-span cable-stayed bridge with different degrees girder damages is performed. The results between the acceleration responses energy-based damage identification strategy and the traditional mode shape curvature strategy are compared. At the same time damage robustness analysis for noise pollution is carried. It can be shown from the analysis results that the acceleration responses energy-based damage identification strategy not only has accurate damage identification ability but also has excellent anti-noise pollution ability.

The main equations of the proposed energy damage identification method based on acceleration responses is written as

$$E = \frac{1}{2\pi}\int_{-\infty}^{\infty} E(\omega) d\omega = \frac{1}{2\pi}\int_{-\infty}^{\infty} \left|\tilde{\ddot{x}}(\omega)\right|^2 d\omega = \frac{1}{2\pi}\int_{-\infty}^{\infty} \left|FFT(\ddot{x}(t))\right|^2 d\omega \tag{1}$$

Considering the relationship $f = 2\pi\omega$, the following equation can be got.

$$E = \int_{-\infty}^{\infty} E(f) df \tag{2}$$

where $E(\omega)$ is the PSD function of acceleration responses.

In the paper, two kinds of energy indexes based on the acceleration energy are used to locate and quantify damages of structures. The energy difference ΔE between the damaged and undamaged structures can be written as follows

$$\{\Delta E\} = \left|\{E_d\} - \{E_0\}\right| \tag{3}$$

where E_d and E_0 are the acceleration energy calculated by Equation (18) of the damaged and undamaged structures, respectively. The location of the damage is then assessed by the largest computed absolute difference between the acceleration energy of the damaged and undamaged structures.

Another energy index is proposed based on idea of mode shape curvature difference, namely, energy curvature difference, which can be written as

$$\left.\begin{aligned}\{\Delta E''\} &= \left|\{E''_d\} - \{E''\}\right| \\ E''_i &= (E_{i+1} - 2E_i + E_{i-1})/h^2\end{aligned}\right\} \tag{4}$$

where $\Delta E''$ is the energy curvature difference, E''_d and E'' are the energy curvature of the damaged and undamaged structures, respectively, E''_i is the energy curvature at the location i on the structure, E_i is the energy at the location i on the structure, and h is the distance between the measurement points $i + 1$ and $i - 1$. As above, the location of the damage is then assessed by the largest computed absolute difference between the acceleration energy curvature of the damaged and undamaged structures.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

SMARTE – Development and implementation of a long term structural health monitoring

V. Perdigão & P. Barros
BRISA – Portuguese Highways, Quinta da Torre da Aguilha, Edifício BRISA, São Domingos de Rana, Portugal

J.C. Matos, H. Sousa & J.A. Figueiras
LABEST – Laboratory for the Concrete Technology and Structural Behaviour, Faculty of Engineering of Porto, Porto, Portugal

I. Dias & D. Pereira
INESC Porto – Telecommunications and multimedia systems unit, Porto, Portugal

ABSTRACT: The accompanying of civil infrastructures behavior during their whole life cycle is growing within the last few years. This fact is due to several problems detected in such structures, which is the main reason for the dispended budget by owners in their maintenance, being even higher than other costs. In another way, traditional surveillance techniques, although important are expensive, slow in time and subjective. Considering such panorama, within a national research project – SMARTE project – a new technique for management and maintenance of structures was developed and implemented in a real structure, a pre-stressed concrete bridge (Sorraia River Bridge). The developed technique, long term structural health monitoring, has as a final purpose the execution of structural maintenance in an objective and efficient way. The evaluation of the structural behavior during whole life cycle is performed in an "on line" continuous way, allowing the on time detection of existent damages.

Civil infrastructures health monitoring should have the potential for the on time detection of possible anomalies or critical situations, diminishing costs related to inspections and simultaneously growing structural and users' security. Inserted in a research project – SMARTE project – a new surveillance technique for the prevention and support of civil infrastructures maintenance and management was applied into a real prototype structure – Sorraia River Bridge (Perdigão et al. 2004). The developed long term structural health monitoring is composed by a sensory, a data acquisition, a communication, a data processing and archiving and damage detection and modelling system (Figueiras et al. 2004, Matos et al. 2005). Figure 1 presents a simple and illustrative scheme of it.

SMARTE project has two main objectives. The first one is related to the installation of the sensory system in the bridge, during it execution phase, which should be liable and robust. In Fig. 2 it is possible to observe the used cantilever constructive process and utilized instrumentation devices (Figueiras et al. 2004). This respective component, based on sensors that were placed in special locations according to a previously established criteria, will allow the readings and storage of most important parameters for a correct interpretation of the structure behaviour during it whole life cycle. The second main objective is related to the development of a component for data processing and archiving. Such system should translate the obtained data in more objective information that could be used as decision criteria for civil infrastructures management.

In this article it is presented the SMARTE project and respective main objectives. A brief description of each system of the developed and implemented long term structural health monitoring scheme is also executed. The initially proposed objectives were achieved and new research themes appeared with this project. A new project was so initialized which aim is to continue the Sorraia River Bridge long term monitoring and to develop the previous identified investigation subjects.

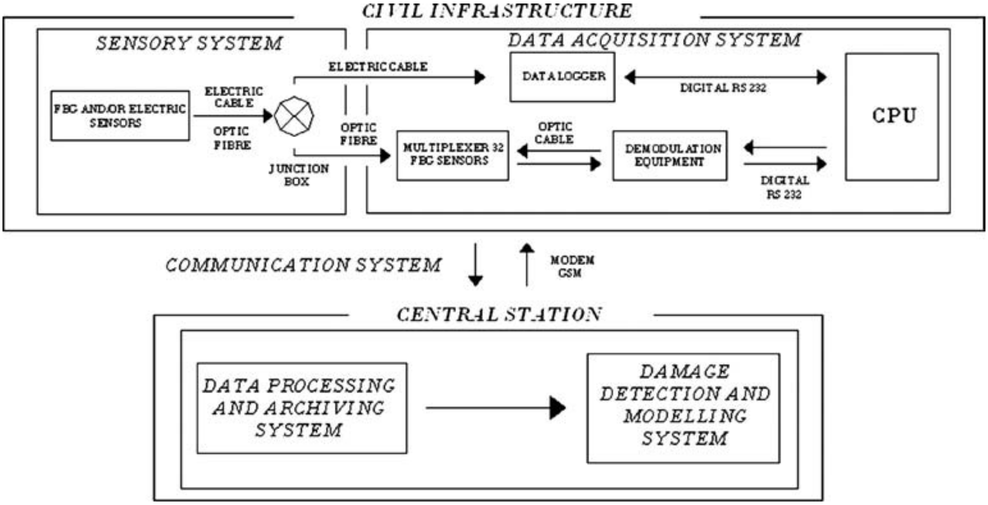

Figure 1. Scheme of the developed long term structural health monitoring.

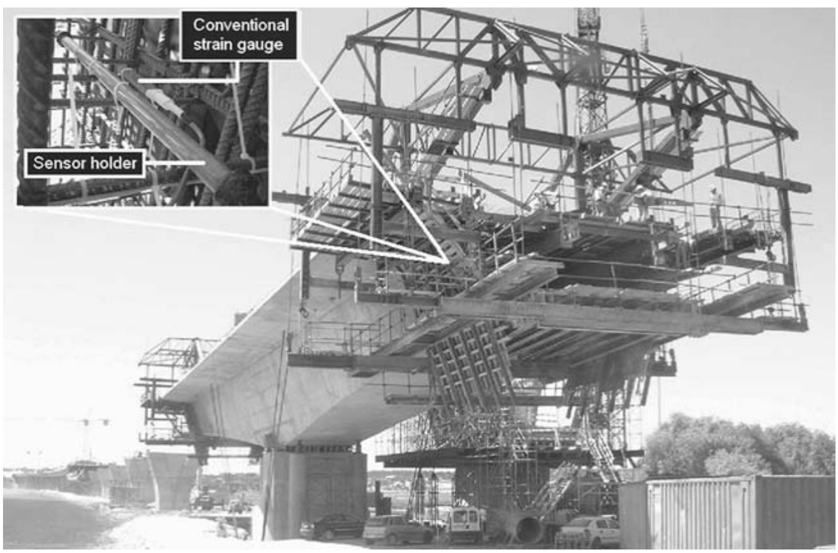

Figure 2. Sensory system and Sorraia River Bridge execution process.

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Cruz, Frangopol & Neves (eds)

Use of mobile measuring system for bridge monitoring

V.N. Fedoseyev
GUP "Gormost", Russia

E. Brodskaia
AGA Group, Inc., Chestnut Hill, MA, USA

ABSTRACT: Continuous monitoring of bridges provides information on the bridge condition and permits to forecast structural deterioration. This allows for prompt measures of repair and rehabilitation to be timely taken and ensures the reliable functioning of a given bridge.

The goal of monitoring the Moskvoretsky bridge (Moscow) is to record the vertical position of the three-joint arch support of abutment spans. In the arch key of this span the subsidence of up to 15 cm had been observed.

Due to intensive traffic, it was impossible to perform direct measurements of vertical displacements in the arch key. It was decided to use indirect methods for determining vertical movement by values of angular motion in the three joints. Recordable values were determined for any moment of time using an equation system, in which proportionality coefficients were obtained by preliminary spatial arch computation. Geodesic survey was carried out simultaneously to verify the data obtained from the transducers. The error of the measurements did not exceed the leveling accuracy.

Monitoring was conducted with the aid of a mobile monitoring unit, the main advantages of which are its repeated utilization relative simplicity of installation and suitability for any period of the bridge's life cycle. The system consists of displacement transducers and data loggers.

Information received from the Data Logger are data arrays convertible with the use of a special software NIKA, which, as the part of AFPγ kit, had been developed to meet the following tasks:

- receiving files in standard formats;
- conversion of electrical measurement units into directly measurable value units;
- saving initial data received from the Data Logger as individual files and updating the existing files with new data;
- primary screening-out of data falling into the total instrument error interval;
- calculation of statistical characteristics, such as expectation, dispersion RMS deviation, variation coefficient.

NIKA also provides usual services, like calculation of algebraic data functions, plotting various diagrams, etc.

The use of continuous monitoring system resulted in an enormous amount of data, which was difficult to promptly process and analyze. The proposed primary data processing method is to allow the user to have a concise data in a visualized form convenient for further analysis. Meaningless data was rejected based on comparison of adjacent readings and the measurement error. Identification of doubtful values was made by statistical data processing using adjustable error criteria.

The processing efficiency of the method proposed exceeds 95%. For instance, processing of output data array obtained during the monitoring of Moskvoretsky bridge allowed to reduce data array size from 9,701 down to 314 strings, i.e. screening rate amounted to 96.76%.

Detailed comparison of curves based on "processed" and "raw" data array demonstrated that no data corruption took place during the processing, only "noise" was eliminated instead, i.e. peaks were cut by an amount exceeding the measurement error.

In course of monitoring, the optimal interval for transducer scanning was found to be three hours, but even with such periodicity the procedure resulted in an enormous amount of data, which was difficult to promptly process and analyze.

As a result of continuous monitoring it was found that the vertical displacements (40–50 mm) in the joint of the arch of the Moskvoretsky bridge were only of seasonal character. Horizontal movements in the arch abutments varied from 2 mm to 5 mm. Furthermore, unsatisfactory performance of the key joint was identified. It had manifested in discrete and delayed movements of up to a few hours as compared with the joints at the arch abutments. Replacement of the joints was fitted into the major repair schedule of 2006–2007.

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Structural health monitoring of Delaware's Indian River Inlet Bridge

M.J. Chajes, H.W. Shenton, D.F. Weston, T.J. Stuffle & Julia West
Department of Civil and Environmental Engineering, University of Delaware, Newark, Delaware, USA

ABSTRACT: Construction of Delaware's new Indian River Inlet Bridge is being planned. An innovative 305 m single-rib concrete arch bridge was designed by the Figg Engineering Group (Pate, 2004). In an effort to allow the Delaware Department of Transportation (DelDOT) to better maintain and operate the bridge, the Center for Innovative Bridge Engineering (CIBrE) at the University of Delaware is planning to install a permanent long-term structural health monitoring system on the bridge. Working closely with DelDOT and Figg from the initiation of the project, the research team has designed a comprehensive monitoring system for the bridge. The system will enable monitoring of critical behaviors during construction and continuous monitoring during normal operation. The health monitoring system will be used to verify the innovative design, to monitor the structure during construction, and to track the long-term performance of the bridge. The data from the system will become part of the maintenance and inspection records for the bridge, which will enable DelDOT to better maintain and operate the bridge throughout its life. The health monitoring system includes numerous embedded and surface mounted sensors distributed throughout the bridge, a networked data acquisition system, power and communications conduits, and a central processing system. Critical to the success of the design was the close coordination maintained with the design team, design review team, and owner throughout the entire design process.

To enable DelDOT to better maintain and operate the bridge, the Center for Innovative Bridge Engineering (CIBrE) at the University of Delaware is planning to install a permanent long-term structural health monitoring system on the bridge (West, 2005). This paper presents the plan for the program and provides a status report on the monitoring effort.

In order to efficiently handle the responsibilities of developing and managing a large-scale monitoring program, CIBrE has developed an in-depth manual that describes the monitoring program in detail and delineates the responsibilities of the involved parties. It is intended to provide important information relating to the instrumentation plan in the preliminary, construction, initial condition, and long-term condition stages of the project. The manual will to serve as an aid to CIBrE, DelDOT, Figg, the contractor, and the independent reviewer of the bridge, T.Y. Lin International.

The manual is divided into chapters that describe each portion of the monitoring program, as outlined below:

- Chapter 1, Introduction: Describes the purpose of the manual and gives background information on the project.
- Chapter 2, Objective of the Instrumentation Program: Details the objectives of the monitoring program.

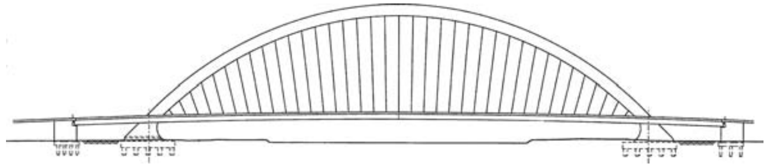

Figure 1. Indian River Inlet Bridge elevation (Modified IRIB Specification and Construction Plans).

- Chapter 3, Responsibilities: Details the responsibilities of the parties involved.
- Chapter 4, Monitoring Project Timeline: Presents and explains the timeline for the implementation of the monitoring project.
- Chapter 5, Stage I – Preliminary Work: Describes the work which must be completed before the start of construction.
- Chapter 6, Stage II – Mechanically Stabilized Earth Wall Construction: Describes the work associated with instrumenting the MSE walls.
- Chapter 7, Stage III – Bridge Construction: Describes the work associated with instrumenting the bridge.
- Chapter 8, Stage IV – Baseline Behavior: Details the required tests that will help capture the baseline behavior of the bridge.
- Chapter 9, Stage V – Long-term Conditions: Details CIBrE's plans to continuously monitoring the bridge during its service life.
- Chapter 10, Instrumentation: Details all the instrumentation that will be used on the bridge.
- Chapter 11, Reporting Expectations: Explains the reports that will be generated during the monitoring program.

The paper will outline, in more detail, some of the key chapters of the monitoring manual developed by CIBrE for the Indian River Inlet Bridge.

The immediate objective of the health monitoring program is to enable the Delaware Department of Transportation to better maintain and operate the innovative new Indian River Inlet Bridge. However, the broader implication of the monitoring program is to contribute to the bridge community's knowledge about bridge behavior during construction and service. In the long term, such knowledge will contribute to better designs and more effective bridge management systems, ensuring that valuable resources are spent where they are most needed.

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Intelligent use of cathodic protection on bridges

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

State-of-the-art on cathodic protection installations and innovative projects

Birit Buhr Jensen
COWI A/S, Kongens Lyngby, Denmark

Erik Stoltzner
The Danish Road Directorate, Copenhagen, Denmark

Cathodic protection has been used for many years to protect steel structures like hulls and sheet piles from corroding. Here sacrificial anodes have been widely used. For reinforced concrete structures below water the same sacrificial anode system using Zink or Aluminium anodes can be used. When reinforced concrete structures placed in the atmospheric zone is to be cathodically protected, the concrete resistance will prevent the protective current form reaching the reinforcing steel. Instead impressed current systems are used with anodes of noble material to serve as electron donator.

The first application in Denmark on a reinforced concrete structure in atmosphere was carried out in 1987. Since then, the design and type of anode and conductive material and use for atmospherically exposed reinforced structures has constantly undergoing changes.

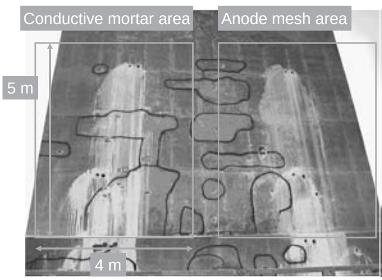
This paper presents different anode systems used in different applications on some of the major Danish bridges.

The Faro Bridges consist of two bridges, constructed using 33 columns in the sea. The bridge was opened to traffic in 1985. As preventive repair strategies the following strategies has been assessed: Membrane to stop further chloride ingress and cathodic protection below water level.

In 1992 two trial installations were applied with the above mentioned repair strategies. The cathodic protection system below water was found not to be able to protect the reinforcement sufficiently above water level up to areas where chloride have ingressed to a critical amount. For this reason a trial installation on the neighbouring column was installed where anodes cast into sawn slots were installed together with a cathodic protection installation below water. The magnetite anode used for the trial installation had sufficient capacity also to protect the neighbouring column below water, and the reinforcement from the new column to be protected was connected to the rectifier used for the trial installation. Anodes in sawn slots were found to be a feasible installation for the above water level as the structure is homogeneous where cover to the reinforcement and location of reinforcement stirrups are concerned. The system has been evaluated latest in 2005 and is found to operate satisfactory.

Based on the relative high chloride concentrations in the depth of reinforcement and the increasing risk of corrosion a cathodic protection installation is planned for in the coming years, starting with an impressed current system below water in 2006 followed by an impressed system using anodes in sawn slots above water in the tidal and splash zone.

The Langeland bridge in Denmark has 20 columns up to 24 m in height and exposed to a marine environment. The bridge was opened to traffic in 1962. After only 15 years of service extensive reinforcement corrosion was detected. Two different repair options have been assessed: Re-skinning of the concrete cover and re-casting this and cathodic protection using anode mesh and concrete overlay. The two different repair options were compared in a life cycle cost analysis and cathodic protection was found the most feasible. The mesh with overlay was selected as a possible cathodic protection system in the life cycle cost evaluation as this has a long proven track record. However new developments, as conductive mortar was considered, as this would also provide an even current distribution, if effective.



The trial installation was conducted on both foundation (over the granite facing) and 5 m up the pier on each side of the column. The conductive mortar anode system is tested under actual Danish marine environmental conditions against mesh with overlay.

The systems will be evaluated in 2006–07. Based on these findings the optimum repair strategy for the entire bridge structure will be prepared.

Along one of Denmarks most traffic loaded motorways with six lanes lies two twin bridges at Karlstrup march. The bridges are constructed as reinforced slab bridges supported on piles. The superstructures are symmetrically constructed around a dilatation pile. Between the abutments and the dilatation pile there are 24 hinged piers. The bridges have been subject to extensive repair works. By a general inspection ongoing corrosion has been found on all four abutments.

Two options of repairing the abutment corrosion problems exist. One is the traditional repair, removal of deteriorated and contaminated concrete, cleaning of reinforcement and casting new concrete. The other repair option is cathodic protection, which is more feasible.

In order to protect the upper and rear part of the abutment wall a 20–22 m long MMO coated titanium anode was inserted through the entire abutment walls. The anode is a cylinder based on 20 mm × 1.5 mm ribbons from Lida and mounting PE spacers each approx. 2 m. The anode is produced particularily for the Karlstrup march abutments. A Ø 42 mm diameter hole was drilled through the abutment, the anode inserted and a conductive gel was injected. For the application at the abutment walls electrical conductivity was adapted. In the operation and maintenance the system is evaluated based on embedded reference electrodes and also it is visually checked that the gel is not disappearing. In addition, a total of 567 anodes were installed on the abutment walls, in lines near the top of the abutment walls and along vertical cracks were deterioration had been detected.

It has by the Danish Road Directorate been documented that cathodic protection can stop corrosion. This has lead to a number of installations not only on structures administered by the Danish Road Directorate but also structures administered by the municipalities and counties. This shows that the Danish Road Directorate has a trend setting function in the Danish society.

It has been aimed at that the installations are being standardized. This in practice has to some extent shown difficult, as new anode systems and new adoptions can turn out feasible. It is the opinion of the authors that cathodic protection shall be used with due consideration and using specialist input. Where limited knowledge exists e.g. on evaluation critierias in the tidal zone, knowledge shall be collected allowing for an optimum repair solution.

With the anticipation that cathodic protection is used where technically and economically feasible, standardized and innovative cathodic protection systems will follow in the years to come. The development will be interesting to follow and participate in.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Cathodic protection as repair option for the Öland Bridge superstructure

R. Sørensen & B. Buhr
COWI A/S, Denmark

A. Maglica
Swedish Road Administration, Sweden

ABSTRACT: The Öland Bridge is a 6,072 m long reinforced concrete girder bridge with 155 spans connecting the island of Öland with the mainland of Sweden. The bridge was opened for traffic in 1972.

In the 1980s and 1990s most of the piers were extensively refurbished due to serious deterioration as a result of reinforcement corrosion. In the years to come the bridge Owner is facing the challenge of rehabilitating the superstructure as well.

This paper presents the considerations regarding finding a durable repair option for the Öland bridge superstructure given the requirement of a minimum residual service life of the bridge of at least 70 years.

As part of the investigation the condition of the bridge structures were evaluated based on analysis of the data from existing previous inspections and condition assessments, but taking the size of the bridge into account in general only a very limited data basis were available. The available data have been extrapolated to represent todays situation. The prevailing deterioration mechanism for the reinforced concrete structures is assessed to be chloride initiated corrosion, the data basis is however not adequate to allow for a reliable estimate for the service life. In conclusion additional systematic concrete assessments are needed and condition assessments are a vital part of the 70 years maintenance strategy allowing for a verification of the condition of the structures.

The evaluation was conducted taken several factors into consideration while evaluating possible repair options. It is among other required that the traffic disruptions and expenses related to traffic arrangements and access equipment shall be minimised, the maintenance and repair methods have to be well documented and the repair durable with a long service life and the optimum time

Figure 1. Ölands Bridge.

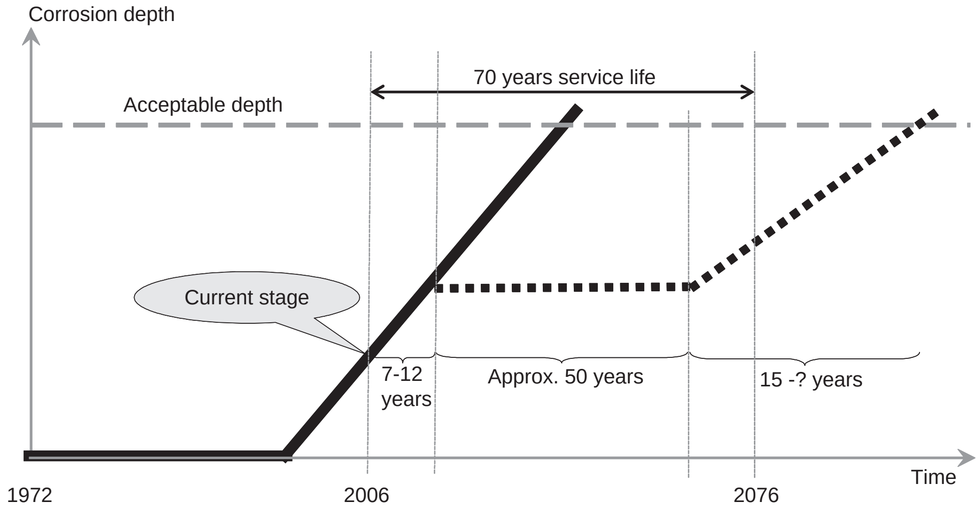


Figure 2. Principle for time schedule for repair strategy with cathodic protection for main girder.

for implementation of the rehabilitation is to be determined in order to minimise the costs. It is considered that cathodic protection is an optimal repair and maintenance strategy for the main girders of the low part of the low-level bridge and for the edge beams including railing posts, technically as well as financially.

The deterioration of a concrete structure is often describe in two phases, the initiation phase where e.g. chlorides penetrate to the reinforcement to a critical content, and the propagation phase where the actual deterioration takes place, until the acceptable degree of deterioration is exceeded.

For the main girder a service life of 70 years is obtained in by prolonging the propagation phase by installing cathodic protection. By implementing cathodic protection on a concrete structure the environment around the reinforcement to be protected is expected to improve due to the electro migration of chloride ions towards the anode (depleting chloride ions immediately around the reinforcement), evolution of hydroxyl at the reinforcement (raising the alkalinity) and furthermore the additional cover to the anode on the concrete surface will delay the chloride ingress, as well as limiting the access of oxygen for the corrosion process.

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Benefits and challenges using cathodic protection from an owners point of view

Erik Stoltzner & Carsten Henriksen
The Danish Road Directorate, Copenhagen, Denmark

Birit Buhr Jensen
COWI A/S, Kongens Lyngby, Denmark

ABSTRACT: In 2001 the Danish Road Directorate established a project team with the objective of evaluating cathodic protection systems among other with respect to the systems' ability to stop or prevent corrosion and the service lifetime of the systems. Furthermore, cathodic protection was evaluated on a technical and financial basis as an alternative method to other repair methods.

The Danish Road Directorate administers 14 structures with cathodic protection applied, of which one structure (Aggersund Bridge) from 1998 has been administered by the County of North Jutland.

The findings and the conclusions of the project team were published in a report in 2003. The report is published in printed as well as electronic version. The latter, in Danish, however with an English summary, can be found on the internet on the Road Directorates homepage under publications www.vd.dk (direct address: www.vd.dk/wimpdoc.asp?page=ocument&objno=72499).

When reinforcement in a concrete structure corrodes, an electrical circuit is established. The electrical corrosion current runs in a circuit between the anode, where the corrosion takes place (dissolving of ion) and the cathode, where the oxygen reacts.

The principle by cathodic protection is to turn the entire metal surface cathodic by altering the current, hereby forcing it to run from an external anode. Practically, the external anode has to be located in the electrical circuit, i.e. in the same electrolyte (concrete pore solution) as the reinforcement.

Cathodic protection installations require operation and maintenance throughout the service life of the system installed. Rectifier systems equipped with computer and modem connection show

Figure 1. The Faro Bridges. Casting of slots with inserted anode ribbons. November 2001.

great perspectives in minimizing maintenance costs as fewer service visits are required on site. It also allows for a rational data collection and management of the systems installed giving the Owner greater confidence in system performance.

The experiences from the cathodic protection installations on structures administered by the Danish Road Directorate show that cathodic protection technically is a method able to stop corrosion. Based on measurements and break-ups in areas, where the protective level is assessed to be effective, no indications on ongoing corrosion have been found.

For one structure ongoing corrosion was found where the protective criterions set forward had not been met. These areas were characterised by repair works which had not been conducted according to the specification, or where damages had not been repaired prior to energizing the cathodic protection system.

On columns/walls, where part of the structural element is below ground level, and the cathodic protection installed above ground level, it had resulted in insufficient degree of protection for the ground and below ground level. Future cathodic protection installation used as corrective repairs is recommended to include below ground level protective measures.

The experience from the cathodic protection systems installed on the Road Directorates structures furthermore shows that cathodic protection does not lead to damages in the concrete, as a result of the applied current itself.

For titanium mesh in overlay shrinkage cracks have been observed and a reduced adhesion as a result thereof have been found. In connection with future installations it is recommended that shrinkage is sought compensated for, and it is recommended to section the works.

Based on the technical experiences including an assessment of the remaining service life of the established systems it is found that cathodic protection generally is a technical useful alternative to other repair methods.

It is found that cathodic protection can be an alternative to other repair methods based on a financial evaluation, among other as preventive measure. Cathodic protection used as preventive measure especially for coastal structures has proved attractive.

With the increasing number of installations there has been an increasing need for automatic data collection, data handling and system adjustment. For the newest installations the Road Directorate has therefore in the tender documents required that the rectifier is installed with a computer and data transmission system allowing for remote data handling and management.

The positive experiences, technically as well financially has resulted in an anticipation from the Danish Road Directorate that cathodic protection in future is considered as repair method on an equal footing with other repair methods. The final selection of repair method will be an economical selection between methods, all found technically relevant in the actual case. The selection of repair method that will result in preparation of a repair project and following tender is hence made based on existing experience with unit prices and performance methods.

For traditional repair works there is a comprehensive knowledge database, which is not the case with cathodic protection to date.

It has therefore been wished for that cathodic protection as method is developed further leading to a standardized deliverable in line with e.g. sprayed concrete enabling the designer and the owner to prepare a realistic budget in the preliminary evaluations of which method to select.

It has by the Danish road directorate been concluded that cathodic protection is able to stop and/or slow down the corrosion rate. It has in addition been shown that cathodic protection is financial feasible in a number of cases, e.g. for cathodic prevention of marine structures.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Cathodic protection of anchorages in deteriorated post-tensioned bridges

Peter H. Møller & Allan Hojgaard
RAMBOLL, Virum, Denmark

Henrik O. Nielsen
The Danish Road Directorate, Ministry of Transport, Copenhagen, Denmark

ABSTRACT: Traditionally, repair of reinforcement corrosion in concrete structures requires replacement of the concrete around the corroding area. However, locally in highly stressed areas (column feet, post-tensioning anchors etc.) replacement of the concrete is often so problematic that other repair methods, like cathodic protection (CP) can offer big advantages. Often standard CP-systems are not suitable for such local repairs because of high initial system-costs and the need of frequent control.

This article describes optimisation of CP in connection with local repairs using corrosion repair behind post-stressing anchors on a highway bridge as an example. Leakage through a defective membrane had led to chloride-induced corrosion around the anchors. Membrane replacement stopped further ingress from water and chlorides, and corrosion was originally planned to be stopped by traditional repair methods. These repair methods would have included both replacement of concrete around the visibly corroding areas (corrosion protection) and replacement of the chloride containing concrete adjacent to the visibly corroding areas (corrosion prevention). It was possible to repair all visible corroding areas, but behind the anchors removal of all concrete containing more than 0,05% (weight of concrete) chlorides would have made very expensive stress relief operations necessary. Instead of using traditionally repairs based on replacement of concrete further corrosion was prevented by CP.

Only 8 anchors needed CP. CP-systems with impressed current have relatively high general costs, regardless of system size, for power supply, systems control and cabling. The high-strength (1800 MPa) prestressing cables ending in the middle of the anchors are prone to hydrogen embrittlement, which means that hydrogen formation from CP-systems must be avoided. CP-systems with impressed current must therefore be carefully controlled by potential measurements. The high general costs and problems of hydrogen formation from systems with impressed current made systems with sacrificial anodes much more favourable, and a system using sacrificial anodes was therefore chosen.

The anchors are of vital importance to the structural capacity of the bridge. Therefore the corrosion condition of the anchors was monitored. If the CP-system works 100% correct corrosion will be controlled by the CP-system, but if the system is unable to provide full protection corrosion will also be controlled by the concrete environment around the anchors. The aggressiveness of the concrete environment is reduced by removing most of the chloride-contaminated concrete and by allowing the concrete to dry out (future water ingress is stopped), and the demands to the CP-system are thereby also reduced. Concrete environment can potentially be improved so much that there is no need for CP at all.

When concrete dries out corrosion rate will decline but so will the range of the anodes of the CP-system. This means that when the need for CP is reduced the possibilities of achieving full protection can be reduced. In order to get at better view of the corrosion both the effect of the CP-system and the need for CP was monitored.

External anode connections and reference cells enables monitoring of CP by depolarisation measurements. On built-in steel rods not connected to the CP-system the general corrosion conditions were measured by means of polarisation resistance, macro cell current, potentials and resistance. The monitoring system is very economical. It consists of 4 reference cells, 12 metal rods and cabling, and the system is without any stationary measuring instruments.

On the basis of the monitoring results from the first 6 months of operation the 4 monitored anchors can be divided into 2 groups. Group 1 consisting of 3 of the 4 anchors and group 2 consisting of the last of the 4 anchors. At group 1 anchors the CP-system achieves approx 45 mV depolarisation, but at the group 2 anchor only 12 mV depolarisation was achieved. For all anchors a current density of approx 0,5 mA/m^2 was measured, and power consumption is so small, that the whole system could be run on 1 AA battery for more than 2 years. Regarding the need for CP, at group 1 anchors 7-day off-potentials of approx -160 mV ($Cu/CuSO_4$) was measured, and on unprotected steel a corrosion rate of 40 μm/year (pulse method) was measured. At the group 2 anchor the corresponding values of -210 mV and 240 μm/year was measured. At all 4 anchors a marked improvement of the CP-efficiency and at reduced need for CP was measured during the 6 months of operation.

At the renovated area a very simple and almost maintenance free system for cathodic protection (CP) of local areas was designed. After 6 months of operation it is concluded that:

– It is possible to establish local CP and a detailed surveillance at very low costs.
– Evaluation of the need for CP requires detailed information of the corrosion conditions.
– Monitoring makes it possible to evaluate errors from the CP system. To unimportant errors action can be omitted, to errors of small importance action can be delayed and important errors can quickly be addressed. Thereby maintenance can be optimised and cost efficient.
– If installation of CP is accompanied by corrosion reducing renovation (protection from moisture, replacement of chloride containing concrete) CP can be unnecessary and when monitoring displays this, further costs for maintenance and inspection of the CP-system can be saved.
– Renovation including corrosion-reducing precautions can affect corrosion for so long that the performance of the CP-system cannot be fully evaluated before 1–2 years of operation.

The results indicate that the embedded sacrificial anodes can prevent corrosion in the concrete adjacent to the repaired area if water ingress is stopped and concrete with high chloride content is replaced. But the anodes can hardly prevent corrosion in moist concrete with high chloride content (>0,1%). In the actual case there might be areas where protection is insufficient and an improvement of anode output, e.g. from battery power, can be necessary. Because this possibility was considered in advance this improvement can be established quite cost effective. However, final evaluation of the need for supplementary precautions can first be made in 1–2 years. All in all the expenses for establishing and maintaining the CP-system will be much lower than traditional repairs, even if improvement of the CP-system becomes necessary.

Power output of sacrificial anodes is so low that battery powered anodes might be a very attractive alternative. Power supply could be central or from built-in batteries. However, near high tensioned cables, power from batteries will make potential based control systems necessary, which can reduce the advantage significant at areas close to high tensioned cables.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Cathodic protection of the west bridge caissons and piers

Peter H. Møller & Mette E. Andersen
RAMBOLL, Virum, Denmark

Ernst Laursen
Storebaelt, Storebaeltsvej, Korsoer, Denmark

ABSTRACT: The paper describes cathodic corrosion protection from sacrificial water anodes at concrete structures. Water anodes can protect the water-covered parts of concrete structures from corrosion, and reduce corrosion in the lower parts of the splash-zone. Corrosion in the splash zone is a well-known problem, but the corrosion reduction from water anodes in this zone is not very well documented. Corrosion in the water-covered parts of the structures is generally considered to be a minor problem. However at many structures underwater corrosion cannot totally be ignored, and therefore underwater inspection and eventually underwater repairs must be performed. Because these underwater works are quite expensive the risk of underwater corrosion can cause considerable expenses. Cathodic protection (CP) from water anodes can therefore be advantageous in the reduction of damages in the splash-zone and reduce the need for inspection and repair under water.

CP from water anodes can inexpensively and almost maintenance free be established with sacrificial anodes. High anode consumption just after installation indicates a short anode lifetime, and therefore high maintenance costs. However, from the very limited oxygen diffusion possibilities in water-saturated concrete a dramatic reduction in anode consumption over time could be predicted. This reduction would cause significant reductions in overall maintenance costs. This reduction is however not well documented, and the overall costs for CP using sacrificial anodes are therefore uncertain. The additional benefits of corrosion reduction in the splash-zone are not well described and therefore the overall advantages of water anodes are still somewhat uncertain. Since there is limited information regarding both costs and benefits using sacrificial anodes it can be difficult to decide if the establishing of a sacrificial water anode system would be advantageous.

By presenting results from long-term experience with sacrificial anode systems we hope to reduce some of these uncertainties making it easier to compare these anodes to other methods of corrosion control. The results presented in this paper come from an inspection of the sacrificial CP-system of the 62 concrete piers of the 7 km Storebaelt West Bridge, in all 120.000 m^2. The CP system was installed during construction to prevent corrosion in casting-induced cracks under water. After installation corrosion in faulty repaired areas were found, and these areas are now also protected. In the lower splash-zone ($-0,5$ $+1,0$ m) minor degradation signs in form of increased chloride ingress and minor surfaced based attacks from freeze/thaw or sulphates were found. These signs are within the predictions of the long-term degradation models, but given the uncertainties from these models problems from corrosion in the lower splash-zone cannot be ruled out. The CP-system could therefore have an extra function in delaying the onset of corrosion in the splash-zone. The inspection included evaluation of: remaining anode lifetime, the anode protection range and the overall CP-effect. Furthermore methods for inspection of water anodes were evaluated.

Usual CP-inspection methods are optimised for steel constructions, but the special conditions in the self-desiccating Storebaelt concrete made method adjustments necessary. Determination of true instant/off potentials and current anode consumption was made possible by changing the direct anode reinforcement contact with separate measuring cables. Average anode consumption and remaining anode volume was determined from cross section measurements. By comparing the current and the average anode consumption it was possible to evaluate how anode consumption

changes over time. The protection range of anodes was evaluated by measuring current flow between the piers. The effect of CP in the splash-zone was inspected by measurement of depolarisation and evaluation of the total polarisation at level 0 to $+2$ m.

The results from the inspection have shown:

- The anodes give full corrosion protection under water. Repair of the underwater damages can therefore be avoided and the need for underwater inspections is reduced.
- Current distribution from the anodes is several hundred meters. Anode positioning is therefore not very critical, and the CP system is very robust to failure of anodes or whole groups of anodes. When new anodes are needed, only the anodes at the easiest accessible areas need to be changed.
- Current output for maintaining protection is now reduced to 0,5 mA/m^2. The anode consumption for the protection of 1 pillar (2000 m^2) is now only 5 kg Al/year (20 $/year).
- The expected lifetime of the system is min. 30–40 years without maintenance and it should be possible to design the anodes to give the pillars lifetime ($\sim$100 years) protection.
- Protection can be inspected by simple measurements without need for divers' assistance.

The inspections indicate that current flow to the water-covered parts of the concrete is very low, and most of the current will be used for protection of the very vulnerable parts at the splash-zone. The inspections indicate that the anodes can provide a significant postponement of corrosion in the splash-zone, but the exact extent of this postponement cannot be estimated precisely on the basis of the inspection.

For sacrificial anode systems installation costs are usually relatively low while maintenance costs (inspection, changing of anodes) are often regarded as quite high. The presented results from longterm experience with sacrificial anode systems on water covered concrete constructions have shown that maintenance costs of such systems are potentially very low. This gives sacrificial anodes advantages compared to cathodic protection using impressed current, but especially compared to a strategy of underwater inspection and repair of corroding areas.

Corrosion in water-covered parts of concrete constructions is however a relatively minor problem, compared to corrosion in the splash-zone. So, if the indications of water anodes being able to postpone corrosion damage in the splash-zone seen from the data presented here are true, the benefits of installing water anodes will rise dramatically. The costs for cathodic protection with water anodes is low compared to the costs for even quite local concrete repairs in the splash zone, so even a limited postponement of corrosion will make installation of cathodic protection with water anodes attractive. However, to fully evaluate these promising possibilities of using water anodes to postpone corrosion damage in the splash-zone, further investigations will be necessary.

Bridge evaluation using field testing

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Experimental and numerical dynamic analysis and assessment of a railway bridge subjected to moving trains

M. Majka & M. Hartnett
Department of Civil Engineering, National University of Ireland, Galway, Ireland

J. Bień & J. Zwolski
Institute of Civil Engineering, Wroclaw University of Technology, Wroclaw, Poland

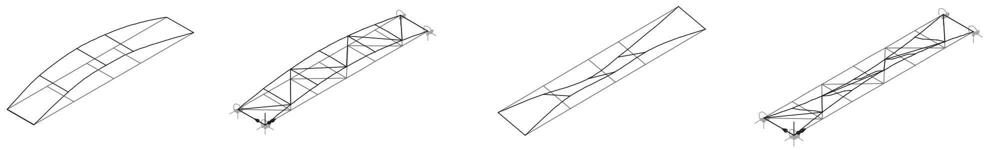
ABSTRACT: Experimental and numerical dynamic analysis of a typical metallic railway bridge subjected to moving trains is presented in the paper. The study was carried out to verify the numerical model developed by the authors and to assess the response of the bridge to dynamic load over a range of speeds. Ambient vibration test of the bridge was performed and the finite element model was calibrated to match the modal parameters identified from the experimental records. Dynamic response test was also carried out to provide the data for direct comparison with the calculations. In general, the verification was successful, however, some difficulties encountered in the process are discussed. The calibrated model of the bridge was used in the analysis and assessment of the dynamic behaviour of the bridge over a range of trains' speeds. The bridge performed well up to the maximum speeds of trains considered in the analysis, however showed excessive accelerations for higher speeds.

1 EXPERIMENTAL TESTING OF THE BRIDGE

The railway bridge investigated in this study consists of two parallel multispan structures, one under each track, each of which comprises of six simply supported spans of equal length. Each span is a wrought iron, riveted plate double-girder deck spanning 11.28 m, with total length of each span 11.58 m and total length of the bridge 70.08 m. Ambient vibration test was carried out in order to identify the modal parameters of the bridge and to calibrate its numerical model. Dynamic response test was also carried out to establish displacement and acceleration response of the bridge subjected to passages of two types of passenger trains (*Train I* and *Train II*). The recorded data were later used for comparison with the results of the numerical simulations performed using the DBTI package developed by the authors.

2 NUMERICAL MODEL OF THE BRIDGE-TRAIN SYSTEM AND ITS CALIBRATION

Numerical model of the bridge-train system was created and analysed using the Dynamic-Bridge-Train-Interaction (DBTI) package that is being developed at the National University of Ireland, Galway. The DBTI package allows for finite element modelling of bridges in combination with multibody models of vehicles. Bridge under consideration was modelled with twonode beam type finite elements that are capable of simulating three-dimensional displacements of the structure. Trains were represented by a composition of three-dimensional multibody railway vehicles with twenty seven degrees of freedom. Each railway vehicle consisted of rigid bodies, lumped masses and spring/dashpot units. The resulting two sets of the equations of motion were integrated numerically by applying the Newmark's method combined with an iterative procedure to take the bridge-train interaction into account and to improve the accuracy of the solution. Despite the fact that the package is tailored for the transient dynamic analysis of bridges subjected to moving loads, it is also capable of performing static and modal analysis of the bridge-train system.



Meas.f_{m2} = 21.875Hz   Calc.f_{c2} = 21.484Hz   Meas.f_{m4} = 38.312Hz   Calc.f_{c4} = 38.544Hz

(a) 1st flexural mode shape – *MAC* = 0.980   (b) 1st torsional mode shape – *MAC* = 0.939

Figure 1. Natural frequencies and corresponding mode shapes measured (identified) and calculated with the use of the calibrated finite element model.

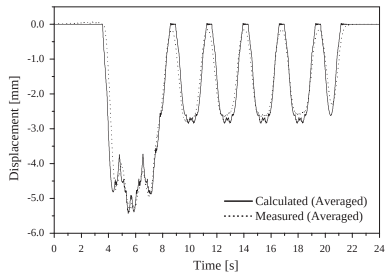


Figure 2. Calculated and measured mid-span displacement of the bridge under *Train I*.

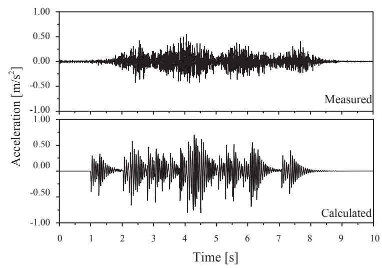


Figure 3. Calculated and measured mid-span acceleration of the bridge under *Train II*.

The data gathered during the ambient vibration test were postprocessed and the modal parameters of the bridge were identified using the pick-picking technique. That allowed for calibration of the model of the bridge to match the identified modal parameters, see Figure 1.

3 RESPONSE OF THE BRIDGE TO MOVING TRAINS AND CONCLUSIONS

Numerical analysis of two train passages recorded during the experimental test was performed using the DBTI package and comparison between simulated and measured displacements is shown in Figure 2. Both figures show good correspondence between the sets of data. The acceleration traces were also compared as shown in Figure 3. The compared data match relatively well, however the numerical analysis overestimated the maximum accelerations, which may result from only approximate values of damping used in the analysis. Nevertheless, the overall correlation between the calculated and measured responses is satisfactory and proves the validity of the DBTI package.

In the next step, the more extensive dynamic analysis of the bridge was carried out to assess the dynamic effects caused by *Train I* and *Train II* over wider range of speeds than recorded during the tests. The bridge showed the dynamic amplification over the static response as high as 13%, whereas the maximum accelerations were kept below the allowed limit of 5 m/s^2 for the maximum real speeds of the two trains considered in the analysis. However, at higher speeds, the bridge showed excessive accelerations that may endanger the safety of the railway traffic and should be considered in the case of introducing faster trains on the line or upgrading the bridge. Generally, numerical dynamic analysis in conjunction with experimental tests has proven to be a reliable and efficient method of dynamic assessment of existing railway bridges.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the support provided by Irish Rail, Enterprise Ireland and the Polish State Railways (PKP) for carrying out this research.

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Effect of bridge live load based on 10 years of WIM data

M. Gindy
University of Rhode Island, RI, USA

H. Nassif
Rutgers, The State University of New Jersey, NJ, USA

The objective of this study is to determine the actual truck loads on US highways. The need for reliable truck weight data has been recognized by many State DOT's and the Federal Highway Administration (FHWA). For many years, trucks were measured at a limited number of stationary scales (truck weigh stations) that are located on major highways. However, there is a justified concern that the heaviest trucks knowingly avoid the scales [3] and as a result the weigh station data is biased to less heavy vehicles. Accurate representation of the actual load spectra on State and Federal highway bridges will reduce the uncertainty in the evaluation of bridge load carrying capacity, prediction of deterioration (corrosion) rates and bridge remaining life (fatigue). The development of the Load and Resistance Factor Design Bridge Design Code [1] was based on a truck survey at a site in Ontario in 1975 which consisted of 9,250 trucks that appeared to be heavily loaded [4]. This relatively limited dataset is now 30 years old and, given the continuous change of the trucking industry, these statistics have undoubtedly changed. In this study, a statistical basis is developed for the live load model for bridges in the State of New Jersey. Major load parameters that affect bridge design and evaluation (i.e. truck volume, type and weight) are considered. The database consists of truck load information collected from 33 WIM sites located throughout the State of NJ over an 11-year period (1993–2003).

Truck loading is site-specific and depends on factors such as truck volume, local industry and economic activity. Site variations of truck statistics are analyzed using the June 2003 datasets from 33 WIM sites (1,146,879 trucks). It is observed that Class 9 vehicles are by far the most occurring truck type for all site conditions and tend to increase with increasing truck volume; 38% (Light), 57% (Average), and 78% (Heavy). The volume of single and multi-trailer trucks (C8-C13) is found to increase while that of single unit trucks (C5-C7) decreases with increasing site volume. Total truck weight statistics are also affected by site variations. The average, 95th percentile (W_{95}) and maximum truck weights are normally distributed with mean and COV of 46 kips|15% (205 kN), 81 kips|18% (360 kN) and 170 kips|18% (756 kN), respectively. It is also observed that 8% of trucks, on average, operate above the legal limit of 80 kips (356 kN), although some of these vehicles may in fact be legal (permit vehicles).

Besides site variations, time variations of statistics are particularly important for truck traffic simulation and for the development of bridge live load models. Oftentimes, time-limited data (2-wks, 1-mo) is used as the basis for simulation of longer time periods and 75-year extrapolation of bridge effects. It has been found that the time and length of the base dataset effect future predictions [2]. Time variations of trucks statistics are investigated using an 83-month dataset containing over 2.8 million truck records for Site 195, a 4-lane interstate highway in a rural area categorized as an Average volume site. The ADTT is observed to steadily increase at an annual rate of 7.7% while the percent of trucks operating above legal load remains relatively unchanged at 13%. The volume of Class 7 vehicles (four or more axle, single unit trucks) shows a dramatic increase from 3% in 1993 to 13% in 2003 while the volume of multi-trailer trucks with six or more axles exhibits only a slight increase. Figure 1 shows the variation of total truck weight statistics with time. It is seen that although the mean and W_{95} statistics remain relatively constant, the maximum truck weight

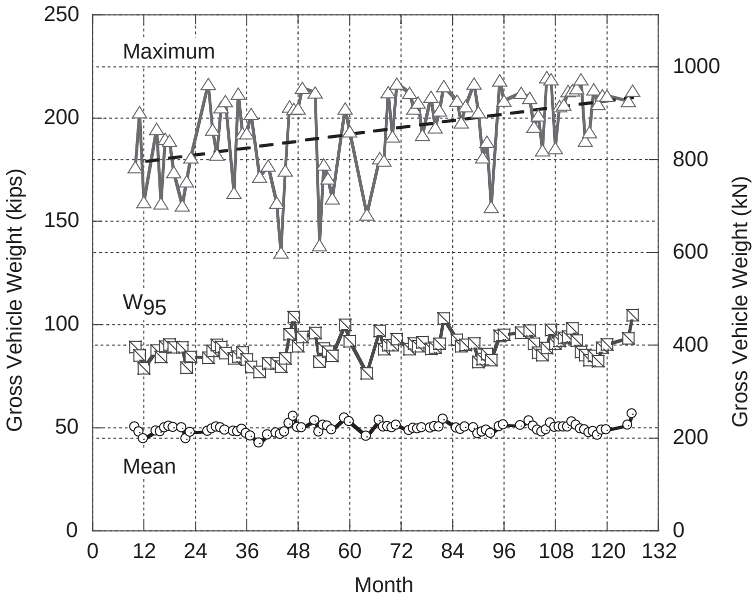

Figure 1. Truck weight variation during 1993–2003 (Site 195).

increases at a rate of 3.5 kips/year (15.6 kN/yr). It is also observed that the percent of trucks in the upper 5% (by weight) of Class 9 vehicles nearly doubles over the past 10 years (4% in 1993 to 8% in 2003).

Truck loading in the State of New Jersey has been statistically described. The extensive database used in this study provides a distinct opportunity to measure the uncertainty introduced by site and time variations. These variations become increasingly important for truck traffic simulation and for the development of bridge live load models. Two major parameters in the evaluation of existing bridges and the design of new bridges are the magnitude and frequency of loads. The truck load statistics and trends resulting from this study are used to describe the live load spectra from a 10-year perspective. It provides an opportunity for reflection and forms a stronger foundation for future predictions.

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Evaluating ultimate bridge capacity through destructive testing of decommissioned bridges

J. Ross, J. Righman, M. Chajes & D.R. Mertz
Center for Innovative Bridge Engineering, University of Delaware, Newark, Delaware, USA

T. Zoli
HNTB, New York, USA

J. Volk
Delaware River and Bay Authority, New Castle, Delaware, USA

ABSTRACT: The strength capacity of bridges predicted by most current design codes is based on the philosophy of computing the capacity of individual members for various force effects (e.g., negative bending, positive bending, shear). However, bridges (especially those with steel superstructures) have significant load redistribution capability; thus, the ultimate load that may be resisted by the system is likely significantly higher than the ultimate load that is computed based on the strength of individual members. While the ultimate capacity of individual bridge members is fairly well understood, the ultimate capacity of a bridge "system" is not. Understanding this system behavior is complicated by the fact that load redistribution occurs only after the structure begins to behave inelastically, which means that full-scale tests of in-service bridges are not possible. Therefore, the best candidate for this type of ultimate capacity test is a decommissioned bridge. This paper discusses two such ultimate capacity tests (one completed and one planned) of bridges owned by the Delaware River and Bay Authority, which is currently in the process of decommissioning several bridges due to roadway realignment. This paper provides details of the testing program, as well as the test results to date.

The objective of the destructive bridge tests discussed herein is to provide insight into the plastic behavior of multi-girder bridges by evaluating the load redistributed to other girders when one girder begins to yield. It is well known that the inherent ductility of steel members allows them to resist loads in excess of those predicted using elastic methods, through distribution of load to less heavily stressed regions. Plastic design methods were first introduced in 1971 (ASCE), and since then much research has been directed at understanding the redistribution of load that occurs within a given member (i.e., along the longitudinal direction of a bridge). However, the redistribution of load from one heavily stressed member to another member (i.e., the transverse direction of a bridge) has not received as much attention and is therefore the focus of this work.

The Delaware River and Bay Authority (DRBA), in reaction to growing needs and deteriorating infrastructure, has begun to realign the roadways near the Delaware Memorial Bridge to permit easier traffic flow. DRBA is also removing existing bridges to minimize construction cost and, eventually, maintenance and inspection costs. The removal of these bridges provides a unique opportunity to destructively test actual bridges in ways that will allow researchers to gain useful information.

Bridge 11 was the first of eight bridges scheduled for demolition. This bridge was destructively tested in 2005 by the University of Delaware and HNTB Since this was the first test to failure and the timetable was very short, the goals were primarily to study the plastic behavior of the girders and provide the requisite experience to establish a test program for the other bridges scheduled for demolition.

The insight gained from this initial test is now being applied to planning of the destructive test of Bridge 7R. Using a refined set of testing procedures, the second destructive test will provide additional information on the ultimate capacity and plastic behavior of the bridge system. The particular focus of this test is on gaining a better understanding of the changes in the load distribution that occur as plastic behavior progresses. Along with the field testing of Bridge 7R, a detailed finite element analysis (FEA) will be used to predict results prior to destructive testing. The FEA and experimental results will later be correlated, and any necessary refinements to the FEA will be performed. The FEA procedures developed will then serve as a tool for conveniently studying the ultimate behavior of other bridge systems.

Bridge 7R is a steel girder bridge with a composite concrete deck consisting of three spans. The bridge was designed in 1961 and built soon after. The 45-year-old bridge consists of a four-girder system. The bridge girders and abutments are on a 37° skew to the roadway. The main span is 32.1 m in length with two approach spans of similar length. The bridge is in relatively good condition with minimal steel deterioration, concrete spalling, or potholes.

Recently a diagnostic test of Bridge 7R was performed using a fully loaded three-axle dump truck. The objective of this test was to collect data at locations of interest in order to develop a better understanding of how the bridge is performing in its current state.

Preliminary observations from the diagnostic test of Bridge 7R include the following. The peak strain due to the truck loading was in the range of 60 microstrain, which occurred in Girder 2 during pass 1. As predicted, the data shows that the parapets on both sides of the bridge are providing a significant amount of strength to the superstructure. This is evident based on the lower strain values for the exterior girders. The gages placed near the support on Girder 3 show the presence of support fixity in this girder. Since gages were not placed at other supports, the amount of fixity at these locations will be determined by further investigating of the strain results. The maximum vertical displacement at midspan that was recorded was approximately 0.33 cm (which corresponds to L/6831).

The evaluation of Bridge 7R will include a detailed finite element model and analysis. The geometry and material characteristics will be configured in the FEmap software program. Within the model, the characteristics for steel and concrete during elastic and plastic behavior will be incorporated. Once the model is created, the ABAQUS analysis program will be used to compute the desired responses of the bridge model using non-linear solution algorithms. These responses will then be compared to the results of the diagnostic test. Based on this test data, we will be able to correct and/or calibrate the FEA model to the actual bridge.

The testing of Bridge 7R will be performed in a more efficient manner than that of the previous destructive test. The load will be applied directly to the bridge deck by the use of ground anchors and tensioning jacks. Ground anchors capable of applying the determined load, plus an additional factored load to ensure failure, will be installed beneath the bridge. Once testing is performed, the collected data will be reviewed and compared to the predicted results of the FEA model.

The testing of Bridge 7R will contribute to the bridge community's knowledge about the plastic behavior of steel bridges, load redistribution of bridge systems, and methods for conducting destructive tests. Many important details related to performing tests like these were learned on the test of Bridge 11. With this information as a foundation, a detailed plan for the testing of this bridge has been developed. The diagnostic test will reveal how the current bridge is performing in service. This information, coupled with a complex finite element analysis of the structure, will allow predictions to be made about the test to failure. Once these elements of the project have been completed, the next phase will be to destructively fail Bridge 7R. The data collected from this test is very important, since these types of test rarely occur. The actual amount of load redistribution through a bridge once an upper yield strength has been reached is, for the most part, unknown. As a result, it is hoped that the modeling and ultimate testing will answer questions about the ultimate design strength of bridges. In the future, other tests can be compared to these tests to refine design considerations and improve the overall performance of our bridge infrastructure.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Fatigue performance of steel girder bridges based on data from structural monitoring

H. Nassif, J.C. Davis & N. Suksawang
Rutgers, The State University of New Jersey, NJ, USA

ABSTRACT: The evaluation of existing bridge for fatigue performance is important in the effort to deal with the deteriorating infrastructure. The effort to prioritize bridge repair and rehabilitation options will highly depend on the identification of live loads and their effects. Bridge live load effects vary for different components and structural details. In many cases, analytical methods do not allow for an accurate estimation of load, in particular the load distribution and actual stress ranges. Structural health monitoring and field testing can be very effective in the evaluation of bridge performance at the serviceability limits. Truck load spectra are site-specific with site to site variations depending on the function of the roadway e.g. local connector, urban expressway, or interstate highway. Additionally, the seasonal variation in truck weight and volume must be investigated. Typically, fatigue analysis uses short term data taken for a few days. The data is then linearly extrapolated to longer periods. Comprehensive fatigue analysis must include site truck load spectra and bridge stress cycles.

The objective of this experimental program is to gather information about the actual truck traffic over various bridges in the State of New Jersey and their responses in terms of deflections and strains, due to such loads. The field monitoring system consists of a bending plate weigh-in-motion system and a fatigue data collection system. The fatigue system records 24 channels of strain data when triggered by an overweight truck signal from the WIM system in addition to continuous rainflow stress cycle counting. Moreover, data from monitoring system is used to verify predictions from analytical and statistical methods. The Doremus Avenue Bridge is a composite steel slab-on-girder construction. As part of a comprehensive field investigation, all ten girders of the 3-span continuous unit were instrumented to measure strains, overall span deflections, bridge vibrations, and truck weights. Bridge response due to normal truck traffic is continuously monitored and recorded for long term (5-years) evaluation of the bridge. Various loading patterns, including single, following, and side-by-side, using heavily loaded 5-axle trucks are considered during testing.

The Doremus Avenue Bridge was anticipated to be a national test bed for the implementation of the LRFD design specifications. As such, the construction contract included provisions for the equipment and its installation. For the purpose of fatigue monitoring two main systems were designed: the long term strain monitoring system, or Fatigue System, and the Weigh in Motion System (WIM). Figure 1 illustrates the instrumentation set-up for the Doremus Avenue Bridge.

1 RESULTS

Currently 20 months of truck weight data and bridge response data have been collected. The damage is estimated using the Palmgren-Miner Linear Cumulative Damage Rule. A typical hourly equivalent stress for 9 months of data in the bottom flange, Span 2 Midspan Girder 9, on the Doremus Avenue Bridge is shown in Figure 2. An estimate of the cumulative damage for a typical location on the Doremus Avenue Bridge is shown in Figure 3. The effect of the limiting truck volume can be seen just prior to year 75 on the plot. After the limiting volume is reached, the damage rate becomes linear. It can also be seen that the cumulative damage for all detail categories is below the designated failure limit of 1. This is not surprising since the bridge design was suited to the heavy truck volumes.

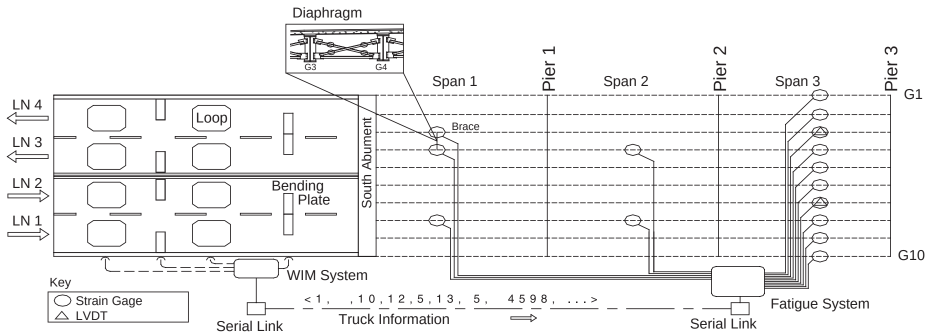

Figure 1. Instrumentation layout of the Doremus Avenue Bridge.

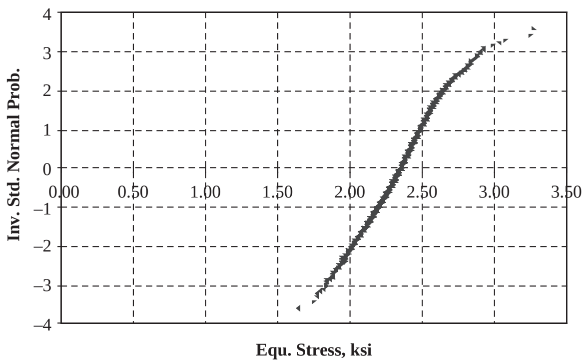

Figure 2. Hourly equivalent stress for 9 months of data. Bottom flange Span 2 Midspan Girder 9. (1 ksi = 6.895 Mpa).

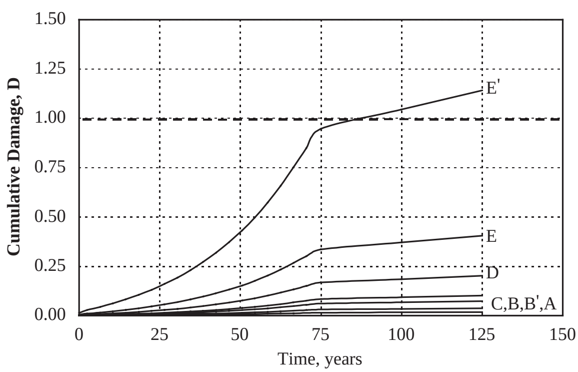

Figure 3. Cumulate damage estimation for a typical bridge detail by category.

2 CONCLUSION

A bridge fatigue monitoring system was introduced that records not only structural stresses, but truck loads. A fatigue life estimation procedure was applied to the stress data and a prediction of the pre-cracking life of the bridge was obtained. The system presented can be applied to other bridges to measure fatigue damage across different regions. Maintenance and replacement decisions could then be made based on remaining life calculations.

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Field test on the noise and the vibration of expansion joint

J.W. Kwark, J.W. Lee, W.J. Chin & Y.J. Kim
Korea Institute of Construction Technology, Goyang, Korea

ABSTRACT: A large variability of expansion joints are used according to the type of bridge. Observing their increasing utilization since 1990s, finger type and rail type joints are predominant in short to medium span bridges, while special type joints like rolling shutter are mainly used in long span bridges. The recent recognition of the importance of maintenance and the increasing volume of traffic are now turning the main selection criteria onto the simplicity of management and replacement. In Korea, the repair or replacement of expansion joints occur after a relatively short period of 5 to 10 years in spite of the careful attention given to maintenance by the relevant managing authorities. The absorption of the continuously increasing traffic and the minimization of indirect social costs provoked by traffic control are requiring the installation of products exhibiting satisfactory performances and securing reliability during installation and replacement.

Recently, the growing social demand for comfortable environment resulted in environment grievance in the surroundings of bridge related to the degradation of riding comfort and the loss of riding safety due to noise and vibration produced by the expansion joints. Even if studies are currently led to suppress expansion joints from bridges to respond to such situation, implementation is limited to several short to medium span bridges from which it can be said that expansion joints will continue to be used in ordinary bridges for a while. Therefore, researches on low noise low vibration expansion joints shall imperatively be performed to improve ride ability and prevent environmental grievance.

As illustrated in Figure 1 for the sensors were installed in the 5th rail when the rail type joint comprises 8 middle-rails. The microphones for the measurement of noise were disposed at a height of 1.5m above the ground. Measurements for each of noise and vibration were performed 3 times per day, in the afternoon (14:00–15:00), at closing hour (18:00–19:00) and on night (21:00–22:00). For each period, measurements were done 4 times during 10 minutes.

As shown in Figure 3, the ranges of the acceleration for both finger type and rail type expansion joints stretch from about 0.13 g to 1.0 g. For the finger type, the distribution of the cumulative frequency reduces as the acceleration increases with reference to 0.138 g, while the distribution reduces at both sides of 0.594 g for the rail type. This reveals that even in the maximum acceleration of finger type seems to be larger than rail type (Figure 2), the value of the acceleration corresponding to the maximum cumulative frequency, which has the largest effect on the durability of the expansion joint, is larger for the rail type (Figure 3). It means that the latter has adverse effect on the durability.

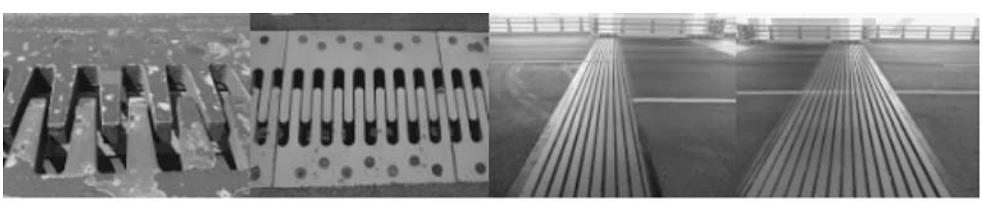
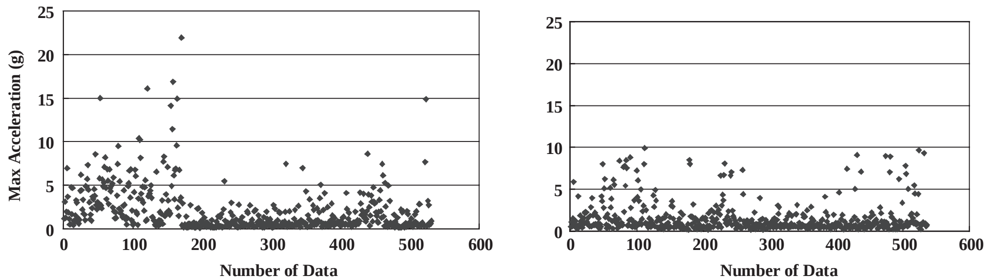
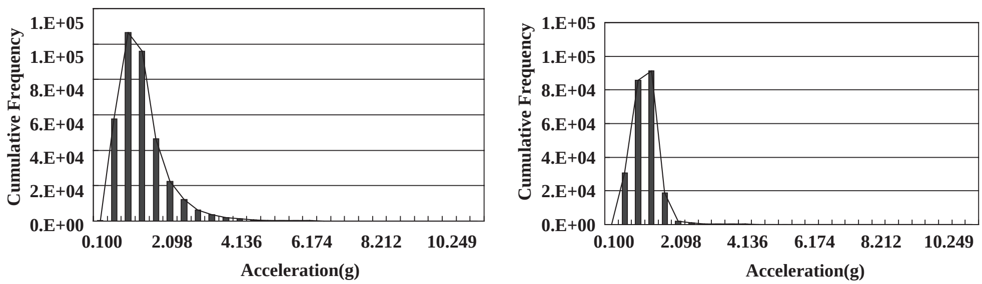


Figure 1. Expansion joints considered for field test measurements.



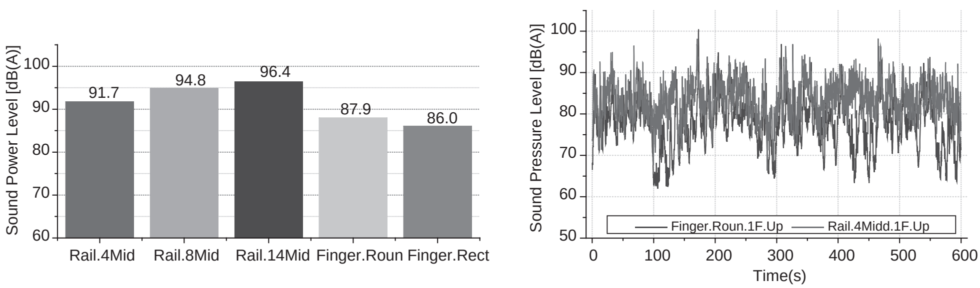
(a) Finger type joint (b) Rail type joint

Figure 2. Distribution of maximum accelerations by type of expansion joint.



(a) Finger type joint (b) Rail type joint

Figure 3. Cumulative frequency of acceleration using rainflow cycle counting.



(a) Maximum sound pressure level (b) Comparison of sound pressure level

Figure 4. Maximum sound pressure level & Comparison of sound pressure level.

The maximum sound pressure levels observed during this this 1 minute are shown in Figure 4(a). Figure 4(b) plots the sound pressure level measured in rail type joint with 4 middle-rails and rounded finger type joint measured during 10 minutes.

Field tests results revealed that the maximum acceleration was about 2.2 times larger for finger type joint than rail type joint. The corresponding maximum mean acceleration was 2.13 g for finger type joint that is larger by about 1.4 times of 1.58 g observed for rail type joint. And, results also verified that most of the accelerations were distributed below 3 g in view of the distribution of the maximum acceleration.

During the passage of lightweight vehicles, which occupy a large portion of the wheel loads applied on the joints, both finger type and rail type joints exhibited similar trends, while occurrence of large difference could be observed during the crossing of heavy vehicles. Following, even if provisions for the vibration control and durability improvement of expansion joints are critical features, the limitation of heavy vehicles also continues a necessity.

As could be observed through the analysis of the cumulative frequency using rain flow counting the range of the acceleration occurring in finger type and rail type joints extended between 0.13 g and 1.0 g. The cumulative frequency appeared to larger by about 1.5 times for the finger type joint compared to the rail type joint. Explanation can be found in the structural type of the expansion joints. Further researches shall be implemented to control the vibrations developed in the free vibration section of the finger type joints.

The analysis of the maximum sound pressure level of the expansion joints revealed that the sound pressure level produced by rail type joints was larger by 2dB(A) to 7dB(A) compared to finger type joints.

Business intelligence and asset management

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Development of the inspection support system for bridge asset management

Takeshi Kigure
Civil Engineering Design Division, Kajima Corporation, Tokyo, Japan

Toru Ishizawa
Asset Management Team, Road Management Division, Aomori Prefecture, Japan

Yoshihiro Hosoi
Design Division, Head Office Service Department, New Structural Engineering, Ltd, Tokyo, Japan

Hideki Fujii, Minoru Iwai & Makoto Kaneuji
Civil Engineering Management Division, Kajima Corporation, Tokyo, Japan

ABSTRACT: The purpose of this paper are to develop an inspection support system for bridge asset management and to report the results of the application for bridges under the jurisdiction of a self-governing body. In most cases, conventional inspection systems are independent systems specialized only for inspection, accordingly such systems usually do not collect condition states data etc. necessary for forecasting deterioration and calculating the life cycle cost (LCC). In this paper, the functions for recording the inspection results, the kinds of data, and system configurations, are introduced regarding the inspection support system for the purpose of acquiring the data necessary for bridge asset management. Furthermore, an example is presented for building a data set as the core for asset management by collecting inspection data on the condition states of each bridge member, applying this system to 742 bridges in Aomori prefecture.

1 GENERAL DESCRIPTION OF BRIDGE INSPECTION SUPPORT SYSTEM

Since inspections are the basis for all management, as well as the basis for all decision-making, it, therefore, is very important. This is a system that has fully utilized IT both for collecting massive and necessary information and for reducing costs. Among the various inspection jobs, inspection data input and reporting are the most troublesome tasks that require labor. This system has realized labor saving and improved precision at the same time by utilizing IT, such as the use of a tablet PC or a personal digital assistant (hereinafter "PDA") for inspection data input or recording the state of changes with digital images. Bridge Inspection Support System is shown in Fig. 1.

This system consists of an expert inspection system that is equipped with a higher level inspection technology for pointing out specific members or portions to be spotlighted or for providing instructions for inspecting other members or portions to identify the causes of the change.

2 RECORD OF INSPECTION DATA

– Estimation of Deterioration Mechanism

At inspection, an estimate will be made of the mechanism of deterioration, for future predictions, or judgment of measures classification for the deterioration or damage found. The inspection support system will, if no change is observed, automatically select the estimated deterioration mechanism according to the environmental conditions and conditions of use by choosing the latent period, which will be explained later. For selection of the deterioration

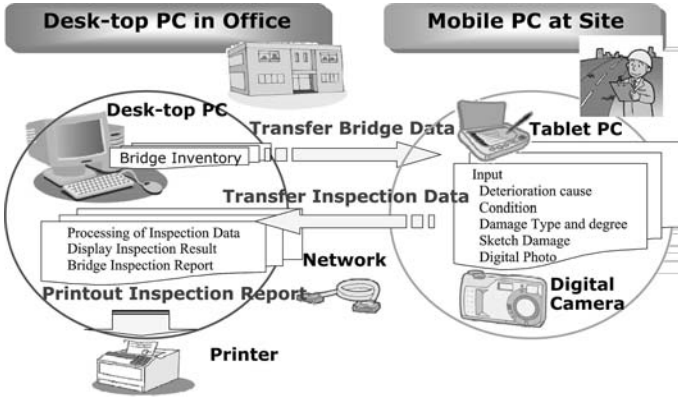


Figure 1. Bridge Inspection Support System.

mechanism, the inspection support system will display only a specific deterioration mechanism according to the data prepared beforehand, with member and material information.

– Evaluation of condition states

  Evaluation of condition states for each target element will be made using a five grade system, which represents five periods such as latent period, progressing period, accelerating former half period, accelerating latter half period, and deteriorated period. Each period indicates the progress in the process of deterioration in consideration of the types and conditions of progress in the deterioration or damage.

– Record of Deterioration and Damage

  For selecting the deterioration or damage mode, this system will display the specific type of damage, locations of damage, conditions, and its spread, which reduces the frequency of beginner's mistakes, and will support an inspector performing the operating job.

– Selection of measures Classifications

  When making the selection of measures Classifications, possible classifications of concern are automatically displayed, based on records of deterioration or damage for each element explained above. (Generated portion, conditions, spread) For example, classifications are automatically displayed by the set of conditions below.

– Function to check missing input for the inspection results
– Function of simultaneous recording for a number of elements
– Copy Function
– Inspection Data Correcting Function
– Making the Bridge Inspection Report

For making the bridge inspection report, it is possible to automatically generate the prescribed form in reference to the inspection results registered in the database (DB).

3 CONCLUSIONS

– A system that collects inspection data necessary for Bridge Asset Management precisely and quickly was developed.
– An example of applying it to actual inspections showed improved efficiency for inspections, quick data collection of results, and the ability to transfer the data for development of the long and medium-term budget plan and medium-term action plan.
– Not only data for estimation of LCC for bridges, but also data necessary for selecting the countermeasure classification have been able to be obtained by acquiring detailed information on deterioration or damage.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

An approach to integrating bridge and other asset management analyses

F. Harrison
Spy Pond Partners, Arlington, MA, USA

D. Gurenich & W. Robert
Cambridge Systematics, Inc., Cambridge, MA, USA

ABSTRACT: Bridge management systems have proved to be of great value for developing preservation strategies, understanding life-cycle costs, and analyzing the benefits and costs at the alternatives at the project and program levels. However, a critical issue in using bridge management systems to address broader asset management needs is that an agency must integrate results generated from bridge management systems with results from other asset management systems in making tradeoff decisions. National Cooperative Highway Research Program (NCHRP) Project 20-57 was undertaken to provide new analytical tools to support asset management. The project's research objectives emphasize the need for tools that help agencies to make the difficult tradeoff decisions for resource allocation while considering both asset preservation concerns as well as the broader set of policy objectives (e.g., safety and economic development) that must be taken into account when making investments in transportation assets.

This paper describes a new analytical tool developed as part of the project, AssetManager NT. The system works with 10- to 20-year simulation results from existing asset management systems and allows users to explore the consequences of different levels of investment within and across asset classes.

AssetManager NT is analogous to a data warehouse for simulation results. It provides an interpolation engine, together with a set of business intelligence data exploration capabilities that operate on these simulation results so that program managers can quickly and easily understand the relationship between expenditure levels and performance across multiple dimensions. Dimensions in AssetManager NT are: time (up to 20 single or multi-year periods), asset classes (e.g. pavement and bridges), geographic areas (e.g. districts or regions), and network subsets (e.g. based on functional class or ownership). For example, the tool can be used to understand how bridge and pavement condition on trunkline routes in the urbanized portion of a state might be expected to change over the next 10 years given different allocations of the budget across these two asset classes.

Some currently available infrastructure management systems provide built-in capabilities to explore simulation results, and off-the-shelf commercial asset management solutions are available that provide integrated analysis capabilities for multiple asset classes. AssetManager NT addresses the needs of agencies wishing to stay with existing tool sets and/or leave the door open for future implementation of best-of-breed tools that have an appropriate level of sophistication for specific asset classes. AssetManager NT does not attempt to implement a consistent approach to deterioration modeling, treatment selection or optimization across asset classes. Rather, the tool serves a single function – investment vs. performance what-if analysis. This is in keeping with a modular, service-oriented approach to software architecture. It also recognizes that there are typically distinct user communities for asset management tools – one concerned in more detailed technical aspects of treatment selection and project prioritization; the other concerned with higher-level resource allocation tradeoffs.

Figure 1 presents the key organizing concepts of AssetManager NT. Using AssetManager NT involves a three step process: running simulations in external tools and creating input files, using

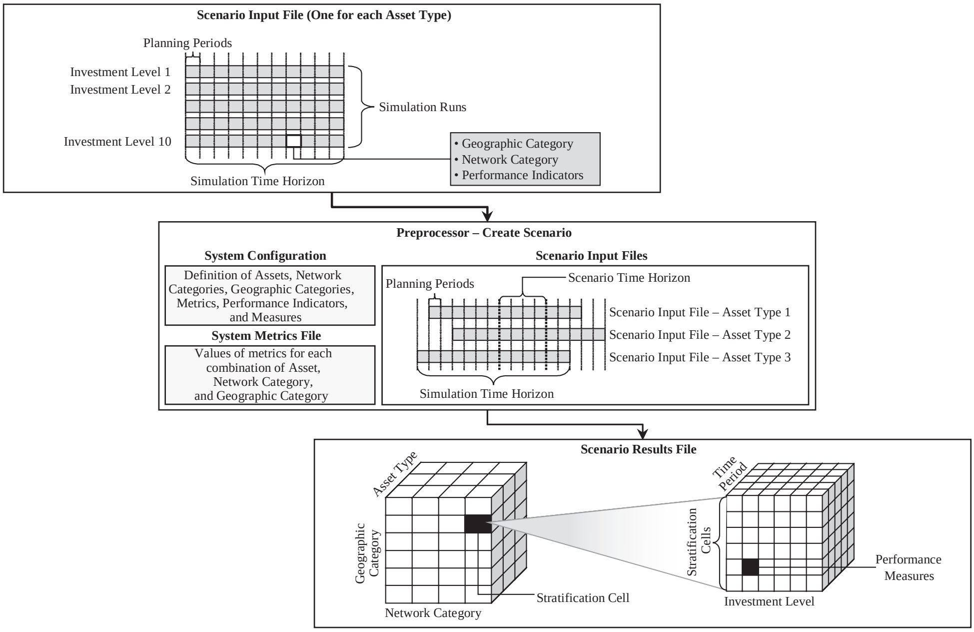


Figure 1. AssetManager NT Organizing Concepts.

the AssetManager NT preprocessor to create a scenario file, and finally, conducting what-if analysis for a given AssetManager NT scenario. Companion “robot” tools also were developed to produce inputs needed by AssetManager NT from systems such as the American Association of State Highway and Transportation Officials (AASHTO) Pontis Bridge Management System.

AssetManager NT was tested at New York and Montana state DOTs. After reviewing the system, staff at both agencies observed that AssetManager NT was a useful tool that could be used both in the context of periodic long-range infrastructure needs analysis (for long range plan updates), as well as for the annual program development cycle. Both states have embraced performance-based planning and programming, and were therefore very receptive to tools that could support their established business processes for making budget decisions based on performance tradeoffs. Both of these agencies were already analyzing infrastructure investment tradeoffs, but AssetManager NT makes this process easier.

The testing process highlighted some limitations of AssetManager NT. First, for some types of assets – notably bridges, expenditures tend to be “lumpy”, i.e. there are a relatively small number of high cost treatments that must be done on an all-or-nothing basis. In this context, performance predictions obtained from piecewise linear interpolation may not be realistic in certain circumstances. A second limitation is that the tool works only with aggregated performance results, and users can not drill down to specific projects or assets that underlie these results.

A new project being launched by the American Association of State Highway and Transportation Officials (AASHTO) AASHTOW are program will implement the current version of AssetManager NT in ten state DOTs and then develop a plan for future enhancement of the tool. Enhancements will be determined based on input from the participating agencies. Some of these enhancements may address the limitations discussed above that were identified in the initial testing process. It is likely that changes to the tool’s software architecture and platform will be made as well – the tool is currently a stand-alone desktop application.

The AASHTOW are project will also encompass a second tool developed in NCHRP Project 20-57 – AssetManager PT, which works with a list of proposed projects and addresses shorter range tradeoff questions. Integration of the NT and PT tools will be explored as one way to address the desire for additional drill-down capabilities.

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The next generation of the Pontis Bridge management system

W. Robert, A. Marshall & S. Hwang
Cambridge Systematics, Inc., Cambridge, MA, USA

J. Aldayuz
American Association of State Highway and Transportation Officials, Washington, DC, USA

ABSTRACT: The American Association of State Highway and Transportation Officials (AASHTO) Pontis® Bridge Management System (BMS) is designed to assist agencies in the preservation and improvement of their bridge networks. Pontis has been in use in agencies across the U.S. and abroad for over ten years. The current version, Pontis 4.4, is a client/server system developed in PowerBuilder and C++.

In 2003 the AASHTO BRIDGEWare Task Force determined that the Pontis product had reached a crossroads. The design of the Pontis product had followed a classic client/server architectural organization, with significant processing deployed to the end user workstations (sometimes called "fat" clients). At that time it was originally implemented, the architecture was state-of-the-practice for database management applications. To date it continues to serve effectively as the architecture for the Pontis product. While stability is without question a desirable characteristic for software, the fact is that the software world has changed dramatically during the history of Pontis. As of 2003, few elements of object-oriented design, web technology, multi-tiered organization, or distributed processing had, but they had become increasingly important to various audiences in the user communities and to the viability of the product generally.

In the interest of ensuring that Pontis would continue to serve the needs of its license base, in 2004 AASHTO and the BRIDGEWare Task Force embarked upon an effort to update the technical architecture of the Pontis product, while adding desirable new functionality to the product and retaining the elements of the system which had proven valuable to the existing user. This effort focused initially on defining the requirements and implementation plan for a new generation of Pontis, Pontis Release 5, to be developed in a web-based environment using Microsoft .NET technology. The system is now under development and scheduled for release in 2006.

Pontis 5 will utilize a multitier application architecture, based on the .NET technology framework. It will reside on a Microsoft IIS web server. The high-level functional requirements for Pontis 5 were developed through consultation with a Technical Advisory Group (TAG) including Pontis users, agency IT professionals, contractor representatives, and BRIDGEWare Task Force participants. The requirements were based on a review of the Pontis 4 capabilities, a list of outstanding enhancement and improvement requests, and the results of a stakeholder analysis. Besides the fundamental shift to a web-based environment, other new functionality planned for the system includes:

- Enhanced bridge desktop featuring improved display of multimedia documents and summary bridge data;
- Support for additional asset types, such as sign structures, noise walls and other structures;
- Import and export of data in XML based on the TransXML schema being developed through National Cooperative Highway Research Program (NCHRP) Project 20-64, XML Schemas for Exchange of Transportation Data;
- Improved approaches to modeling bridge needs through incorporation of research results from NCHRP 12-67, Multiple-Objective Optimization for Bridge Management Systems;

– Improved user interfaces for bridge-level analysis and project planning; and
– Seamless support to its licensees for any potential changes in the U.S. National Bridge Inventory (NBI) coding standards.

Pontis 5 development will be phased over a span of four years, delivering functionality in specific areas in three main releases termed 5.0, 5.1, and 5.2. The first release will focus on creating a core application framework which will form the basis for future development utilizing Microsoft .NET Framework 2.0 technology. Pontis 5.1 will build on the 5.0 product, adding a new Inspection Module with full range of inspection data collection capabilities. Pontis 5.1 will permit data entry, XML data exchange, enhanced data validation, archiving capability, and map display. In addition, the database will be redesigned to support any changes to the NBI data specification. The design process for Pontis 5.1 release begins in Spring 2006 with delivery to end users in 2007.

Pontis 5.2 will continue to extend the product with the goal of replacing Pontis 4.4 altogether. This version will be completed in 2008–2009. Pontis 5.2 will offer support for Pontis modules not migrated in Pontis 5.0 and 5.1, including Preservation, Programming, and Project Planning. This version will leverage ongoing NCHRP research to add improved bridge modeling and analysis tools.

Pontis 5 is expected to have a positive impact for the agencies using the system. Though transition to the new architecture represents a significant challenge, the change is expected to have positive impacts including streamlined deployment, an up-to-date technical platform, expanded data management capabilities, support for server side processing, and improved functionality for supporting customization.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

The role of the bridge management system in bridge asset valuation

Reed M. Ellis
Stantec Consulting Ltd.

Paul D. Thompson
Consultant

Today, most if not all transportation agencies or Departments of Transportation (DOT) are moving towards implementing asset management systems or components of asset management systems.

Asset management systems for bridges generally answer fundamental questions about ownership, location, and condition, and most also address the cost of preservation and improvement needs, as well as the forecasting of future performance. But so far there are few tools to estimate the economic value of bridge assets and the effect of agency policy on this valuation. This question has become especially important to many agencies in response to GASB 34, and it is also a useful performance measure for management decision support.

Often without the ability to answer the question of "what is the value of our assets?" engineers and/or administrators are unable to adequately support an argument for increased funding of infrastructure (particularly when borrowing to finance improvements). In essence, the civil engineering community is facing a new era where valuation of infrastructure assets is being required, but the procedural guidelines/models needed are yet to be developed or adequately defined.

Fortunately, most modern BMS such as the Stantec Ontario Bridge Management System (OBMS) have all the necessary data and models to perform the asset valuation calculations for individual bridges, subsets of bridges, or for the entire network. In the particular case of OBMS, a new performance measure called the Bridge Condition Index (BCI), which is calculated based on bridge condition and financial weight or importance of contributing elements of the bridge, can be directly tied to the current asset value of the structure.

In this paper, a case is made that as the demands on agencies and those responsible for managing bridges increases, then there should be higher expectations on requirements for bridge management systems (BMS). Further, bridge management systems can be a very useful tool in the asset valuation of bridges. Since the majority of data necessary to perform asset valuation calculations are contained in most BMS, then it only follows that the system should perform the valuation calculations as well. The fact that the bridge condition index used in the condition inspection and as a performance measure can also be used in the bridge valuation makes the argument even stronger. Methods to determine the asset valuation of bridge assets are discussed, and means to carry out these calculations are shown which can be implemented in a bridge management system.

Different GASB 34 compliant methods are shown which can be used depending on the agency preference. An example of how this has been carried out in the Ontario Bridge Management System is provided.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Multi-objective optimization for bridge management

P.D. Thompson
Consultant, Castle Rock, CO, USA

V. Patidar, S. Labi & K. Sinha
Purdue University

W.A. Hyman
Applied Research Associates, Inc.

A. Shirolé
Consultant

ABSTRACT: National Cooperative Highway Research Program Project 12-67 has recently completed the development of a multi-objective optimization framework and decision support tool for bridge management. Designed to work with Pontis, a bridge management system offered by the American Association of State Highway and Transportation Officials, the software is also adaptable to other bridge management systems containing suitable data

The new framework is distinctive in its ability to optimize bridge level and network level programs having multiple objectives and multiple constraints beyond life cycle cost. A utility function combines performance measures into a single objective function operating at both levels.

Performance constraints may be applied at the bridge level to govern intervention timing, and/or at the network level to set performance goals. The program manager can readily move performance measures between the objective function and the constraint set to explore the feasible solution space.

Optimization has become a common feature of Bridge Management Systems (BMS), but one criticism of the technique as implemented so far is that it doesn't reflect the full range of benefits of bridge maintenance and of a high standard for bridge condition and functionality. This project was designed to improve this situation by developing a new optimization methodology that accounts for multiple objectives and constraints. This methodology was implemented in a framework and decision support tool that are due to be completed in spring of 2006, to be published in an upcoming NCHRP report.

The framework uses state-of-the-art life cycle cost analysis similar to what is already practiced in BMS, but adds several additional condition and performance measures that participate in the model through a utility function in the objective function, and in the performance constraints at both the bridge and network levels. Among the 28 performance measures considered in the system are new indices of structure vulnerability to scour, fatigue, earthquake, and other hazards; and three custom measures that can be defined by the agency.

Important for usability of the research results are two highly graphical software modules that help the decision maker to visualize the solution space at the bridge and network levels.

The bridge level tool features a graphical presentation that shows the performance tradeoffs that occur with changes in the scoping and timing of an intervention. The maintenance planner uses a digital dashboard to adjust the definition of a candidate until the performance outcome is what is wanted. For each candidate life cycle activity profile, the dashboard forecasts future performance, including future interventions that may be required.

At the network level, the program manager can freely move performance measures between the objective function and constraints, to see how the achievement of one target may affect other

targets under budget constraints. This allows the balancing of several different program performance concerns to develop an appropriate utility function and realistic expectations of program outcomes.

Bridge management has evolved to the point where agencies want the ability to analyze more realistic policies than have been possible so far, especially policies that maintain a set of welldefined performance standards represented as constraints on a multi-objective optimization framework.

NCHRP Project 12-67 has shown that it is feasible to analyze such policies by development of suitable algorithms, using supporting models (such as deterioration, agency cost, and user cost) and data that already exist. To make such models implementable in a transportation agency, a highly graphical digital dashboard presentation can make the solution space easy to understand, helping bridge maintenance planners and program managers to make more confident decisions.

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Probabilistic model for aging of bridges

M. Petschacher
Petschacher Consulting, Feldkirchen, Austria

ABSTRACT: The maintenance of structural equipment along the Austrian road network is supported by a database system named BAUT. The aim during development of BAUT was to provide a very flexible tool to map the various types of structures like bridges, tunnels, noise barriers, retain walls and so on into the digital space. With the help of this database system the maintainer should retrieve a comprehensive overview of administrative and technical parameter for decision making.

BAUT contains a description of inventory and time dependent data mainly of those structures, which are critical from a safety point of view. The tool can be applied from either on project level, i.e. retrieving detail data, pictures, inspection reports, as well as on management level by means of summary tables and reports or even OLAP methods.

BAUT is owned by ASFINAG and holds data nearly of all constructions on the Austrian road network. The aim of the paper is to give a brief overview of the possibilities, the easiness of use and some implementation details. The strength of BAUT is a compromise between amount of information stored in the database and completeness of data in it. Concrete a probabilistic approach for prediction of degradation and necessary investment of structures is presented. On basis of available basic data and stored inspection results from the past 6 years valuable results can be presented.

Life Cycle Cost Analysis (LCCA) should be considered in all phases of construction, maintenance and operation. The analysis period of LCCA should be long enough to capture long-term differences in discounting life cycle costs among competing alternatives and rehabilitation strategies. Future costs should be discounted to the net present value (NPV). It is important to employ LCCA to maximize the return of investment.

In order to optimize the reliability of the bridge stock and need to minimize the maintenance costs, it is necessary to develop deterioration models. With such models it is possible to predict future situations within the system. Knowing the deterioration tendency of the bridges rehabilitation and repair measures can be better planned to guarantee a high quality of the road network. Further, prediction of future expenditures can be improved.

On base of the probabilistic code VaP 2.2 the Cohort Survival Simulation was implemented. It is an add-on to BAUT. The key idea is adopted the approach originally used in population science to infrastructure, particularly to bridges in this paper. The life cycle process can be thought similar to that of humans. When a bridge is built and taken into operation the process of ageing and deterioration starts. In case of a failure of a bridge this is equivalent to the death in the human cohort model. Deaths in the human population are replenished by the natural reproduction of its members. In the bridge cohort model objects are replaced by new constructions, or they are repaired or strengthened. These actions need monetary input. As with social planning that occurs with human population prediction, the road administration needs a tool for planning the monetary investment amount in future.

The paper presents results for a typical bridge population in Austria. The actual forecast is done for 60 years with a individual discounting rates. The average costs are taken from projects during the past 5 years. Two types of maintenance actions are distinguished, repair and replacement. Due to the growing net also newly built bridges are considered in the simulation but they contribute not very much. In principal the ageing is described by a hazard function which

describes the transition of a structure from one condition level to the next worse. The survival functions are estimated from available data whereby two basic distribution types are offered, i.e. Weibull and Herz. The parameters and strategies are easily changed during runtime to figure out sensitivities.

Load and resistance assessment of railway bridges

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Structural assessment of concrete railway bridges: Non-linear analysis and remaining fatigue life

M. Plos, K. Gylltoft & K. Lundgren
Chalmers University of Technology, Göteborg, Sweden

L. Elfgren
Luleå University of Technology, Luleå, Sweden

J. Cervenka
Cervenka Consulting, Praha, Czech Republic

E. Brühwiler
Ecole Polytechnique Federal de Lausanne (EPFL), Lausanne, Switzerland

S. Thelandersson
Lund University of Technology, Lund, Sweden

E. Rosell
Swedish Road Administration, Borlänge, Sweden

ABSTRACT: For a sustainable development in Europe, there is a need to at least double the railway transports in the coming 20 years. In order to reach this, the residual service lives of existing concrete bridges need to be extended, at the same time as they are subjected to higher axle loads, higher railway speeds and heavier traffic intensity. Today, many concrete bridges are replaced or strengthened because their reliability cannot be guaranteed based on the structural assessments made. The aim of the work presented here is to provide enhanced assessment methods that are able to prove higher load carrying capacities and longer fatigue lives for existing concrete railway bridges.

One main objective is to facilitate the use of non-linear analysis for structural assessment. In addition to higher load carrying capacities, the methods give improved understanding of the structural response, forming a better basis for decisions in the assessment. Another main objective is to improve knowledge about the fatigue behaviour of concrete bridges and to develop realistic methods for the evaluation of remaining fatigue life of existing bridges. The emphasis here is on short-span bridges and secondary elements.

The work presented is a part of the ongoing EU-project *Sustainable Bridges*. The results will be implemented in the *Guideline for Load and Resistance Assessment of existing European Railway Bridges* that is being developed.

1 ASSESSMENT OF CONCRETE BRIDGES BY NON-LINEAR ANALYSIS

Non-linear analysis is the method for improved assessment of existing structures that reflect the structural behaviour in the most realistic way. It removes the inconsistency included in standard design approaches where the check of cross-sections is done using non-linear material assumptions while the cross-sectional forces are determined based on purely linear analysis. However, it puts higher demand on the engineer and it may require considerable computational resources, compared to traditional methods. For practical applications, the finite element method (FEM) is commonly used for general non-linear analyses.

A non-linear analysis can include geometric non-linearity (such as buckling) as well as material non-linearity (such as reinforcement yielding and concrete cracking), giving the complete response of a bridge under a given imposed loading. Nowadays, non-linear FE analysis represents a powerful tool for the estimation of remaining load-carrying capacity of existing structures. The work includes a state-of-the-art review of models for assessment of concrete bridges, together with examples and general recommendations for practical applications.

2 EVALUATION OF MATERIAL PROPERTIES

The purpose of assessing the material properties for an existing bridge is to obtain the best possible information about the relevant resistance parameters for that specific structure. It is also important to describe the uncertainties associated with each parameter. Important bases for evaluation are the material specifications from the original construction as well as testing of the current in-situ properties for the materials in the existing bridge.

The work includes a state-of-the-art review of the determination of in-situ material properties in old concrete structures. The materials included are concrete, reinforcement and prestressing steel. The objective is to improve the possibility to make a reliable determination of in-situ material properties. A particular aim is to develop better methods to describe the development of concrete properties with age and degree of hydration.

3 THE EFFECT OF CORROSION ON REINFORCEMENT BOND

The volume increase that takes place when reinforcement in concrete corrodes causes splitting stresses in the concrete. Thereby, the bond between the reinforcement and the concrete is influenced. In this work, the effect of corrosion on the bond was investigated and described in a systematic way. Literature studies of experimental work were combined with non-linear finite element analyses to develop an overview of the corrosion effect on the bond properties and to form recommendations for assessment of corroded railway concrete bridges. The aim for the future work is to give an overview also regarding fatigue and to further develop general recommendations for practical assessment of existing bridges.

4 SIMPLIFIED FORCE AND MOMENT REDISTRIBUTION IN LINEAR FE ANALYSIS

For complicated bridge geometries, three-dimensional FE structural analysis can rationalise and improve the bridge assessment. For bridges, linear analysis is normally used in practice, often leading to high stress concentrations, e.g. at concentrated supports. However, in analyses of concrete bridges, these stress concentrations do often not exists in reality. This is due to the cracking of the concrete and the yielding of the reinforcement, leading to stress redistributions.

The objective of this work is to develop general recommendations for how such calculation method dependent concentrations, obtained by linear FE analysis, can be smoothened for assessment of concrete bridges. It is important that the smoothing is made so that it resembles the actual response of the existing bridge. The results show the importance of keeping separate the concentrations emanating from simplifications in the geometrical modeling and those coming from the assumption of linear material response.

5 BENDING-SHEAR-TORSION INTERACTION

A good understanding of the load-carrying mechanisms in combined bending-shear-torsion gives a possibility to make more accurate assessments than with standard code methods. In this work, such methods and examples from bridge evaluations are presented. The modified compression field theory is used in the study of several Swedish bridges to follow the successive increase of stresses in critical sections.

6 REMAINING FATIGUE LIFE OF REINFORCED CONCRETE BRIDGES

A fatigue safety assessment of a railway bridge is made to determine if the fatigue effects of future traffic loads will impair the safety of the structure during its intended service life. For railway bridges, all structural elements are generally checked, and in particular those subjected directly to wheel loads. Normally, the fatigue safety check can be concentrated on the steel reinforcement, based on existing knowledge from steel structures. Fatigue failure of concrete is very unlikely if the concrete is in good condition.

A rational methodology in three steps is proposed for fatigue safety assessment of existing reinforced concrete railway bridges:

1. Study of the bridge structure and evaluation of reinforcement detailing
2. Inspection of the existing bridge and study of the past performance
3. Fatigue safety check.

In the future work, the methodology will be validated and examined on existing railway bridges for determination of remaining service life.

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Considerations for traffic loads in the assessment of existing railway bridges

G.A. James & R. Karoumi
Royal Institute of Technology, Stockholm, Sweden

1 INTRODUCTION

The dimensioning philosophy adopted when writing new bridge codes and consequently when designing new structures, is conservative in nature. The traffic load models of these codes need to cover all European conditions and cater for future heavier traffic. Despite this, these traffic load models are often used to evaluate existing infrastructure where the actual traffic conditions can be vastly different from those assumed by the code writers. This conservative philosophy should be abandoned when assessing existing bridges as replacement cost or strengthening cost may be extremely expensive.

Prior to recent developments in the rail freight market, the locomotive was always the heaviest load on a bridge and this load was clearly defined from manufacturing specifications. The uncertainty in the traffic loads on bridges arose primarily from the dynamic effects and from possible future changes in locomotive design. Nowadays, with the constant demands for increased allowable axle loads on freight wagons, the loaded wagons themselves are beginning to constitute the heaviest traffic loading on a bridge. Since the company wishing to transport the freight is often responsible for loading, the level of this loading is associated with higher degrees of uncertainty and needs therefore to be statistically analyzed. In contrast, the static weight of high-speed passenger trains are clearly defined, however, their dynamic effects may cause serious problems both for the bridges and by increasing the risk of derailment through ballast and track instability as a result of excessive bridge deck accelerations.

Two of the main aims of the European sixth framework program entitled "Sustainable Bridges", are to increase the transport capacity of existing railway bridges by increasing allowable axle loads to 33 metric tons for freight trains with moderate speeds and to increase allowable speeds for passenger trains with low axle loads to 350 km/h. In order to achieve these goals it was judged important to improve knowledge on the actual traffic loads and their associated dynamic factor. At the start of the program it was decided to do a survey of some present European national codes for assessing existing railway bridges, paying particular attention to the traffic load models and associated dynamic amplification factors. The work from this survey is presented in this paper. The national codes covered in the survey and consequently presented in this paper are the German, Swiss, French, Danish, Swedish and British codes.

2 TRAFFIC LOADS FROM EUROPEAN ASSESSMENT CODES

The majority of the European codes use as a starting point for assessment the traffic load model LM71 of Eurocode 1 or a derivative thereof. The use of the factor α defined in Eurocode 1 is often adopted for an application of this traffic load model on a national or specific line basis. For example Sweden and Denmark use an α-value of 1.33 for their heaviest load model. The value of the factor α is thereafter reduced to lower bridge classification levels.

The French and Swiss, and even the Spanish methods of assessment appear to closely follow the European codes for new bridges with no real distinction for existing bridges. However, in the French assessment code special attention is made to the dynamic effects of high speed trains. The British code is by far the most extensive and provides a comprehensive set of rules and guidelines for the assessment of existing railway bridges. The main content of the work treats the material strengths and properties as opposed to the loading. However, only the traffic load model and associated dynamic effects are shown in this paper. The British traffic load model for assessment purposes is not based on the LM71 traffic load model. The German code appears to work from the basis of the Eurocode 1 but makes allowances for special circumstances associated with assessment of existing bridges. This is especially noticeable by the use of reduced partial safety factors when extra information is available which supports this reduction; a technique which is employed by many of the studied assessment codes. Apart from the LM71 traffic load model, both the Swedish and Danish assessment codes allow for the assessment of bridges based on realistic wagons. Both countries have adopted the wagons from the line classification system used for the wagon marking system of the RIV agreement and then included extra wagons of their own which suit their countries specific traffic needs. The paper gives examples of these realistic wagons and discusses their use.

The paper also discusses the dynamic factors used with the different load models and the different limit states. The paper identifies a need to investigate the dynamics of secondary and transverse members.

2.1 *Traffic load models for fatigue*

One problem when assessing an existing bridge for fatigue is to establish a load history for the bridge. The Swedish and Danish Rail Authorities have good records of the total tonnage passed on different lines which can either be in terms of gross or net tonnage. This information, together with a simplified traffic model is used to assess the likely stress range and number of stress cycles for varying periods throughout the last century. This information can then be used in a conventional fatigue analysis. The traffic load models used by the Swedish and Danish Authorities are described in this paper.

3 CONCLUSIONS

The survey of several European codes for the assessment of existing railway bridges shows that many countries use the traffic load models of Eurocode 1 for the assessment of existing bridges even though these traffic load models are intended for the design of new structures. Several countries have adopted their own policies for assessment purposes and it is clear that a common guideline for European railways would be advantageous. The dynamic amplification factors adopted by most countries are based in some manner on the dynamic amplification factors of Eurocode 1. However, even here national interpretations have been made. The dynamic amplification factor relevant for the assessment of cross girders and secondary members of existing structures is a topic for further research as the dynamic factors of Eurocode 1 were developed from research on main girders. Only the British code makes a special distinction for the dynamic effects of this type of member. The pragmatic approach of the Danish, Swedish and British Railway Authorities on how to handle the topic of an historical fatigue model appears to be a useful method, provided the railway authority in question has reliable records of the gross or net tonnage throughout the history of the bridge under assessment. The Swedish and Danish use realistic wagons for assessment purposes, the uncertainties associated with overloading, uneven loading, model uncertainties etc are presumably incorporated into the partial safety factor for actions. However, verification of the extent of overloading of freight wagons need to be addressed, preferably by some form of weigh-in-motion measurements as the authors are unaware of any such published study for European traffic conditions. The subjects of dynamic amplification factors for secondary members and obtaining reliable weigh-in-motion data for statistical analysis of actual axle loads of freight trains are both topics for further research for the load group working within the Sustainable Bridges Programme.

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A new assessment method for masonry arch bridges

C. Melbourne & A.K. Tomor
University of Salford, Salford, UK

ABSTRACT: Masonry arch bridges make up approximately 40% of the bridge stock of Europe. As these structures deteriorate with age and are subjected to ever increasing demands (not only in terms of the volume of the traffic, but also in terms of increasing axle loading); assessing engineers are faced with ever increasing pressures to improve their assessment methods to better reflect the 'true' carrying capacity and determine residual life.

1 MODES OF BEHAVIOUR

Masonry arch bridges are primarily particulate gravity structures that mainly depend upon their geometry to determine their behaviour and carrying capacity. It is usually assumed for a single square spanned bridge that the failure mechanism involves the formation of 4 hinges. This is entirely consistent with the structural idealisation of the barrel as an elastic fix-ended arch continuum or as an assemblage of rigid blocks. Unfortunately, this only represents one of many mechanisms that should be considered. For example, if the arch formed 3 hinges and the abutment was able to move horizontally, this would represent 4 'releases' thus creating a mechanism. An overview of the interaction between the constituent elements of the bridge is required to ensure that the critical mechanism is identified.

There are many other ways in which the bridge might fail. These include: crushing of the masonry; ring separation in multi-ring arches that is brought about by the failure of the mortar between the rings; punching shear failure that is a result of failure of the radial mortar joints; foundation failure that might result from the effects of scour; failure of the backfill and many more. Any assessment procedure needs to consider each of the possible failure mechanisms separately and in combination.

2 METHODS OF ASSESSMENT

Currently, there are three methods that are used to determine load carrying capacity:

- Semi-empirical Methods: MEXE
- Limit Analysis Methods
- Solid Mechanics Methods

These methods are discussed and their limitations identified.

3 LIMIT STATES

The ultimate limit state (ULS) for masonry arch bridges can be defined as the condition at which a collapse mechanism forms in the structure or its supports.

The serviceability limit state (SLS) for masonry arch bridges can be defined as the condition at which there is a loss of structural integrity which will measurably affect the ability of the bridge to carry its working loads for the expected life of the bridge.

4 NEW ASSESSMENT METHOD (THE 'SMART' METHOD)

The application of the philosophy of fatigue performance to the assessment of masonry arch bridges is in its infancy and much new data are required for its verification. However, it can be seen that the method offers an insight into the long-term performance of masonry arch bridges which has not been available to bridge owners hitherto.

The first step is to determine the geometrical details and all the constructional details of the bridge including the foundations and the nature of the backfill.

Secondly, the loading regime should be determined. Currently, this involves studying the nature of the traffic and incorporating a factor to take account of dynamic effects.

Next the properties of all the constituent materials should be classified, e.g. the masonry could be strong, medium or weak. These should not only be in terms of monotonic loading strengths but, more significantly, also in terms of their performance under fatigue/cyclic loading which should be expressed in terms of a predefined S-N curve (which may be ambient stress level dependent). The predefined S-N curves should include all modes of failure.

There should then be a series of analyses that consider a range of statically determinate (and, where appropriate, statically indeterminate) configurations subjected to factored loads. The loading regime should then consider the distribution and frequency of the traffic, grouping it to allow analysis of the number of stress events that occur in each prescribed stress range. This will allow an assessment of residual life (using for example Miner's Rule). Alternatively, a serviceability limit state for each relevant mode of failure could be determined and a 'safe' SLS established.

Finally, an ultimate limit state analysis should be undertaken to ensure that the bridge can sustain a once in a life-time overload.

5 CONCLUSIONS

The full paper discusses current assessment methods and concludes that although they were found to be able to predict ultimate carrying capacities with some confidence serious concern was identified with respect to predicting long-term behaviour and residual life.

A new assessment procedure has been presented (SMART) which differs philosophically with all the current techniques in as much as it defines a SLS and facilitates a technique by which permissible working loads and residual life can be determined.

A simple example is presented in the full paper to demonstrate the new technique.

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General basis and criteria for the capacity assessment of European railway bridges

J.R. Casas
Technical University of Catalunya, Barcelona, Spain

E. Brühwiler
Swiss Federal Institute of Technology EPFL, Lausanne, Switzerland

J. Cervenka
Cervenka Consulting, Praha, Czech Republic

G. Holm
Swedish Geotechnical Institute, Linkoeping, Sweden

D. Wisniewski
University of Minho, Guimaraes, Portugal

ABSTRACT: The paper deals with a summary description of the main safety criteria adopted in the Guideline for Load and Resistance Assessment of existing European railway bridges developed in the WP4 of the SUSTAINABLE BRIDGES project, funded by the European Commission. A more complete and detailed description of the general basis and criteria can be found in the complete paper and in the background documents (Casas 2006, Casas and Wisniewski 2006, Casas et al 2006, Brüwhiler 2006, Holm 2006).

1 SAFETY CRITERIA

The safety criteria is based on the adoption of a limit states format with different levels of complexity (partial safety factor method or full probabilistic analysis) depending on the importance of the structure and the results of previous assessments. In the first part, the description about the available safety formats and required levels of safety is presented. The description is divided according whether a member (element) or system (whole bridge) assessment is carried out. In the last case, the paper describes how to define a safety format and the corresponding target value of the level of safety to take into account the inherent level of redundancy (longitudinal and/or transversal) present in most of the bridges currently in the bridge network. Also in this case, particular emphasis is put on the reliability-based assessment using non-linear analysis. The available methods are presented and applied to several examples. Due to the high complexity of the complete probabilistic non-linear analysis, two simplified methods are also developed and applied to the examples to show their applicability and accuracy.

The safety level is also considered at a member and system approach. The proposed target reliability levels proposed in different countries and by different international bodies (Eurocode, ISO) are presented, jointly with the most significant assumptions. In this way, the engineer responsible for the assessment can choose the most suitable safety level for each specific case into consideration.

2 FATIGUE ASSESSMENT

A rational procedure for the examination of fatigue safety which proceeds by stages using both deterministic and probabilistic methods of increasing sophistication is proposed. Probabilistic methods enable the explicit consideration of the scatter of the parameters that influence the fatigue strength and the fatigue damaging effect. Inspection methods and intervals are an integral part of a comprehensive fatigue examination, and field testing may be appropriate to calibrate the model for structural analysis by which fatigue relevant stresses are calculated.

The fatigue safety is examined using a procedure in stages with increasing level of sophistication including the following steps: (1) Simplified deterministic method, (2) Simplified probabilistic method, (3) Consideration of monitoring, (4) Detailed probabilistic method.

The aim of the first stage is to identify the fatigue critical members of the structure. The probability of fatigue fracture of a structural detail or element is assessed in stages 2 and 4. The probability of crack detection during inspection and monitoring is evaluated in stage 3, and subsequently linked to the (calculated) probability of fatigue fracture to obtain the probability of failure.

The condition of a railway bridge is monitored by regular inspection at least every 5 years throughout its service life. The monitoring of construction details identified as being fatigue vulnerable makes it possible to increase safety of the bridge. Appropriate monitoring techniques (including detailed inspections) allow for determining the probability of detection of a fatigue damage indicator (usually a crack). Depending on the accessibility of a construction detail to be monitored, the reliability and precision of the monitoring technique applied as well as the interval of inspection or measurement of a specific structural property, values for the probability of detection of an anomaly due to fatigue damaging may be determined and justified for use in the fatigue safety check.

3 GEOTECHNICAL ISSUES

An aspect of interest in the safety assessment of existing bridges is how to take into account the long-term behaviour of the subsoil in transition zones. In fact, for existing railway bridges the assessment of long-term settlement is important. The foundation of the bridge is in most cases stiffer than the embankment why it is likely that differential settlement will occur. Hence, the ability to calculate the increase in settlement for a certain time interval is crucial to judge the need for maintenance or strengthening measures in a long-term perspective. To improve the basis and criteria for decision on actual level of safety, uncertainties in the calculation of settlements have to be addressed. Therefore, the Guideline proposes a probabilistic approach to the calculation of settlement taking into account the uncertainties in the calculation, the information from the existing ground investigations, the load and the accuracy of the measurement.

ACKNOWLEDGMENTS

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Improved assessment methods for static and fatigue resistance of metallic railway bridges in Europe

A. Patrón & C. Cremona
Laboratoire Central des Ponts et Chaussées, France

S. Hoehler
RWTH Institute of steel construction, Aachen, Germany

B. Johansson & T. Larsson
Lulea University of Technology, Sweden

M. Maksymowicz
Universidade do Minho, Portugal

ABSTRACT: A very important part of the bridges in the European railway networks are metallic bridges that have been built during the last 100 years (some of them are much older). The increasing volume of traffic and axle weight of trains means that for many structures the loads today are much higher than those envisaged when they were designed.

This paper deals with the research program established to develop improved assessment methods for existing metallic bridges in the context of the work package 4 (Loads, Capacity and Resistance) of the 6th framework European project *Sustainable Bridges*. Three main research topics under development.

The first topic concerns information about material properties of steel and iron used in old existing railway bridges. Any safety assessment for an old bridge requires the knowledge of the structural resistance and therefore the information of the material properties. For the resistance to static loads the material strength expressed as yield strength (f_y) is the significant parameter. In order to ensure sufficient fatigue resistance, next to the classic fatigue methods using damage accumulation further assessment models have been established that are based on fracture mechanics. For the classic fatigue assessment the fatigue properties are needed. Fracture mechanical approaches, taking into account that crack-like defects are very likely to be in the structure, use the fracture toughness as material resistance. This is usually given by J-Integrals (J_i, J_c) or stress-intensity-factors (K_{Ic}). Also further crack growth parameters, e.g. threshold values for crack growth, are important. The early metal bridges in the 19th century were fabricated of cast iron or puddle iron (wrought iron). Puddle iron surpassed cast iron having a lower carbon content that goes along with a better ductility and it allowed forging and an easier workmanship. Yet at the beginning of the 20th century puddle iron was superseded by mild steels that obtained higher qualities concerning the chemical composition and cleanness as well as better technological material properties (e.g. weldability, strength). For these old metal bridges, that were built between 1870 and 1940 in particular, the material parameters are in many cases not available. One reason is that although steel production and construction technology (bolting and welding of joints) developed quickly, appropriate testing methods to examine relevant properties as toughness, fatigue, etc., were missing completely and were not available until many decades later. Therefore only fragmentary knowledge exists concerning early iron materials, complicating the handling and assessment of old metal structures. Therefore this research activity is based on the collection of material data from more than 120 old bridges from Sweden, France and Germany.

The second topic is associated with the development of new assessment methods for resistance of riveted structures. This mainly concerns the study of resistance and deformation capacity of

cross sections formed by riveted slender plates. Modern standards for design of steel structures like Eurocode 3 cover riveted structures but they do not give complete information. Old design standards on the other hand are quite incomplete concerning instability phenomena and they cover elastic design only. In this research Eurocode 3 is considered as the starting point and some additional information relevant for riveted structures will be developed. The cross section classes in Eurocode 3 are essential in defining the resistance to bending moment. They are defined for rolled or welded sections but those definitions are not sufficient for riveted girders. First the maximum distance between rivets in the stress direction has to be defined. Further, there are some beneficial effects of confinement of plates in certain cases. The traditional method for assessing the resistance of steel bridges is based on elastic analysis. In case the resistance in ULS is insufficient it is possible that allowing for plastic deformations gives a more favourable answer. This is very obvious if the girders are stocky enough for using plastic hinge analysis. This is rarely the case but also more slender girders have some plastic deformation capacity, which can be utilized for a limited redistribution of moments in the girders.

The third research activity is related to the assessment of fatigue life of riveted and welded bridges. Fatigue related failures are the most common cause of failure of riveted bridges. Riveted structures were constructed over a period of more than 100 years up to the 1950s. A large number of riveted bridges (thousands) can be still found on the European railway networks. Constantly increasing loads and the fact that these bridges were not explicitly designed against fatigue raise questions regarding their remaining fatigue life. Economically its not justified to replace a bridge when it reaches the end of its *design life*. Often the design life it's an arbitrary value and there is considerable reserve. As is well known, metal fatigue exhibits high levels of uncertainty and can be influenced by a very important number of structure and environmental factors. An important amount of service life may be justified by a better knowledge of the fatigue behavior of riveted connections. Furthermore, the load history, which plays a main role in fatigue life evaluation, is largely unknown in most of cases. In that context, there appears to be a need to develop a comprehensive fatigue assessment methodology for riveted railway bridges.

Since the 1950s welding become a useful procedure for assembling components of metallic bridges. In welded joints cracks are often localized at the welding. Indeed the welding process induce some defects which help small cracks to appear. These defects can growth under cyclical loading and can induce the joint failure, and depending of the redundancy degree of the bridge can lead to the failure of the structure. The conditions governing crack growth are respectively structural geometry, initiation site, material characteristics and loading. In general all these conditions are highly random. Therefore, an appropriate analysis of fatigue phenomena consists by treating the problem in a probabilistic manner. Fatigue durability and inspection planning are then very important issues in the design and scheduled inspection of welded bridges. Usually welded structures have to be designed for a finite life with an accepted probability of failure based on S-N approach. Consequently cracks may propagate and become critical during the estimated *safe-life*, unless detected in time and repaired. If fracture is not acceptable, supplementary safety measures must be taken through in-service inspection requirements specifying appropriated non-destructive inspection techniques (NDT) and inspection planning. This leads to the *damage tolerance concept*: a welded joint containing a crack has to resist the service loadings for some time. All through this time there must be a large probability that the crack can be detected (and repaired) before it becomes critical. In order to verify this probability, the reliability against such a failure have to be evaluated as a function of service time in support of inspection strategy. The fracture mechanics and the reliability theory provides the necessary tools for these calculations. This approach (so called *probabilistic fracture mechanics*) is used on the assessment of welded joints of metallic bridges. This kind of analysis is carried out using the first order reliability method (FORM) or Monte Carlo simulations, which have become the standard methods in structural reliability. A limit state is formulated by applying linear elastic fracture mechanics (LEFM) the uncertainties of the main parameters can be considered by treating them as basic random variables. One problem, which usually appears, is that the necessary statistical informations of these variables (mean value, standard deviation, and distribution type) are not known. Another problem is that this kind approach does not give the statistical distribution of the cumulated damage, or in different words, the statistical distribution of the crack sizes at a given time. This information is important to evaluate the performance of different NDT methods used to control welded joints in bridges. An alternative approach, based on concept of Markov chain is proposed.

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Development of a guideline for load and resistance assessment of existing European railway bridges

J.S. Jensen
COWI A/S, Kongens Lyngby, Denmark

C. Melbourne
University of Salford, Salford, United Kingdom

J.R. Casas
Universitat Politècnica de Catalunya, Barcelona, Spain

R. Karoumi
Royal Institute of Technology, Stockholm, Sweden

M. Plos
Chalmers University of Technology, Göteborg, Sweden

A. Patrón
Lab. Central des Ponts et Chaussées, Paris, France

1 INTRODUCTION

In December 2003 a European project on railway bridges was commenced. The project is part of the European Commission's sixth framework programme and is an integrated project named "Sustainable Bridges – Assessment for Future Traffic Demands and Longer Lives". As the title indicates the main objectives for the project are to:

– increase the load carrying capacity of the existing railway bridges and allowing axle loads up to 33 tons for freight trains with moderate speeds
– increase the speeds for passenger trains and thereby allowing for a higher capacity for passenger traffic
– increase the service life of existing railway bridges by 25%
– enhance methods for strengthening and repair.

The present paper presents results from a work package in the project working on load and resistance assessment (WP 4).

WP 4 is dedicated to prepare a guideline for load and resistance assessment of existing European railway bridges which is assumed to present best practise.

2 REQUIREMENTS

The requirements for safety are divided into requirements to the safety format and to the safety level. The Guideline adopts the safety concept most commonly used in bridge engineering for design or capacity assessment: the limit state (LS) approach. The adoption of different safety formats is proposed in parallel with the use of less or more advanced levels of assessment. This is based on the philosophy of assessment assumed in the Guideline: to divide the assessment in different levels or phases with increasing level of sophistication.

The safety level is also considered at a member and system approach. The proposed target reliability levels proposed in different countries and by different international bodies (Eurocode, ISO) are presented in the Guideline, jointly with the most significant assumptions behind them. In this way, the engineer responsible for the assessment can choose the most suitable safety level for each specific case into consideration. Because bridge assessment is highly case-specific, the Guideline does not just propose a value to be adopted for the safety level as a general value, but gives information and guide how to fix this level for each case under study.

Similar to safety, the requirements about service life (durability) are assessed on the basis of a limit states format. The safety level and reliability requirements for service life might be different from the requirements for structural safety due to economic, social and sustainability considerations and the level of knowledge of the technical history of the bridge in question.

3 LOADS AND DYNAMIC EFFECTS

The working group on loads, in the Sustainable Bridges project, has initiated the following main research projects:

– dynamic behaviour and dynamic amplification factors for bridge elements,
– load distribution through ballast,
– assessment of actual loads using Bridge-Weigh-In-Motion (B-WIM),
– influence of rail roughness on dynamic behaviour of bridges.

Based on results from above research topics, recommendations for the assessment of existing railway bridges, in relation to loads and dynamic effects, will be reported in a chapter of the Guideline which is dedicated to this. By utilizing the recommendations, the engineer should be able to make improved predictions of load effects in the structure prior to permitting new traffic and/or increased speeds.

4 CONCRETE BRIDGES

The research work on concrete bridges is planned based on the need for improved methods and knowledge in the Guideline. The aim is to provide enhanced assessment methods that are able to prove higher load carrying capacities and longer fatigue lives for existing railway bridges. The work is focused on non-linear analysis and remaining fatigue life. The use of non-linear analysis will lead to higher load carrying capacities, but also to improved understanding of the structural response, forming a better basis for decisions in the assessment. The methods for fatigue assessment will lead to increased remaining service life of reinforced concrete railway bridges.

5 METAL BRIDGES

Four research activities are under development. The activities concerns the *Material properties of metal railway bridges*, the *Fatigue on riveted structures*, the *Updated assessment methods for riveted structures*, and the *Use of non-destructive tests (NDT) for load capacity reassessment*. A summary description of these research activities is presented in the paper.

6 MASONRY ARCH BRIDGES

In order to develop improved guidance for the resistance assessment of masonry arch bridges, a program of research has been initiated that seeks to address the following issues.

– modeling of the load path from the axle to the extrados of the arch barrel
– modeling of the effects of defects

– studying the behavior of masonry under cyclic loading
– determining the load carrying capacity using a probabilistic approach
– studying the soil-structure interaction.

It is not expected that the above will result in a definitive set of guidelines that will be true for all masonry arch bridges, but it is expected that a 'step change' in the assessment methods will result in an enhanced confidence in the guidance and a reassurance for bridge owners in their asset management systems.

7 CONCLUSION

A large part of the European railway bridges are rapidly getting closer to their service life end and at the same time the railway operators calls for higher axle loads for freight trains and higher speeds for passenger trains. This requires new and better approaches for assessing both the railway loads and the resistance of the railway bridges.

The guideline which will be developed in the project and is assumed to be first of its kind, is expected to provide the basis for a significant step forward in the efficiency in management of railway bridges in Europe with focus on both load and resistance.

Design and analysis

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Service and ultimate limit state of precast segmental concrete bridges with unbonded prestressing and dry joints

J. Turmo
Civil Engineering School, University of Castilla-La Mancha, Spain

G. Ramos & A.C. Aparicio
Civil Engineering School, Technical University of Catalonia, Spain

ABSTRACT: Segmental concrete bridges with external prestressing and dry joints are associated with a span-by-span construction process that is thought to be the fastest of its type. Its more significant characteristic is the nonexistence of bond reinforcement crossing the joints, neither active nor passive. In these bridges, Ultimate Limit State of bending can be defined in two ways. Eurocode 2 states that ULS is reached when the joints open in more than 2/3 of their height (ULS-EC2). Nevertheless, most of the codes define collapse of these bridges when the maximum strain in the concrete attains a value of 0.0035 (ULS-$\varepsilon_{\mathrm{II}}$). A 2D-FEM study is presented in this paper comparing the structural behaviour when these criteria are applied (Figure 1). The model used in the analysis has been previously validated with six beam shear tests. The most important feature of the FEM model is that it has been calibrated with experimental tests which were specifically designed to fail in shear.

A simply supported bridge as well a continuous bridge was modelled. The simply supported structure had a 50.00 m span and 2.70 m constant depth (slenderness relationship 1/18) consisting of 17 segments of 3.00 m length. The girder has a box type cross section (figure 3), with a 12.70 m width upper flange and a 5.70 m width bottom flange, both flanges of variable thickness. The continuous bridge is a five span, 48 m length per span and 2.70 m constant depth structure (slenderness relationship 1/18) consisting of segments of 3.00 m length. Each span included three independent families of prestressing strands, which were anchored at the intermediate on-pier diaphragms.

Some general conclusions can be extracted from the analysis. For simply supported bridges, it is the ULS that guides the structural design, which can be carried out either following the EC2 criterion or that of limiting the maximum compressive deformation of concrete at failure. For continuous bridges, it is either the ULS-EC2 or the SLS that guides the structural design. In hyperstatic structures, the limited rotations accumulated by the sections when the ULS-EC2 is reached does not allow the application of the plastic analysis to determine the state of stresses. The design that takes advantage of the stress redistribution leads to a prestressing force that does not satisfy service conditionings. Therefore, the criterion that limits the maximum concrete strain to 3.5‰ cannot be applied to this type of structures. Thus, the design of the prestressing force according to SLS o ULS-EC2 allows the structure to have a real and considerable capacity reserve.

Design following EC2 criteria leads to a stress state at ULS where no meaningful tensile stresses appear, except at the anchorage zones. On the other hand, if the design of simply supported bridges follows the criterion of limiting the maximum concrete deformation at failure, care should be taken regarding the reinforcement details of the segments.

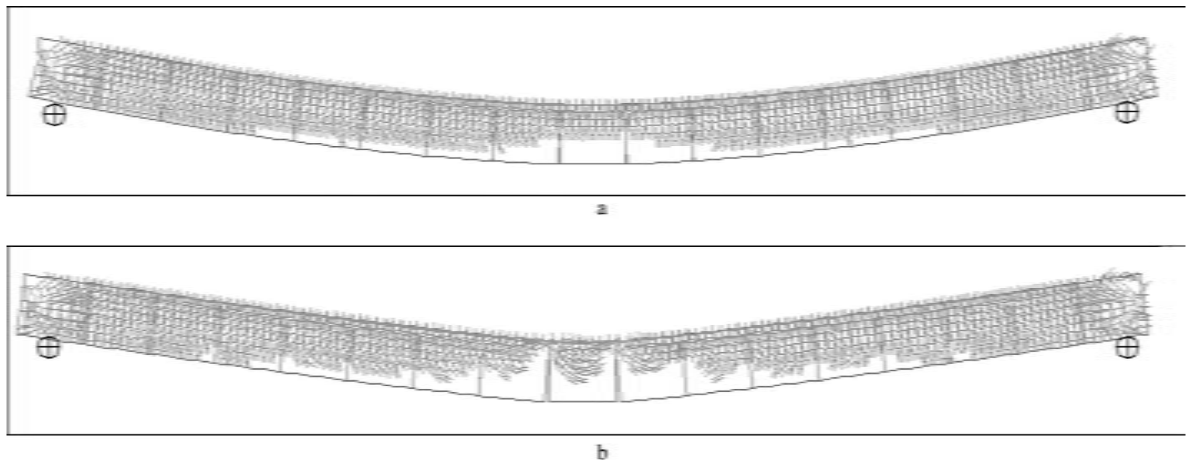

Figure 1. Comparison between the principal compressive stress field ($|\sigma_{II}| > 2.5$ MPa) at ULS- EC-2 (a) and ULS-ε_{II} (b).

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Airtrain JFK – the longest segmental girder construction erected in the New York city environs

Henry W. Hessing
URS, New York, USA

ABSTRACT: The fully automated JFK Airtrain was designated a finalist in the competition for ASCE's 2005 Outstanding Civil Engineering Achievement Award. The $1.9 billion Airport Access Project connects John F Kennedy International Airport (JFKIA) located in Jamaica, New York with two major intermodal connections – Long Island Rail Road (LIRR) and New York City Transit (NYCT).

Design/Build/Operate Maintain (DBOM) was the method selected to deliver this project.

DBOM shortened design and construction time by several years. The shortened time duration was reflected in lower overhead costs and a reduction in overall project costs. In the first year of operation, the PA estimated that 2.6 million passengers used AirTrain JFK.

Parking demand in the Central Terminal Area (CTA) decreased substantially because of the convenience JFK Airtrain provides. Comparing air passenger growth (in non-connecting flights) to increase in CTA parking demand, there is a net decrease in parking demand of 6,200 vehicles per day for the year 2005.

AT JFKIA, soil is deep, loose fine aggregates. Soil borings indicated the potential for liquefaction up to a depth of 20 feet under a seismic event with peak rock acceleration of 0.15 g. Additionally, the water table is high.

Besides calling for conventional design for forces and displacements, the seismic design criteria also required that additional limitations be met by the foundations and superstructure in order to allow the system to return to operation shortly after a seismic event meaning repairs would be limited to track work only.

Underpinning & Foundation Constructors, Inc. was awarded a contract to install more than 5,500 150-ton and 200-ton capacity piles to support the 8.3 mile long guideway. Underpinning invented a new pile type using a tapered steel lower section (25 feet) with steel pipe upper section (45–75 feet) and called it the Tapertube. These piles are 50 percent thicker than monotube piles, which allows them to be driven to higher driving resistances at lower stresses. The heavier steel also allows piles to be driven quicker once underway. The work was performed using hydraulic driving equipment built in Finland. The high mobility of the pile rigs, combined with highly efficient hammers, allowed for rapid installation in the tight confines of the active airport roadway system and in the center median of the heavily traveled Van Wyck Expressway." Underpinning & Foundation Constructors, Inc. 2005

Piles were topped with 20 foot by 20 foot by 5 to 8 foot deep concrete footings. Cast in place concrete piers were constructed on the footings and varied in size up to 45 feet in height and six (6) feet in diameter.

Preliminary design envisioned single and twin – box concrete girders. The alternate was a composite steel box with a composite reinforced steel deck slab.

The final design for the guideway superstructure was precast segmental construction utilizing seismic isolation. The primary defining features are single-cell or dual cell box girders with cantilevered deck slab.

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Analytical prediction of displacement capacity and length limits of integral bridges

M. Dicleli
Middle East Technical University, Ankara, Turkey

ABSTRACT: An integral bridge is one in which the continuous deck and the abutments are integrated to form a rigid frame structure. The abutments are generally supported on a single row of steel H-piles to provide the required flexibility for accommodating the longitudinal bridge movements due to temperature variations. In lieu of expansion joints at the bridge ends, such movements result in imposition of cyclic lateral displacements on the abutments, backfill and the steel H-piles. The magnitude of these cyclic displacements is a function of the level of temperature variation, type of the superstructure material and the length of the bridge. Many transportation agencies tend to push up the length limits of integral bridges to increase their range of application so as to eliminate the maintenance problems concerned with expansion joints in conventional bridges. Thus, the thermal-induced cyclic displacements in the components of such long integral bridges will become larger as well. Consequently, the piles may experience cyclic lateral deformations beyond their elastic limit. This may lead to a reduction in the service life of the bridge due to low-cycle fatigue effects in the piles. To minimize such detrimental effects, the lengths of integral bridges should be limited. Currently, rational guidelines to determine the maximum length of integral bridges do not exist. In general, state departments of transportation utilize in-house specifications that provide length limits for integral bridges based on the past performance of existing integral bridges. Therefore, a rational methodology is required to determine the maximum length limits for integral bridges.

The main objective of the presented research is to develop analytical tools for calculating the lateral displacement capacity and maximum length limits of integral bridges built on sand based on the low cycle fatigue performance of steel H-piles under cyclic thermal variations. For this purpose, an analytical equation is derived based on the low-cycle fatigue performance (i.e cyclic displacement capacity) of the steel H-piles under cyclic thermal variations. The methodology followed to achieve the main research objective is described below.

To formulate the displacement capacity and maximum length limits of integral bridges based on the low-cycle fatigue performance of steel H-piles under cyclic thermal variations, first, steel H-pile sections that may sustain large plastic deformations are determined considering their local buckling instability. Then, a low-cycle fatigue damage model is employed to determine the maximum cyclic curvatures and corresponding bending moments such piles can sustain. Next, this information is utilized in the static pushover analyses of two integral bridges to study the effect of various geometric, structural and geotechnical parameters on their displacement capacity as determined by the low-cycle fatigue performance of steel H-piles under cyclic thermal variations. Using these pushover analyses results and an equivalent cantilever concept for the piles, an equation is derived to estimate the displacement capacity and maximum length of integral bridges based on the low cycle fatigue performance (i.e. cyclic displacement capacity) of the piles. The derived equation is as follows:

$$\Delta_p = \frac{(\lambda l_c)^2}{6}\left[\frac{M_y}{E_p I_p}\left(1+\frac{M_y}{M_p}\right)+\frac{0.0085}{d_p}\left(2-\frac{M_y}{M_p}-\left(\frac{M_y}{M_p}\right)^2\right)\right]+n_p M_p \lambda l_c\left(\frac{L_D}{3E_D I_D}+\frac{h_c}{E_a I_a}\right)+\frac{n_p M_p L_D}{3E_D I_D}\left(\frac{(h_c+h_r)^2}{\lambda_v l_c}+(h_c+h_r)\right)$$

In the above equation, λ and λ_v are factors equal to 0.6 and 0.30 respectively. M_y and M_p are the yield and plastic moment capacity of the steel H-pile, n_p is the number of piles per girder, E_p, I_p, d_p and l_c are the modulus of elasticity, moment of inertia, width and critical length of the pile respectively, E_D, I_D and L_D are the modulus of elasticity, moment of inertia and length of the deck adjacent to the integral abutment respectively, E_a, I_a and h_c are the modulus of elasticity, moment of inertia, and cantilever length of the abutment respectively and h_r is the depth of the deck below its center of rigidity.

Assuming a typical construction temperature of 15°C, the positive and negative design temperature ranges for moderate and cold climates are calculated. Then, assuming A36 steel for the piles and medium-dense sand for the foundation soil, the length limits for steel and concrete integral bridges are calculated using the above equation. The results and conclusions are presented below.

The displacement capacity and maximum length limit of integral bridges are found to be complicated functions of the properties of the bridge, piles and foundation soil as observed from the equation presented above. It is found that the maximum length limit for concrete integral bridges ranges between 115 and 250 m. in cold climates and 140 and 310 m. in moderate climates and that for steel integral bridges range between 65 and 140 m. in cold climates and 95 and 210 m. in moderate climates. It is also found that the displacement capacity of integral bridges decreases considerably for sands with larger density. Generally, the orientation of the piles about their strong axis of bending is recommended for enhancing the displacement capacity of the bridge as determined by the low-cycle fatigue performance of piles especially in sands with larger densities. Furthermore, taller abutments are found to enhance the displacement capacity of integral bridges based on the piles' ability to sustain cyclic thermal-induced displacements. Moreover, Integral bridges with larger size or stiffness are found to have smaller displacement capacity as determined by the piles' low-cycle fatigue performance under thermal variations. Finally, it is found that concrete bridges are more suited for integral bridge construction as they are less sensitive to temperature variations and are recommended especially in cold climates.

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Effect of thermal displacements on the performance of integral abutment-backfill system

M. Dicleli
Middle East Technical University, Ankara, Turkey

ABSTRACT: Bridge expansion joints have caused considerable maintenance problems to transportation agencies (Wolde et al. 1988a, b, Steiger 1993). Therefore, in many parts of North America, the overall economy of jointless construction is made applicable to bridges where the abutments are cast integral with the deck and supported on a single row of steel Hpiles. In such integral bridges, the abutment-backfill system and the abutment piles (Dicleli 2005) may be influenced considerably by the thermal-induced displacements of the continuous bridge deck. The magnitude of the deck displacements is a function of the level of thermal variation, type of the superstructure material and the length of the bridge. As the length of integral bridges increases, thermal-induced displacements and the forces in the bridge components may increase as well. Since many transportation departments tend to push up the length limits of integral bridges to eliminate the maintenance problems concerned with the expansion joints, the abutments of such long bridges may be subjected to large internal forces in excess of their ultimate capacity. A reasonable estimation of the internal forces in the abutments is necessary in the design to ensure satisfactory performance of the integral bridge throughout its service life. This requires a careful study of the behavior of the abutment-backfill and soil-pile system under thermal variations. At present, design guidelines for determining the forces in integral bridge abutments considering the effect of thermal displacements on the shape and intensity of the backfill pressure and the effect of the pile-soil system do not exist in AASHTO (American Association State Highway Transportation Officials) Bridge Design Specifications (1998). Therefore, a rational method for the design of integral bridge abutments is required. Such a method may also be useful in determining the length limits of integral bridges based on the ultimate strength of the abutments.

Accordingly, the main objective of the presented study is to develop rational design equations for determining the maximum forces in the abutments and maximum length of integral bridges based on the strength of the abutments. The presented study is limited to symmetrical, non-skewed integral bridges with expansion bearings on intermediate supports. The abutments are supported by steel H-piles driven in sand. A fixed connection is assumed between the piles and the abutment based on the current state of practice. The piles are assumed to accommodate large plastic lateral deformations without global instability. Furthermore, the abutments are assumed to sustain the effects of the internal forces produced by thermal variations until the piles reach their plastic curvature limit. This enabled the examination of the abutment-backfill behavior under a large range of thermal-induced displacements (e.g. 2" for bridges with stub abutments). The restraining effect of the approach slab to the longitudinal movements of the bridge is found to be negligible and is not included in the analyses.

To reach the above stated objectives, first, structural model of a typical integral bridge is built considering the nonlinear behavior of the piles and soil-bridge interaction effects. Static pushover analyses of the bridge are then conducted to study the effect of various geometric, structural and geotechnical parameters on the performance of the abutment-backfill system under positive thermal variations. Using the pushover analyses results, design equations are formulated to determine the maximum forces in the abutments and maximum length of integral bridges based on the strength

of the abutments as a function of the thermal induced displacements in the bridge deck and the properties of the backfill and pile-soil system.

It is found that the magnitude of the internal forces in the abutments and the backfill pressure intensities are functions of the thermal-induced abutment displacements. Therefore, the internal forces in the abutments need to be calculated in correlation with thermal induced displacements of the bridge. For this purpose, analytical equations are derived to calculate the abutment's maximum shear forces and bending moments as a function of the thermal-induced abutment displacement, sub-grade reaction modulus of sand and properties of the steel H-pile. In general, the density of the sand and size and orientation of the steel H-piles are found to have only negligible effects on the shape and intensity of the backfill pressure. These negligible effects gradually diminish as the displacement of the abutment towards the backfill increases. It is observed that the internal forces in the abutments increase for denser sands. Therefore, the effect of sand density needs to be considered in the design of integral bridges. At small abutment displacements where the backfill and the sand around the piles remain within the elastic limits, the size and orientation of the piles do not have a significant effect on the magnitude of the abutment's bending moment and shear force. However, at larger abutment displacements, as the size of the pile increases, the abutment internal forces increase as well. The abutment height is found to have a notable effect on the shape of the backfill pressure distribution and magnitude of the internal forces. The pressure distribution assumes a triangular shape for abutments shorter than 3-m and a parabolic shape for abutments taller than 5-m. Nevertheless, the classical triangular distribution may be considered as a reasonable conservative estimate for the purpose of design. As anticipated, the bending moment and shear force is larger for taller abutments. It is observed that compacting the backfill results in very large backfill pressure intensities, and abutment internal forces. Therefore, using non-compacted backfill is strongly recommended in integral bridges to reduce the abutment shear and flexural forces and to maximize the length limits of integral bridges as determined by the flexural capacity of the abutment. For abutments taller than 4-m, the flexural capacity of the abutment may control the maximum length limit of integral bridges under positive temperature variation. Therefore, stub abutments are strongly recommended in integral bridge construction. Generally, the orientation of the piles about their weak axis of bending is recommended for enhancing the maximum length limits of integral bridges as determined by the flexural capacity of the abutment.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Development of a steel-concrete composite bridge deck with perfobond ribs

H.-Y. Kim & Y.J. Jeong
Korea Institute of Construction Technology, Goyang, Gyeonggi-Do, Republic of Korea

S.-K. Park & K.B. Han
Sungkyunkwan University, Suwon, Gyeonggi-Do, Republic of Korea

Y.K. Shin
Kyongdong Engineering Co. Ltd., Anyang, Gyeonggi-Do, Republic of Korea

ABSTRACT: This paper deals with a steel-concrete composite bridge deck with perfobond rib shear connectors. The proposed deck system is composed of profiled steel sheeting, perfobond ribs, steel reinforcements, and concrete. Figure 1 shows a schematic of the composite bridge deck system proposed in this study. The deck consists of profiled steel sheeting, perfobond rib shear connectors, steel reinforcements, and concrete. The perfobond rib acts as a shear connector and as a longitudinal stiffener within the profiled sheeting. The perfobond rib shear connector consists of a steel plate with a number of uniformly spaced holes. If the holes in the perfobond rib are filled with concrete, concrete dowels are formed, which provide horizontal shear resistance between the steel and the concrete.

In this study, a push-out test was conducted to determine the spacing of holes within perfobond rib. The diameter of hole for perfobond rib was assumed as 0.42D (50 mm), where D is the diameter of hole. The spacing of holes was assumed as 1.5D, 2.0D, and 3.0D. The results of test have indicated that the influence of spacing of holes on the ultimate shear resistance was insignificant.

A full-scale flexural test under combined action of shear and bending is required to verify the effectiveness of the shear connectors used for the composite decks. To examine the load and deflection behavior of a composite deck with perfobond ribs, eight full-scale deck specimens were fabricated and tested. The 1.0 m × 3.7 m deck specimen was simply supported by a clear span (L) of 3.5 m. The shear span ratio of the specimen was assumed as L/2, L/3, L/4, and L/5.3. For the comparison purpose, typically designed reinforced concrete deck specimens were also tested.

The composite deck specimens showed linear load-displacement behavior up to failure. As the load increased, the cracking of concrete occurred near the support of the specimen. A number of cracks began at the bottom of the concrete near the support and progressively spread towards the top of the concrete at loading point, as the load further increased. The measured end slip at the interface between the sheeting and the concrete was nearly zero until the load reached 500 kN. The maximum horizontal end-slip at the interface was less than 1.6 mm. The maximum load was reached just before the failure that showed a brittle behavior.

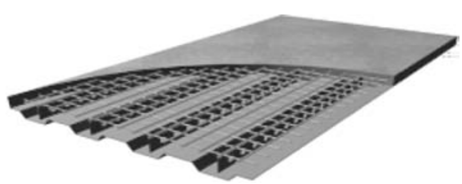

Figure 1. Schematic of proposed deck profile.

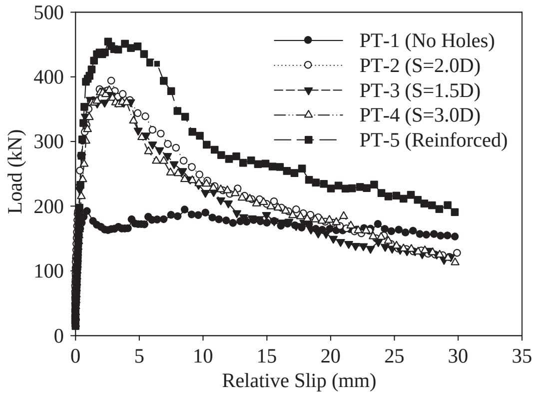

Figure 7. Load-relative slip curves for push-out specimens.

Figure 9. Set-up for full-scale test.

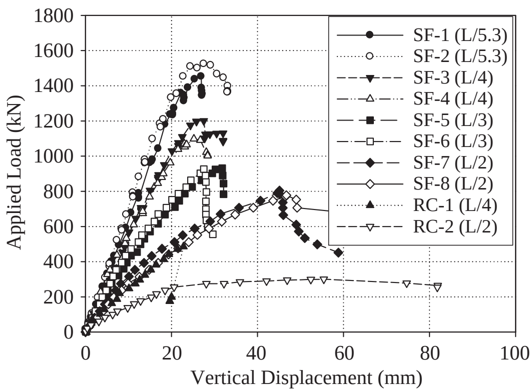

Figure 10. Load-displacement curves for specimens.

The results of this experimental study indicated that the perfobond rib shear connection can be effectively used for the proposed deck system. In Eurocode 4, the behavior of composite decks can be defined as ductile if the failure load exceeds the load causing the end slip of 0.1 mm by more than 10%. Therefore, the behavior of the composite deck tested in this study is considered ductile.

The flexural strength of the proposed deck system under positive bending is approximately 2.5 times greater than that of a typically designed cast-in-place concrete deck with the deck weighing 30% less. The initial concrete cracking load for reinforced concrete deck was about 14% of the proposed deck system. This may be attributed to the potential advantages of the proposed deck system over the conventional one.

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Static performance of concrete encased composite columns with low steel ratio

Y.-S. Chung, C.-S. Shim, C.-K. Park & J. Min
Chung-Ang University, Ansung-Si, Korea

ABSTRACT: The advantage of concrete-encased composite structural members is that the concrete used for covering a structural steel not only increases its strength, but also improves ductility. In this paper, experiments on shear strength of the concrete-encased composite column were performed to investigate the effect of confinement by transverse reinforcements, mechanical interlock by holes, and shear connectors. Confinement, mechanical interlock and stud connectors increased the shear strength effectively to secure a composite action of SRC columns. Seven concrete-encased composite specimens were fabricated. The cross-sections of these specimens are composed of concrete-encased H-shaped structural steel columns and a concrete-encased circular tube with partial in-filled concrete. Through the static test, it was evaluated the strength and ductility of SRC composite specimens. The specimens were comprised of structural steel ratio under 2%. The member strengths of the specimens are calculated based on the Eurocode-4, ACI318 code and AISC-LRFD specification. Test results are compared with the P-M interaction curves based on the Eurocode-4, ACI318 code and AISC-LRFD specification. The Eurocode-4 and ACI318 approach generally give closer predictions than AISC-LRFD does. It was concluded that structural capacities of concrete-encased composite columns were excellent in bending strength and ductility. Especially, the comparison results revealed that the concrete-encased circular steel tube column showed the best performance in the structural capacities.

1 INTRODUCTION

Design provisions for SRC columns are currently included in ACI-318, AISC-LRFD and Eurocode-4. Experimental research for these codes was mainly on the encased composite columns with high steel ratio, which is more than 4% for building structures. According to the several researchers (Saw and Liew 2000), these provisions often give significantly different nominal member strengths because each code treats the design of SRC columns through extensions to provisions for reinforced concrete and structural steel columns, respectively. Even though there were many tests on SRC columns, the current state of reliability on composite columns is in need of improvement. Especially, it is necessary to investigate the structural capacity of SRC columns for bridge piers with low steel ratio, which is less than 2%.

In this paper, two aspects of the composite column design were treated through the static tests. Push-off tests were performed to study the effect of confinement by transverse reinforcements, mechanical interlock by holes, and shear connectors. SRC columns with a H-shape rolled beam and with a circular steel tube were fabricated and statically tested under bending only and under compression-bending.

2 EXPERIMENTAL WORK

For the evaluation of bond strength of SRC columns, an experimental study was made on 8 push-off specimens. The interface length of the specimen was 300 mm and the cross-section of the column was circular with a 400 mm outer diameter. The main parameters were confinement reinforcements,

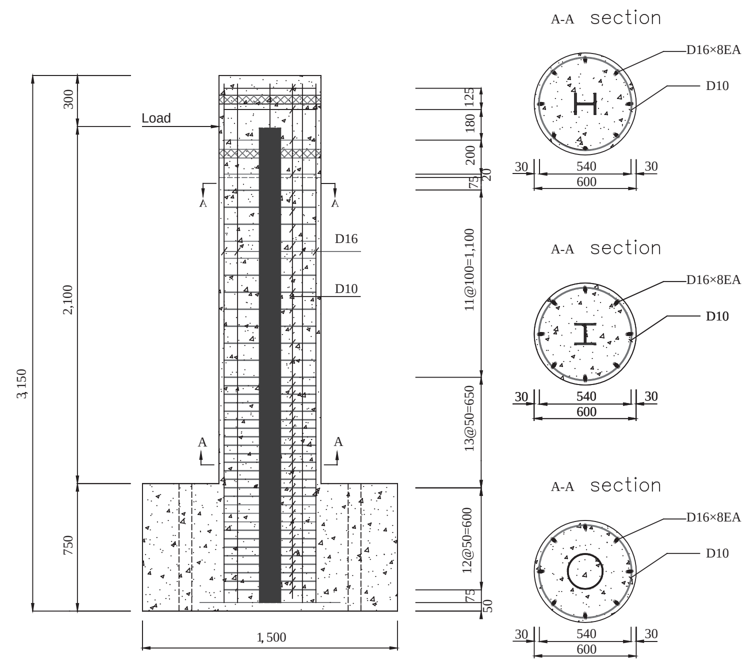

Figure 1. Test specimens cross section (unit: mm).

mechanical interlock, and mechanical connectors. The shear strength of the interface with no mechanical treatment showed the lowest values but much greater than the design shear strength of the steel encased composite section in Eurocode-4.

Five composite columns with embedded H-shape steel beam and two columns with partially concrete filled steel tube were fabricated to investigate the static behavior of SRC columns. Main parameters were embedded steel member, loading condition, confinement reinforcement ratio. In the notation of the specimens, BA indicates H-shape rolled beam loaded to the strong axis and WA means weak axis loading. SR indicates low confinement reinforcement ratio, 0.53% for the limited ductility requirement. CT member had a steel tube of 5.3 mm thickness filled with concrete up to 600 mm from the bottom of the column. Total longitudinal steel ratio including a steel member and reinforcements for CT specimen was 1.77% and 1.63% for the other specimens. Yield strength of the embedded steel was 137.3 MPa and main reinforcements and confining reinforcements were 343MPa and 333 MPa, respectively. The 28-day compressive strength of concrete was 24.0 MPa.

Seven composite columns with steel section completely covered by concrete were statically tested and it is concluded that for composite columns with low steel ratio:

1. The composite columns with low steel ratio showed significantly enhanced ductility when the transverse bars are designed as common RC columns considering the seismic actions. Especially, composite columns with a partially concrete filled steel tube gave the greater strength and ductile behavior than those with a H-shape steel beam due to the confined concrete in the plastic hinge of the tube.
2. In the region of this study, the amount of confinement reinforcements did not affect significantly on the flexural strength of the SRC columns. However, it is important to provide adequate confinement reinforcements to ensure the ductile behavior for the seismic design. Encasing core steel makes the deformation capacity of SRC columns to be large.
3. When composite columns having structural steel ratio less than 2.0% are designed, it is appropriate to evaluate the P-M interaction diagram by concrete-based design codes such as ACI-318 and Eurocode-4.

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Unconventional high performance steel bridge girder systems

H.H. Abbas, B.-G. Kim & R. Sause
Advanced Technology for Large Structural Systems (ATLSS) Center, Department of Civil and Environmental Engineering, Lehigh University, Bethlehem, Pennsylvania, USA

ABSTRACT: The use of high performance steel (HPS) in the construction of steel bridge girders in the USA has been growing steadily in recent years. HPS offers several improved properties over conventional bridge steels, including high strength, excellent fracture toughness, good weldability, and resistance to corrosion, which makes it well-suited for highway bridge applications. The use of HPS may lead to considerable weight savings in design (e.g., 15 to 20%), although design limits that impeded their effective use in conventional I-girders with transverse stiffeners include web instability, excessive deflections, and fatigue (Sause 1996). To overcome these limitations, unconventional I-girder systems have been proposed (Wassef et al. 1997, Sause & Fisher 1995), including I-shaped girders with corrugated webs (CWGs) and I-shaped girders with concrete-filled tubular flanges (CFTFGs), shown in Figure 1.

The primary advantages of CWGs (Fig. 1a), compared with conventional I-girders with transverse stiffeners, are: (1) the corrugations provide web stability under shear and CWGs do not require transverse stiffeners to resist shear; (2) critical Category C′ fatigue details are eliminated because transverse stiffeners are not used; (3) corrugated web girders are lighter in weight (Wassef et al. 1997); and (4) CWGs have greater potential for automated fabrication than I-girders with transverse stiffeners. The primary advantages of CFTFGs (Fig. 1b and 1c), include: (1) the concrete-filled tubular flange provides more strength, stiffness, and stability than a flat plate flange with the same amount of steel, reducing the need for bracing; and (2) the vertical dimension of the tube reduces the depth of the web, reducing problems with web slenderness design limits.

CWG and CFTFG systems have been the subject of recent research programs at the ATLSS Center at Lehigh University. The research on CWGs included studies of elastic flexural-torsional behavior (Abbas et al., in press), shear behavior (Driver et al. 2006), fatigue life (Sause et al., in press), and fabrication procedures (Sause 2003). The first three areas are summarized in the paper. The research on CFTFGs focused on CFTFGs for simple span bridges, where the concrete-filled tube is used as the top (compression) flange. The research included initial design studies, analytical

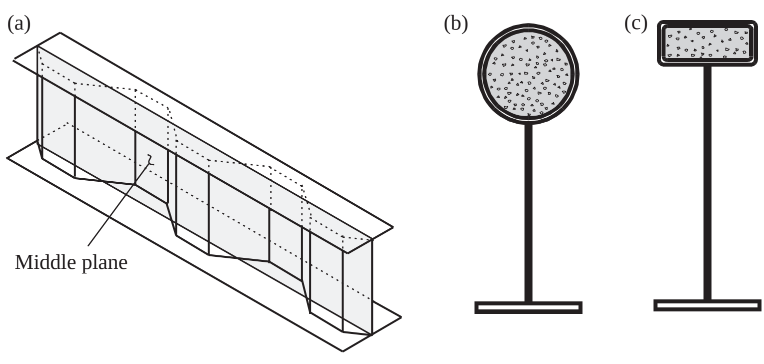

Figure 1. Unconventional bridge girder systems: (a) corrugated web I-girder; (b) & (c) concrete-filled tubular flange girders.

parametric studies, and experimental studies (Kim & Sause 2005, Wimer & Sause 2004). The first and the last of these three areas are summarized in the paper.

Based on design recommendations that resulted from these research programs, the Pennsylvania Department of Transportation (PennDOT) has recently completed the design and construction of a demonstration HPS highway bridge that uses CWGs, and is currently developing the concept for another demonstration bridge that utilizes CFTFGs. These demonstration highway bridges, which are briefly described in the paper, are the first of their kind in the United States.

The paper shows that the use of a corrugated web eliminates the need for transverse stiffeners, and can lead to a lighter girder weight by minimizing the web thickness. Moreover, the fatigue life of a CWG is longer than that of a conventional I-girder with transverse stiffeners. CFTFGs provide more strength, stiffness, and torsional stability than a conventional I-girder with equal weight. The torsional stability can be used to significantly reduce the number of interior diaphragms. It is well known that fatigue crack initiation in conventional I-girders is often associated with diaphragm connection details and other stiffener details. Eliminating or reducing the number of these details by using CWGs or CFTFGs has the potential to reduce inspection and maintenance costs, although this feature of these unconventional girder systems is yet to be studied. Finally, it is worth noting that the CWG and CFTFG systems can be combined, and studies on CFTFGs with corrugated webs have been reported recently by Kim et al. (2005).

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Steel bridge system – simple for dead load, continuous for live load

A. Azizinamini & D.T. Kowalski
National Bridge Research Organization, University of Nebraska-Lincoln, Lincoln, Nebraska, USA

ABSTRACT: This paper describes the design and construction of a new steel bridge system that is being used on a more regular basis in the U.S. The system is designed to force the elements of the bridge to act as simple spans for dead loads and continuous under any superimposed loads, after the concrete has cured. This paper provides highlights of the design and construction of the first bridge in the U.S. that utilized this concept in conjunction with the use of box sections. The use of this system has resulted in significant economy and is making steel bridges competitive with alternate concrete bridges. Following Nebraska, Tennessee, Illinois, New York, Colorado and Oregon have recently begun using this new system.

A joint effort by the Nebraska Department of Roads (NDOR), Federal Highway Administration (FHWA), National Steel Bridge Alliance (NSBA), National Bridge Research Organization (NaBRO) at the University of Nebraska-Lincoln (UNL) and the steel industry recently led to the design, fabrication and construction of the first steel bridge in the U.S. utilizing 689 MPa (100 ksi) High Performance Steel. The project is part of the FHWA IBRC initiative. The bridge is located near Grand Island, Nebraska where highway N-2 passes over interstate highway I-80. The N-2 over I-80 bridge is a two-span steel box-girder bridge, with each span being 42.4 m (139 ft) long. Since the bridge crosses over the I-80 interstate highway, minimizing the interruption to traffic was a major design consideration. The bridge was designed by NDOR and NaBRO at UNL. It incorporates several features that facilitated both fabrication and construction of the bridge and it incorporates unique and innovative design features.

Traditionally, in Nebraska, the maximum bridge girder length has been limited to approximately 36.6 m (120 ft). This limitation reflected the crane capacity of the fabricators, rather than the shipping limitations. The maximum crane capacity for most fabricators in the region is limited to approximately 267 kN (60,000 lbs). Therefore, the maximum weight of each girder had to be kept under this weight limit.

The design of the box-girders was based on the assumption that the girders would utilize an hybrid arrangement, with the bottom flanges of the box sections using 483 MPa (70 ksi) High Performance Steel and webs and top flanges utilizing conventional 345 MPa (50 ksi) steel. Based on the parametric study carried out on this hybrid arrangement, it was determined to be the most ideal for the bridge. After the design was completed, conservatively the 689 MPa (100 ksi) HPS was substituted for all webs and flanges. The intent was to demonstrate that there is nothing unusual about 689 MPa (100 ksi) HPS and that it can be fabricated and constructed similar to other steel types.

The use of HPS in a hybrid arrangement limited the maximum weight of each girder to 267 kN (60,000 lbs) – the maximum crane capacity of the local fabricators, while increasing the girder lengths from the traditional 36.6 m (120 ft) to 42.4 m (139 ft). The use of HPS plates allowed the use of thinner material for the bottom flanges and reduced the web depth, while meeting all design limitations.

The webs of the girder were perpendicular to the bottom flange and were not sloped. This significantly reduced the fabrication time and cost. This allowed the fabricator to use equipment and procedures commonly used for fabricating I-shapes.

A unique feature utilized in the bridge was a new system developed at NaBRO. This system is referred to as simple for dead load, continuous for live load. In this system, each individual girder is placed over the supports, in this case over the abutment and middle pier. Prior to hardening of the deck slab, each girder behaves as a simple beam, with maximum moment being at the middle of the span. The detailing is such that the bridge behaves as continuous for any loads applied after the concrete deck has hardened. This includes the live loads due to traffic and dead loads that are applied after the concrete has hardened, such as the weight of the barrier and overlays. The continuity for live and superimposed dead loads is provided by placing additional reinforcement in the slab over the pier, similar to what has been practiced in prestressed concrete bridges for years. In the simple for dead load, continuous for live load bridge system, the need for bolted splices is eliminated, significantly reducing costs.

In the new system, the girders are joined together over the pier by casting a concrete diaphragm. The challenge is to connect the girders over the pier in such a manner that would allow them to act as a simple beam during casting the slab. Under the live loads, the bottom flanges of the girders near the pier are subjected to relatively large compressive forces. This compressive force has to be transferred to the concrete diaphragm, joining the girders over the pier. Therefore, if the girders were simply embedded in the concrete diaphragm, there would be a possibility of crushing the concrete in the diaphragm near the bottom flanges of the girders. The same possibility also exists for prestressed girders. However, the difference between steel and concrete girders is that in the case of prestressed girders, bottom flanges are larger and the compressive forces are transferred to a larger area, reducing the compressive stress applied to concrete diaphragm near the bottom flanges. Further, the modulus of elasticity of steel is several times larger than concrete. Full scale tests carried out at NaBRO confirm the fact that under even service loads, there would be a possibility of crushing the concrete in the diaphragm near the bottom flanges if the steel girders are directly embedded in the concrete diaphragm. For the case of I-girders, preliminary research has been conducted at NaBRO to come up with a possible solution. Test data indicates that the proposed detail passes both fatigue and ultimate load tests.

The use of the new steel bridge system, namely simple for dead load and continuous for live load, significantly reduced the time required to place the girders over the abutment and middle pier. The I-80 interstate highway had to be closed for only 90 minutes for placing the three girders for each span. This significantly accelerated the construction time and reduced the interruption to traffic.

The N-2 bridge over I-80, by utilizing several unique design features and HPS, resulted in a steel bridge system that demonstrated faster construction and reduced cost. The use of HPS allowed the increase of the span length of each girder to beyond the traditional 36.6 m (120 ft), while keeping the total weight of each girder below the crane capacity of local fabricators. The use of the new steel bridge system also allowed the elimination of bolted splices. The girders were connected over the pier with a concrete diaphragm, using a detail recommended and developed by NaBRO.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Experimental tests of behaviour of unconventional steel-soil structure

D. Beben
Civil Engineering Department, Opole University of Technology, Opole, Poland

Z. Manko
Institute of Civil Engineering, Wroclaw University of Technology, Wroclaw, Poland

ABSTRACT: This paper discusses the results of tests of a steel, single-span unconventional road bridge constructed from flat steel sheets attached together with joints, put under trial static and dynamic loads. It is a detour bridge located in Makolno near Klodzko (the South-West of Poland), intended for use until the reconstruction of the existing bridge has been completed.

1 INTRODUCTION

As it is a prototype character a structure other than the one planned in the original reconstruction plan of the existing bridge, displacement tests and their detailed analysis were necessary. According to the results, the load-carrying capacity of the bridge was characterised and, subsequently, temporary road traffic for the period of approximately four months was admitted (Manko and Beben 2002).

The tests were directed to recognise the response behaviour of the arch structure under determined loads in order to verify the brief design assumptions (Michalski 2002). It mainly applied to the assessments of the real rigidity of the load-carrying structure made of flat steel sheets as well as to the width of the road base layers interacting in load transfer, especially transverse load distribution onto the particular sheets (load distribution range). The tests were carried out at two bridge span sections, with the consideration of two diagrams of asymmetrical loading locations along the longitudinal axis of the span.

2 DESCRIPTION OF THE BRIDGE STRUCTURE

The tested bridge is a single-span structure in the form of restrained arch with the main span effective length of the load carrying structure amounting to $l_t = 5.25$ m (Figures 1 and 2). The steel structure was rested on the ground with use of steel rolled channel sections rested on concrete long strip footing. The steel shell was implemented within an arc with the radius 2.75 m. The overall width of the bridge amounts to 16.50 m while the angle between the intersection of the detour axis and the obstruction (a stream) is approximately 40°.

The load carrying structure consists of a layer of flat steel sheets. They are 23 mm thick and 1.50 m wide, arched, and jointed to each other along the entire width of the span. The shell structure was covered with 0.20 m thick layers of soil suitably compacted ($I_D = 98\%$ – according to Proctor Normal Density) to allow road surface placement on a break stone layer. During the soil compaction period, a support system of the PPRM scaffolds was introduced along the overall width of the structure to avoid any uncontrolled transfer of the steel sheets (see Figure 1). The thickness of the soil layer over the steel structure amounts to approximately 0.80 m, at the underside of the crown. Both lateral edges of the bridge were secured by gabions that are steel buckets consisting of nets filled with rubble.

3 THE RANGE OF TESTS UNDER STATIC AND DYNAMIC LOADS

In tests, two vehicles, Liaz and Jelcz, were used, with the carrying capacity 18.9 Mg and 23.6 Mg, respectively. For the field static load, the ultimate numbers of vehicles accommodated on the tested span, with the biggest acceptable axle load were used each time. They were positioned along the vertical and horizontal axis of the span in such a way that the maximal values of deflection and strain were achieved in the tested cross-sections of the structure.

Full range tests under static and dynamic load were performed, including both the displacement and strain measurements of selected points in given cross-sections of the structure. Indirectly, normal stress was tested. In load scheme I, the two vehicles were positioned asymmetrically along one of the edges of the road, on the side of the tailwater. In scheme II, they were also placed asymmetrically on the opposite side of the road, in reference to scheme I. The assessments of long strip footing settlement were omitted, as it was irrelevant for this structure.

4 THE RESULTS OF STATIC AND DYNAMIC MEASUREMENTS

Prior to introducing the load onto the span of the soil-steel bridge, zero readings were taken on the measuring instruments. After the move of the ballasting load, the read out was taken every 10 minutes for at least 30 minutes, and after unloading – every 10 minutes for at least 20 minutes till the read out was stabilised.

The dynamic measurements were performed directly after the field static load. The results were recorded by means of two indicators. Indicator A was placed on the edge of the asphalt surface while indicator B – on the gabion (see Figure 5). Forced vibrations of the bridge were induced by one vehicle with carrying capacity 18.9 Mg passing by a threshold sized 0.03×0.10 m, placed in the mid span, with the speed of 30 km/h. The amplitude of vibration and time measurements is presented in Figures 6 and 7. Greater amplitude of vibration at point B might have resulted from the vibrations of the gabion which was not firmly attached to the bridge structure or the road surface. The greatest amplitude of vibration speed of the road surface at the measuring point A, when the first axle of the vehicle run over the threshold, amounted to 4.53 m/s. Increasing speeds to 35 km/h resulted in enhancing the amplitude to 10.00 m/s. The frequency of vibrations of the span (27.937 Hz) induced by the vehicle passing was determined according to analysis of the obtained time amplitudes.

5 RECAPITULATION AND FINAL CONCLUSIONS

1. Displacements measured in selected elements of the steel shell structure proved significantly smaller (by up to approximately 48%) than expected and smaller than allowable deflections (Manko and Beben 2002).
2. Strain quantities were smaller than expected by approximately 55%; this applies to almost all of the considered points and sections of the load carrying structure.
3. Slight differences between initial and final read outs of displacements and strains prove that slight displacements and permanent strains detected during the measurements of the load carrying structure resulted, to a certain extent, from permanent deflections of the welded shell structure, error readings of the measuring instruments, and possibly, settlement of the long strip footing.
4. The span and its supports did not provoke any concern during the trial load. The average measured deflection and strain quantities were significantly smaller than those computed in reference to the same load. This shows that the real rigidity of the structure is much higher than it was assumed in the static-strength calculations (Michalski 2002).
5. As a result of the performed dynamic tests, slight amplitudes of proper vibrations were obtained. The vibration frequencies were considerably smaller than those allowable in this kind of structures (Manko and Beben 2002).
6. On close inspection of the bridge, after the experimental research had been completed, and during complementary and verification tests, no damage to structural elements was reported.

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An experimental study of soil-arch interaction in masonry bridges

M. Gilbert & C.C. Smith
Department of Civil and Structural Engineering, University of Sheffield, UK

C. Melbourne & J. Wang
School of Computing, Science and Engineering, University of Salford, UK

1 INTRODUCTION

It is now well established that soil-arch interaction has a significant influence on the load carrying capacity of a typical soil-filled masonry bridge. However, the backfill material is typically treated in a rather crude manner in existing masonry arch bridge analysis/assessment procedures. For example, in most mechanism analysis programs the backfill is modelled indirectly, with the amount of live load spreading and the magnitude and form of the passive restraining pressure distribution being specified separately. A major simplification is that both these are typically assumed not to depend on the mode of response of the bridge. Consequently there is a need for improved soil-arch interaction models; high quality experimental data is needed to validate such models.

However, most laboratory bridges tested to date have been constructed between rigid abutments and have been backfilled with a granular fill material. In contrast, recent intrusive investigations performed on local authority owned bridges in the UK have frequently identified that abutments are relatively insubstantial (e.g. of comparable thickness to the arch barrel), and that clay backfill is present. Hence more experimental data is clearly required. This paper therefore describes the development of a new full scale masonry arch bridge test rig facility, and also provides some initial results.

2 TEST RIG & INSTRUMENTATION

A large (8.3 m long $\times$ 2.1 m high) testing tank has been designed to house test bridges. The tank sidewalls have been designed to be extremely stiff to provide effectively plane strain conditions.

The tank also incorporates large observation windows along one face to permit digital imaging and subsequent particle image velocimetry (PIV) processing of the soil and arch movements. Currently a set of six digital cameras are positioned at intervals along the length of the tank to capture images.

Additionally, conventional displacement transducers are positioned to measure both arch and frame movements in real-time, whilst soil pressure cells are being used to measure soil-arch pressures. Acoustic emission gauges have also been placed on the arch barrels of the initial test bridges, to enable real-time fracture activity to be recorded.

3 TEST BRIDGES

Two bridges have been tested to date, with the overriding initial objective being to prove the test apparatus described.

The first test bridge was designed to be as similar as possible to the 3 m span bridges tested at Bolton in the 1990s (Melbourne and Gilbert 1995), thereby permitting direct comparisons to be made. Thus a similar crushed limestone fill material was selected. However, unlike the Bolton bridges, which had been constructed between rigid abutments, potentially movable abutments were specified.

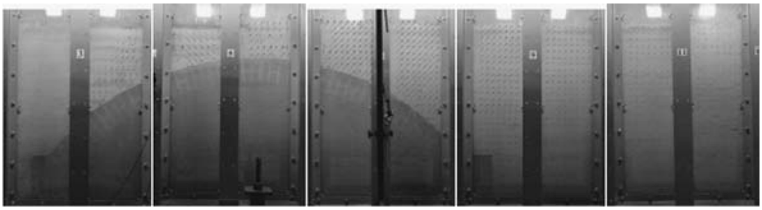

Figure 1. Images of arch and limestone backfill with PIV soil displacement vectors superimposed.

The second test bridge was designed to be identical to the first with the exception that fill material below the level of the crown of the arch was replaced with a soft clay, representative of that found in some local authority owned bridges in the UK. Crushed limestone was used above the crown to reflect the fact that competent near surface road/sub-base material is normally present in real bridges. Additionally, had this not been used then a local failure of the soil in the vicinity of the applied load would have been likely.

4 TEST RESULTS

Both bridges ultimately collapsed in four-hinged mechanisms. However, prior to ultimate collapse, some spreading of the abutments was recorded (up to ¼ 8 mm in the case of the clay-filled bridge).

Imaging and PIV processing proved very successful. An example processed image is shown in Fig. 1. Here a significant soil wedge is present on the passive side of the arch. Full analysis of the results is currently in progress. This work also involves detailed comparison between the results from the tests and results from newly developed numerical soil-arch interaction models; these will be described elsewhere in due course.

Deformations of the frame were small and within the design limits set (maximum of ¼ 0.3 mm and ¼ 0.8 mm in the cases of the limestone and clay filled bridges respectively).

5 CONCLUSIONS

- Many masonry arch bridges in the UK have been found to contain clay backfill and abutments which are relatively insubstantial (e.g. of comparable thickness to the arch barrel). To properly investigate the behaviour of such bridges, and to further improve our understanding of granular soil-arch interaction, a new large-scale plane strain laboratory test rig has been developed; details of this have been outlined in the paper.
- Tests on two pilot bridges have been described. One bridge was backfilled with clay fill, the other with crushed limestone.
- Results from the two pilot bridge tests indicated that the test rig performed as designed, with minimal frame deflection (thereby giving effectively plane strain conditions).
- Inclusion of large windows along one side of the test chamber permitted the acquisition of good particle image velocimetry (PIV) data which is being used to help better understand the nature of the soil-arch interaction. The use of greased latex to reduce the wall friction did not prevent the acquisition of good quality image data.
- Both pilot bridges failed in hinged mechanisms, although some abutment movement was recorded in both cases. This was particularly pronounced in the case of the clay filled bridge.

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An analysis of simplified cable stayed bridge with FRP components

J. Park & C.H. Chung
Department of Civil & Environmental Eng., Dankook University, Seoul, Korea

I.C. An
School of Civil, Urban & Geosystem Eng., Seoul National University, Seoul, Korea

ABSTRACT: As an emerging material in construction fields, FRP(fiber reinforced polymer) has been used for its light weight and high strength. One of the major parts of its usage is the bridge constructions in civil engineering. There is already a pedestrian bridge (Aberfeldy, Scotland 1992) that all parts of the bridge including cables are made by FRP components. Since then, many FRP composite decks were developed and installed in US, Europe and other countries. A Korean company also developed a deck system (Delta tech) recently.

The 2nd Jindo-Bridge is a cable-stayed bridge that has 484 m span, and two pylons. The deck of the bridge was made of steel box girder and design load is DB-24, the 1st class bridge by Korea Standard Code. In this study, we set the 2nd Jindo-Bridge be a prototype, and tried to figure out the effect of replacing the steel components by FRP components by the simplified 3-D FEM analysis. The results show that low elasticity modulus of FRP increases the deflections of bridges, though it reduces total weight significantly. The natural frequency of the bridge can be controlled by the thickness of the FRP cables.

1 INTRODUCTION

2nd Jindo-Bridge is the twin bridge of the 1st Jindo-Bridge which connects Jindo-Island and Hae-Nam(land) in Korea. The prototype is modeled by the 2nd Jindo-Bridge data, and it is assumed to consist of three main parts: Pylon, Cable, and the Girder. Those three parts are simplified maintaining their major properties. The main purpose in this study is to figure out the effect of replacing components by FRP ones.

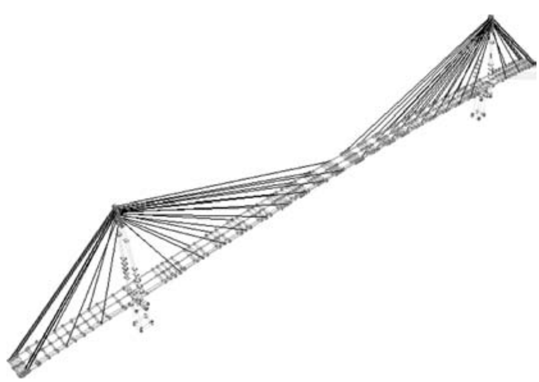

Figure 1. 3-D Bridge Model.

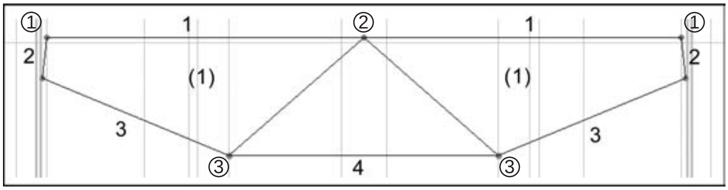

Figure 2. Box Girder Model.

The pylon is simplified to be a steel solid which has rectangular section. The model's moment of inertia (or a second moment) is equal to that of the 2nd Jindo-Bridge to see the girder's behavior according to the variation of deck and cables. The 2nd Jindo-Bridge has four kinds of cables which have different section areas. Therefore, the model also has four kinds of cables. For the girder's case, the original steel box girder is replaced by a composite GFRP deck and steel girder or by a complete GFRP deck and girder.

For the analysis, the section of the box girder deck is divided by four parts (Figure 2). Then parts of the steel girders or all of the steel girders are replaced by FRP. Basically, the section area and the moment of inertia are equal to those of 2nd Jindo-Bridge. The girder is modeled as shell components.

In this study, we made three cases for the box girder: steel, GFRP deck, and GFRP all and modified cable section area accordingly. Then we replaced the cable by CFRP components. Because GFRP has small modulus of elasticity, excessive vertical displacement was calculated.

The strength of FRP and steel are in the close range, and the maximum moment and shear force of the box girders after the analysis are not very different such that the design process of steel box girder can be applied to FRP components in the cable-stayed bridge similarly. Due to the light weight of the FRP, the total bridge design load is lowered, and the natural frequency of the bridge can be controlled by the FRP to steel ratio and types of cables.

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Regressive model for the partial-interactive ultimate strength of steel-concrete composite deck

Y.-J. Jeong
Korea Institute of Construction Technology, Goyang, Gyeonggi, Republic of Korea

S.-H. Kim
Yonsei University, Seoul, Republic of Korea

H.-Y. Kim & H.-B. Koo
Korea Institute of Construction Technology, Goyang, Gyeonggi, Republic of Korea

ABSTRACT: This paper deals with a regression model to predict the partial-interactive ultimate strength of steel-concrete composite decks. A series of nonlinear partial-interaction analyses with various degrees of interaction and shear span ratios were conducted to formulate the partial-interactive problem. Through the regression analysis of these data, a regression model for the partial-interactive ultimate strength was proposed. The proposed model was verified by a test program. It is clear that a regression model based on a partial-interaction analysis is a powerful tool to predict the partial-interactive structural behavior of composite members.

1 INTRODUCTION

According to the degree of interaction, the steel-concrete composite member behaves in three ways; full-, partial-, and no-interaction. Generally, the steel-concrete composite members show partial-interaction due to the deformation of shear connectors and slip at the interface under the applied loads. The degree of interaction between the two materials affects the shear flow and strain distribution of the members, and finally it has an influence on the structural performance. Therefore, the assumption of full-interaction may cause an over-estimation of the structural performance and assuming no-interaction may result in an under-estimation of that.

The objective of this study is to develop a regression model for the partial-interactive ultimate strength of steel-concrete composite deck, as shown in Figure 1. The partial-interactive behaviors of steel-concrete composite decks under flexural loading were examined by experimental and analytical studies. Finally, in order to easily predict the partial-interactive behavior, regression model for the partial-interactive ultimate strength was drawn based on the results of the partial-interaction analysis and this model was verified by test results.

2 TEST AND ANALYSIS

To formulate a correlation between the partial-interactive behavior and the degree of interaction with different shear span ratios, an experimental and analytical study was conducted. Analytical studies were used to formulate a regression model and experimental studies were used to verify the regression model. Nine flexural tests, as shown in Figure 2, were carried out, three span length groups with three different degrees of interaction for each group, and twenty-five nonlinear partial-interaction analyses were performed, five span length groups with five different degrees of interaction for each group.

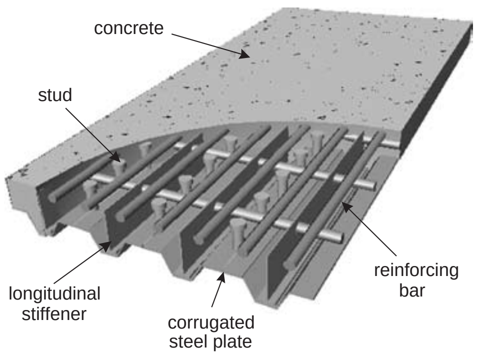

Figure 1. Steel-concrete composite deck.

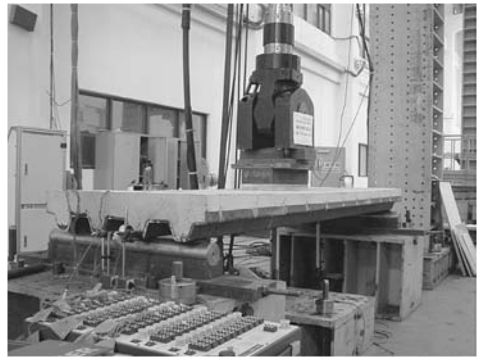

Figure 2. Static flexural test set-up.

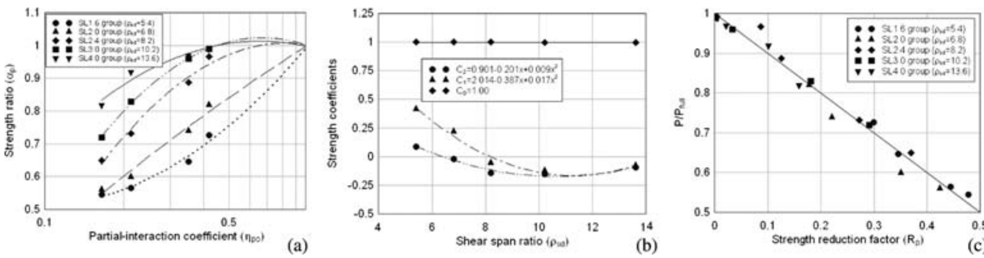

Figure 3. (a) & (b) Regression analysis; and (c) correlation (R = 0.993).

To have various degrees of interaction between the steel and the concrete, the spacings of the studs considered for each span length group were 150, 250, and 450 mm. In analytical study, the interaction between the steel and the concrete was idealized by using interface element and properties of that were determined from the results of push-out tests.

3 REGRESSION MODEL FOR PARTIAL-INTERACTION

The partial-interactive strength model was assumed to be a function of the partial-interaction coefficient (η_{po}) and shear span ratio (ρ_{sd}). To derive expressions for the partial-interactive structural behavior and partial-interaction coefficients in accordance with the shear span ratio, a regression analysis (Figure 3(a) & (b)) was carried out using the results of a nonlinear partial-interaction analysis. Finally, the partial-interactive ultimate strength can be expressed as follows

$$P = \alpha_p \cdot P_{full} = (1.0 - R_p) \cdot P_{full}, \text{ in which } R_p = -(C_1 \ln \eta_{po} + C_2 \ln \eta_{po}^2)$$

The regression model was compared with the results of partial-interaction analysis and flexural test, and it was found to be correlated well, as shown in Figure 3(c).

4 CONCLUSION

The results obtained by the regression model proposed in this study are well correlated with both the results of the partial-interaction analysis and flexural test. However, the m-k method has a limitation in that a full-size test is always required and the results are only valid for the specific case that is tested. On the other hand, the proposed regression model requires only a simple push-out test to predict the partial-interactive structural behavior. This may be attributed to the potential advantage of the proposed method over the conventional m-k method.

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Flexural behavior of external prestressed H-beam

K.S. Kim & S.-K. Park
Department of Civil Engineering, University of Sunkyunkwan, Suwon, South Korea

K.B. Han
Department of Civil Engineering, University of Nebraska, Lincoln, USA

D.S. Yang
Department of Civil Engineering, University of Sunkyunkwan,
Suwon, South Korea

ABSTRACT: Recently, prestressed H-beam bridges with external unbonded tendons are increasingly built. The mechanical behavior of prestressed steel H-beams is different from that of normal bonded PSC beams in a point of the slip of tendons at deviators and the change of tendon eccentricity that occurs, when service load are applied in external unbonded steel H-beams. The concept of prestressing steel structures has been widely considered, in spite of long and successful history of prestressing concrete members. For this experiment, five different types of test pieces were applied. To evaluate the effectiveness of the reinforcement in case of external load, the ordinary H-beam was set as the standard test piece and the number of steel bars and stress level or eccentricity are set as the variable to study the dynamic characteristics through the flexural test of the H-beam.

$300 \times 300 \times 10 \times 15$ mm H-beams are used as the test pieces and the total length is limited to 430 cm with the ground length of 400 cm. Also, in consideration of the potential buckling with the loading, they are reinforced with a vertical stiffener, and a steel bar with 25 mm diameter is used as the tendon. The overall illustration of the test pieces are as shown in Figure 5.

In this test, the externally stressed test piece with a 98.07 kN weight showed approx. 18 ~ 26% increasement of yield load and approx. 14 ~ 21% of limit load than the standard test piece. Also, when a 196.13kN weight is used for the external stress, the test piece displayed approx. 33 ~ 40% increasement of yield load and approx. 28 ~ 37% of limit load than the standard test piece. Therefore, by applying external stress to the existing bridge erection, the ground distance can be lengthened or the section can be reduced for great advantages to workability and economical efficiency.

The deflection of the column decreased by approx. 14 ~ 28% in the limit load than the standard test piece, but increased by approx. 14 ~ 26% in the yield load. Also, the increasement of the load was bigger than or similar to the decrease of deflection in yield load.

For the 482.49 kN weight, which is the yield load of the standard test piece, the deflection of the external reinforcement displayed a decrease, and the externally stressed test pieces performed decreased deflection than the standard test piece for all tested loads ($0.4f_y$) to reveal better usability. For the reinforced test pieces, the weight was loaded until the load started to decrease.

The externally stressed test pieces with 98kN weights displayed approx. 18 ~ 24% and 14 ~ 18% increasement for the yielding and limit loads, in respect, than the standard test piece. Also, when the weights were altered to 196 kN, the yielding load increased by approx. 33 ~ 36% and the limit load by approx. 28 ~ 34%, compared to the standard test piece.

In deflection, the limit load decreased by approx. 14 ~ 28% than the standard test piece. In tested loads, the externally stressed test piece displayed less deformation than the standard test piece to prove bettered usability.

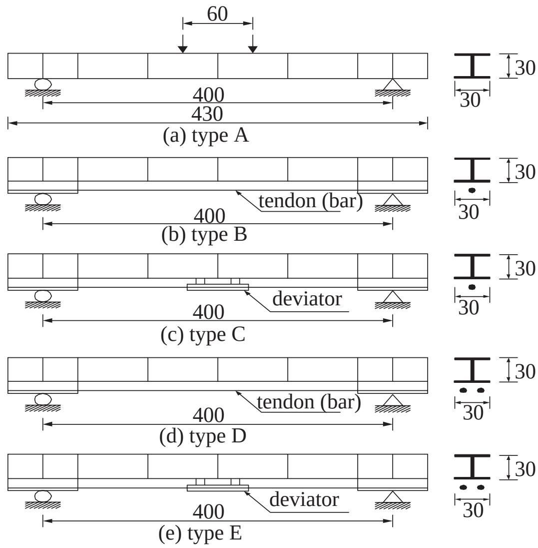

Figure 5. Illustration of the test pieces (unit:cm).

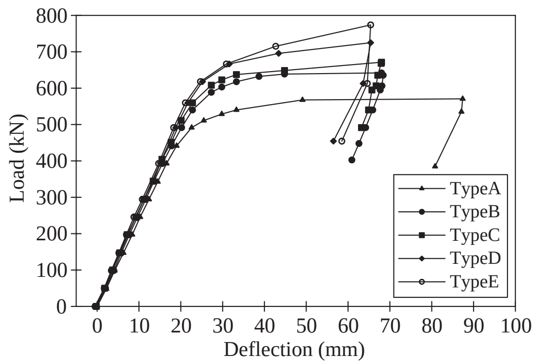

Figure 9. Load-deflection curve (LVDT 1, 3).

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In-plane buckling strength and design of parabolic arch ribs in uniform compression

J. Moon, S. Kim & H. Lee
Korea University, Seoul, South Korea

K. Yoon
Sunmoon University, Asan-si, Chungnam, South Korea

ABSTRACT: The purpose of this paper was to investigate the nonlinear elastic and inelastic behaviors of fixed parabolic arch ribs, due to the lack of study on fixed parabolic arch ribs. The in-plane nonlinear elastic and inelastic buckling behaviors and the strength of fixed parabolic arch ribs, when subjected to a vertically distributed load, were investigated using a finite element analysis. The FE models used in this study were verified using the results obtained by other researchers. The verification showed that the FE model used for the analysis was able to depict, with a good degree of accuracy, the in-plane behaviors of parabolic arch ribs. 75 FE models were used to investigate the nonlinear elastic behavior of a fixed parabolic arch subjected to a vertically distributed load. From the nonlinear elastic analysis, the buckling mode of arch ribs was observed to be dependent on the rise-span-ratio and S/r ratio of the arch ribs. 107 FE models, including residual stresses and initial imperfections, were analyzed to observe the inelastic behavior of fixed parabolic arch ribs. The effects of residual stresses and initial imperfections were investigated. Finally, it was found that the buckling curve for a straight column can be apply to fixed parabolic arch ribs subjected to a vertically distributed load, with the exception of shallow arch ribs.

1 FINITE ELEMENT ANALYSIS RESULTS

1.1 *Nonlinear elastic analysis results*

The slenderness ratio, which prevents the symmetrical snap-through buckling mode (Limit slenderness ratio), is influenced by the ratio, h/L. From the FE results, if the h/L ratios are equal to 0.05 and 0.15, the limit slenderness ratios are 250 and 85, respectively. Modified slenderness ratio ($\bar{S}/r$) is suggested in this paper to determine the limit slenderness ratio, regardless of the h/L ratio. The modified slenderness ratio ($\bar{S}/r$) is defined by Eq. (1).

$$\bar{S}/r = (S/r) \times (h/L) \tag{1}$$

From the nonlinear elastic analysis results, it can be summarized that the buckling mode of a fixed parabolic arch depends on the limit modified slenderness ratio, and the value of the limit $\bar{S}/r$ equals 12.8 for fixed parabolic arch ribs. If $\bar{S}/r > 12.8$, an antisymmetrical bifurcation mode occurs, and the classical buckling theory can be directly applied to predict the buckling load of the fixed parabolic arch. However, when $\bar{S}/r < 12.8$, fixed parabolic arch ribs may buckle due to a symmetrical snap-through mode.

1.2 *Inelastic analysis results*

The FEA results shown in Fig. 2 include the effects of residual stresses and initial imperfections. The distribution shape of the FEA results can be seen to agree well with the column buckling curve.

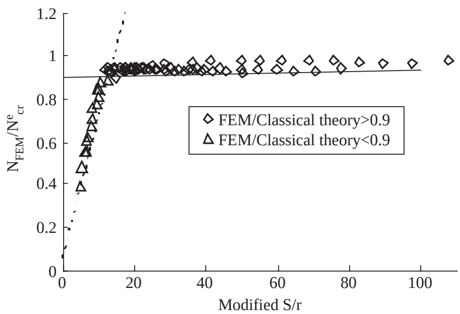

Figure 1. Analysis results (all data).

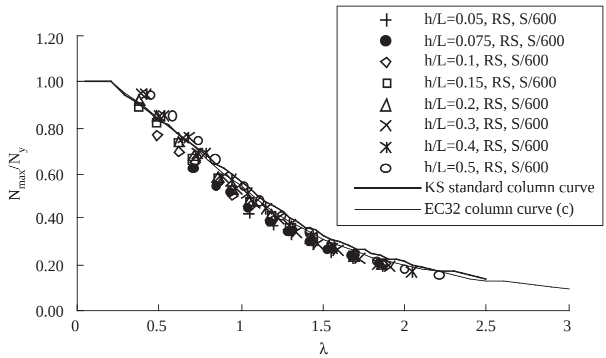

Figure 2. FEA results (with RS & initial imperfection, modified $S/r > 12.8$).

However, some of the FEA results were located below the column buckling curve, but within a 10% error. The maximum allowable magnitude of the initial imperfections in the EC3 column curve (c) is $L/400$. If arch ribs have a $h/L = 0.5$, $L/400$ is equal to $S/600(S \approx 1.5L$, when $h/L = 0.5)$.

From the nonlinear elastic and inelastic results, it can be summarized that the KS standard or EC3 column curves can accurately predict the buckling strength of fixed parabolic arch ribs under uniform compression if the modified slenderness ratio ($\bar{S}/r$) is lager than 12.8 and maximum initial imperfection (e) is smaller than $s/600$.

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Field tests of prefabricated, composite girders

M. Łagoda & P. Olaszek
Road and Bridge Research Institute, Warsaw, Poland

ABSTRACT: Prefabricated composite constructions are currently taking a greater role in bridge building for economic and production technology reasons. While using VFT® girders technology, the thin concrete slab is made before the final assembling on supports. This slab is collaborating with steel girders and serving as shuttering for concrete bridge deck, constructed after final girders assembling on supports. The paper presents the results of field tests on two bridge structures with VFT® girders. At the presented field tests the attention was concentrated on the analysis of girders behavior. During static tests and stresses measurements the specific construction behavior was observed. This specific behavior was manifested by "wavy motion" of stresses signal during constant load level. This phenomenon occurred at different intensity for two tested structures. The stresses redistribution was caused by different factors. The main and the most probable causes of the stresses redistribution are the main subject of investigations.

1 TECHNICAL DESCRIPTION

Both engineering objects are the first VFT®-s in Poland. Both of them were also constructed at locations, where old viaducts over railroads had existed. Designers encountered certain limitations – the road's grade line at the top and railroad outline from the bottom. Thus, the spans' number and their lengths, as well as superstructure's height were already given. Designed VFT® girders were lower than those used in Germany for analogical spans' lengths.

Object No1 was designed as seven spans, continuous, of the following lengths: 18.17 + 18.49 + 18.50 + 24.56 + 18.44 + 18.44 + 17.92 [m]. At its cross-section the structure consists of 12 prefabricated VFT® girders. Object No 2 was designed as four span, continuous, of the following lengths: 19.00 + 26.00 + 22.00 + 19.00 [m]. In its cross-section the superstructure consists of 8 prefabricated VFT® girders.

2 TESTS' SCOPE

Tests described hereunder were conducted during load testing for both structures. Above research was the part of the acceptance procedure prior to opening for the traffic. Testing programme was elaborated taking into account the construction's specific features as well as limitations connected with terrain conditions and financial possibilities. For detailed tests, on both structures, one span was chosen. There was no possibility to stop railway traffic while tests, so not the longest spans were chosen, where railroad is positioned, but neighboring spans.

To analyze the structure's behavior in cross-section, the mid-span deflections were measured for all girders, using inductive transducers connected with computer registration system, enabling the deflection's measurement both under the static and dynamic loads. At remaining spans deflections were measured only for two chosen girders, taking into account the strain degree and measurements' possibilities. To analyze steel girders behavior and their cooperation with reinforced concrete deck for chosen, most heavily loaded girder strains (stresses) were measured for steel girder's top and bottom flanges. Above was measured in two sections: at midspan and near support zone. For measurement, electric resistance wire strain gauges were used connected with computer registration system, enabling the measurement both for the static and dynamic loads.

3 TEST RESULTS AND STRUCTURE'S BEHAVIOR ANALYSIS

Static and dynamic test loads conclusions has allowed for discovering the real structure's behavior. Under static test loads both objects behaved as elastic structures and introduced very small durable displacements and deformations. Measured displacement and deformation values were smaller than those calculated ones, what confirms the fact that the structure's rigidity is greater than that assumed in calculated model. Differences were so significant that we can conclude that mentioned excess values were influenced not only by non-structural elements (such as pavement, safety barriers, sidewalk concrete, handrail beams, etc.) The basic reason of such increased rigidity one should search in increased rigidity of material used, especially the concrete deck.

VFT® structures' specificity is such that in the first phase the superstructure behaves not only as a steel structure but as composite one, i.e. steel girder plus thin concrete slab. In such arrangement, with the thin concrete slab, the neutral axis is always situated inside the steel part. After span's assembly, prefabricates are mainly loaded with a fresh concrete, what results with compressive stresses inside concrete slab. In both engineering objects, at the second stage, the superstructure behaves as a composite one and the neutral axis is located within the concrete slab. Stresses caused by live loads, in second phase, are tensile stresses, occurring in the bottom part of the concrete slab and they are reaching excessive values. The level of compressive stress in concrete slab, "remembered" from the first phase, should be enough to level tensile stress introduced during the second phase.

During stresses measurement, specific behavior of the structure was observed. It occurred as "signal waving", taking place without any changes in test loading level. The specific behavior occurred for both examined structures but at different degree. It probably was caused by cracks development inside the concrete slab. At certain moment cracks are developing in concrete at support zones (superstructures are of continuous type) and simultaneously at midspans. While unloading an inverse phenomenon is taking place. It means that concrete cracks are closing and the cross-section "reclaims" its primal stiffness. A hypothesis presented here calls for the scientific confirmation, for example analyzing it with "tension stiffness" method.

4 CONCLUSIONS

Poland is a country, which calls for definite improvement of its road network. The best technological and economical solutions are needed. Great hopes are connected with VFT® girders using. Many objects, where superstructures are dimensionally low, call for renovation and rebuilding. Objects described in this paper are typical examples of above. So it calls for detailed acceptance tests and procedures for them.

When steel girders are low, tensile stresses occur in bottom parts of the deck slab, what was confirmed by tests described in this paper. When these tensile stresses are exceeding compressive stresses obtained at first phase, the superstructure's durability is lowered as well as its failure-free lifetime. Above can also cause the necessity for suitable anti-crack protection of the slab's bottom. To solve the problem completely careful studies and researches are needed.

Anyhow, despite many doubts, positive results in tests described above encourage to wider responsible and well thought out and adjusted to situation application of VFT® girders.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Robustness of highway overpasses

H. Stempfle & T. Vogel
Institute of Structural Engineering, Swiss Federal Institute of Technology ETH, Zurich, Switzerland

ABSTRACT: Bridges are planned for a long service life with changing demands that can hardly be fully anticipated. The traffic is still growing, the live loads increase, de-icing salt is used more and more since the seventies of the last century and additional actions like failure of a column, terrorism, settlements etc. arise that are normally not considered when the structure was designed. Due to the amount of traffic it is not possible anymore to reduce the number of traffic lanes or even to close the whole bridge during repairs. In fact, during repair periods the full traffic is diverted on side-strips that are not designed for that purpose. Although all these facts lead to higher risks, the society claims to limit risks more than ever. A concept to overcome this situation is to design new bridges for robustness and to judge the robustness of existing ones.

If different authors discuss about robustness of structures, they use a lot of terms (robustness, redundancy, vulnerability, system, etc.) in different contexts (serviceability, structural safety etc.). In this paper the robustness of structures is considered in the context of structural safety. The robustness of the structure itself is investigated and the correlation between the environment and the structure is neglected.

An investigation of the robustness of a structure always concerns the whole system and not only parts of the structure. To evaluate the robustness of a structure it is necessary to define both the term *robustness* and the system itself. We have to describe the bridge as a system which consists of a complex combination of different subsystems. The whole system is divided in subsystems to investigate the interactions between the different subsystems. The robustness of a structure depends on the interactions between the subsystems. It has to be evaluated how they depend on each other and what are the different influences of each subsystem concerning the whole system. Highway overpasses with V-columns are treated as an example because they are widely common in Switzerland.

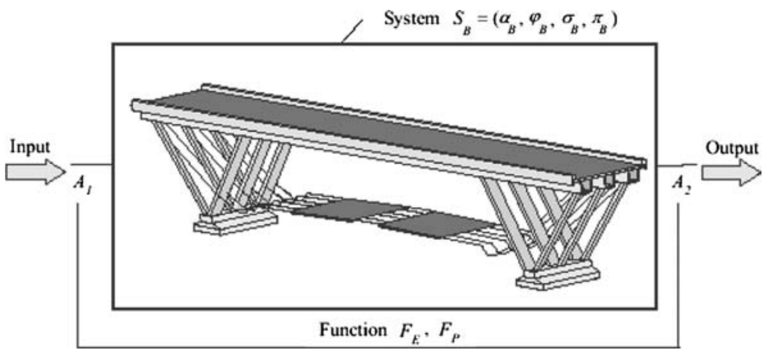

Figure 1. Functional concept of a system of a highway overpass with V-columns.

In the first part of the paper a proposal for a definition of the term robustness is shown. Based on *Systems Theory of Engineering* by G. Ropohl [1] a definition of a bridge system is described in the second part. Finally a concept to evaluate the robustness of a bridge is shown.

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Neural network modelling of perfobond shear connector resistance

J.C. Vianna
Civil Engineering Department, PUC-RIO – Pontifical Catholic University of Rio de Janeiro, Brazil

S.A.L. de Andrade
Civil Engineering Department, PUC-RIO – Pontifical Catholic University of Rio de Janeiro Structural Engineering Department, UERJ – State University of Rio de Janeiro, Brazil

P.C.G. da S. Vellasco
Structural Engineering Department, UERJ – State University of Rio de Janeiro, Brazil

M.M.B.R. Vellasco
Electrical Engineering Department, PUC-RIO – Pontifical Catholic University of Rio de Janeiro Systems Engineering Department, UERJ – State University of Rio de Janeiro, Brazil

1 INTRODUCTION

Shear connectors are used in composite structures to enable the combined action of the concrete slab and the steel profile. The Perfobond shear connector, main focus of this paper, was developed by Leonhardt to improve the fatigue strength of a Venezuelan composite bridge. The Perfobond shear connector is made from a rectangular steel plate, with holes, welded to the steel beam top flange. Subsequently the perfobond holes are filled with concrete forming dowels that improve the connector shear strength and inhibits the vertical split at the concrete/steel interface.

Various push-out tests have been made to evaluate the perfobond structural response. The test layout comprises the steel profile supported by two adjacent concrete slabs. The composite action is guaranteed by the use of shear connectors welded to both steel beam flanges. The structure is subjected to a vertical load that introduces shear stresses in both sides of the concrete slab versus beam interfaces. Several parameters significantly affect the test results like: the concrete characteristic compressive strength, the number and diameter of the perfobond holes, the use of transverse reinforcement bars, the perfobond steel plate dimensions, etc.

The main objective of the present investigation is to determine the parameters that significantly influence the composite steel concrete action through a detailed investigation of the perfobond shear connector structural response. However, due to the large number of parameters involved the use of neural networks, to forecast the perfobond shear capacity, calibrated against the existing experimental data, seemed to be one of the most feasible alternatives.

2 PERFOBOND SHEAR CONNECTOR RESISTANCE

Some early proposal of perfobond shear strength, Eq. (1), presented by Oguejiofor & Hosain (1994), considers the contribution of three fundamental parameters: the concrete slab in shear, the transverse reinforcement, and the concrete dowels formed in the perfobond holes.

$$q_u = 0.590.\,A_c.\sqrt{f_c^{'}} + 1.233.\,A_{tr}.f_y + 2.871.\,n.d^2.\sqrt{f_c^{'}} \tag{1}$$

– q_u is the nominal perfobond shear connector resistance (N);
– A_c is the concrete shear area per connector = slab longitudinal area minus connector area (mm^2);

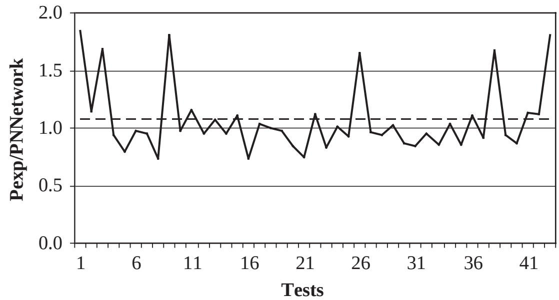

Figure 1. Experimental/neural network results errors.

– A_{tr} is the transverse reinforcement area present in the perfobond shear connector holes (mm^2);
– d is the perfobond hole diameter (mm);
– n is the number of holes present in the perfobond shear connector;
– $f'_{c,}$ is the characteristic concrete compressive strength (MPa);
– f_y is the ultimate steel strength in tension (MPa).

The main objective of this paper is to use the neural networks to forecast the perfobond shear capacity. The network input data utilized current experimental evidence present in the literature.

3 SOLUTION DESCRIPTION

A Bayesian training algorithm (Bayesian networks) was adopted since this hybrid model has the capability of improving convergence and overfitting of networks with a small input dataset. Basically the Bayesian networks use the Bayesian inference mechanism to evaluate the neural model parameters (weights). The MatLab software was used for training and testing the adopted neural networks. Noisy input data were introduced to increase the dataset and improve the neural network performance. The original performance was compromised due to the small experimental dataset available. However a maximum number of noisy data was established not to be well beyond the actual (experimental) data and disguise the results. The neural networks were trained varying: The neural network input and output data were normalised by the maximum/minimum range of values $\{x''_i = (x_i - \min)/(\mathrm{Max} - \min)\}$.

The network did not produce trustable results with the original 41 experimental data where the data used in the test phase is illustrated. When the various trained neural network generalisation errors where compared the best solution was reached by the network using 123 inputs, (41 actual experimental and 82 noisy data) with the following topological characteristics:

−9 inputs; −1 hidden layer with 5 processors; −1 output.

The mean absolute error of the experimental to neural networks to was 1.07 significantly better than the analytical to experimental error of 1.27, Figure 1.

4 CONCLUSIONS

With the present paper some of the advantages of the Bayesian learning algorithms were highlighted: minimizing overfitting problems in training; achieving a good generalisation; enabling an efficient use of the training data, not requiring a separated set of values for hiper-parameters cross-validation; and finally the use of the weight decay technique enabling the neural network to produce smooth values for the approximated output functions.

Using the original 41 experimental data results the neural network did not produce a good generalisation due to the small data number. This motivated the inclusion of noisy data to improve the network generalisation capability. The neural network errors proved to be small (compared to the usual civil engineering safety coefficients) confirming the possibility of using this method to create new data. This new data, incorporated in the original dataset, could enable a complete parametric analysis to be performed that could lead to the development of a more accurate design formula and the definition of a single consistent resistance factor.

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Cruz, Frangopol & Neves (eds)*

Nonlinear analysis of prestressed concrete structures using unbonded tendon model

J.-G. Park & H.-M. Shin
Department of Civil and Environmental Engineering, Sungkyunkwan University, Jangan-gu, Suwon, Kyonggi-do, Korea

T.-H. Kim
Civil Engineering Research Team, Daewoo Institute of Construction Technology, Jangan-gu, Suwon, Kyonggi-do, Korea

J.-H. Choi
Department of Civil Engineering Hankyong National University, Seokjeong-dong Ansung, Kyonggi-do, Korea

1 GENERAL INTRODUCTION

In this paper, a numerical model for tendons is proposed based on the finite-element method, which can represent the interaction between the tendon and concrete. This model can be used for prestressing transfer processes for one or multitendons efficiently and analyze the unbonded post-tensioned concrete structure succinctly. Another relevant model for representing the friction and bond effect at the interface of tendon and the concrete is also proposed at the same time. By using these models, a numerical procedure for material nonlinear analysis of prestressed concrete structures, including the time-dependent effects due to load history, relaxation of prestress, and creep and shrinkage of concrete is presented. This procedure can predict the response of pretensioned and bonded or unbonded posttensioned beam-column or plane concrete structures such as elastic and plastic deformation, cracking, and damage patterns throughout their service-load history. Finally, a numerical example is presented to verify the validity and applicability of these models and the numerical procedure.

2 NONLINEAR FINITE ANALYSIS PROGRAM RCAHEST

RCAHEST is a nonlinear finite element analysis program for analyzing reinforced concrete structures. The program has been developed by Kim and Shin (2001), at the Department of Civil and Environmental Engineering, Sungkyunkwan University.

3 NUMERICAL EXAMPLES

The comparison of load-deflection curves for the A-1 beams(Tao and Du 1985) and slab (Ritz et al. 1975) are plotted in Figure 1 and Figure 2. The program RCAHEST predicted the load-deflection behavior of the unbonded beams with good agreement at various states of loads from the cracking load up to and beyond the ultimate load.

Chiu et al. (1996) conducted long-term experimental studies on prestressed concrete beams. The experimental program included the creep ad shrinkage tests on concrete material used and the flexural tests of prestressed concrete beams. Figures 2–3 show a comparison between the experimental and analytical behavior for this test. The analytical results show reasonable correspondence with the experimental results.

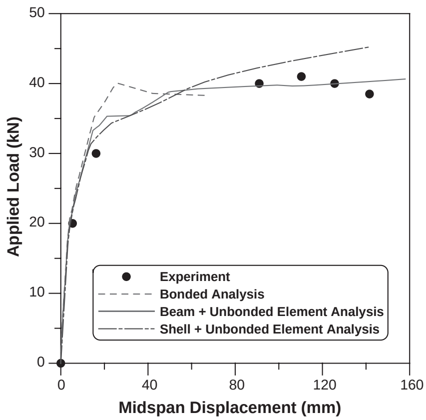

Figure 1. Load–displacement curve of beam.

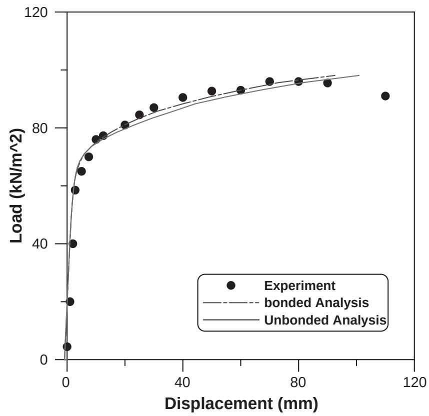

Figure 2. Load–displacement curve of slab.

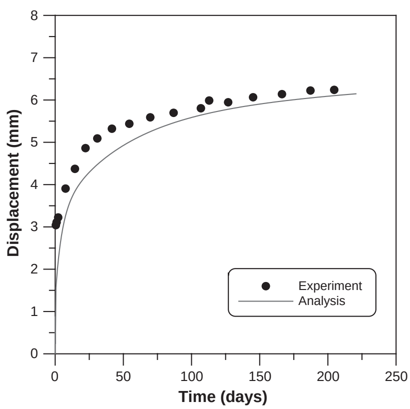

Figure 3. Displacement–Time Curve for SB13C.

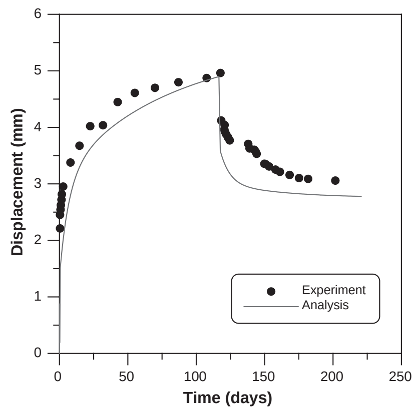

Figure 4. Displacement–Time Curve for SB17B.

4 CONCLUSION

The results obtained with the numerical model in the case of three examples of application are presented and compared with experimental and analytical results of Instantaneous or long-term analysis of prestressed concrete structures. Through those examples the capability of the numerical model to realistically represent the nonlinear behavior of prestressed concrete structures throughout their initial, cracked and ultimate ranges is shown.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Ultimate strength of compression members undergoing buckling interaction

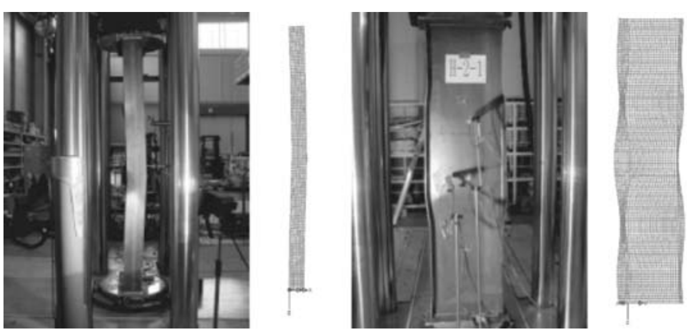
Y.B. Kwon & N.G. Kim
Yeungnam University, Gyongsan, Korea

ABSTRACT: This paper describes a series of compression tests performed on welded H-section and channel section columns fabricated from mild steel plate of thickness of 0.6 mm with nominal yield stress 240 MPa. The ultimate strength and performance of the compression members undergoing the nonlinear interaction between local and overall buckling were investigated experimentally and theoretically. The compression tests indicated that the interaction between local and overall buckling had a significant negative effect on the ultimate strength of the thin-walled welded steel section columns. The direct strength method, which was newly developed and adopted as an alternative to the effective width method for the design of cold-formed steel sections recently by NAS(AISI, 2004), was calibrated for the application to the welded steel sections by the test results. The paper proves that the direct strength method can predict properly the ultimate strength of the welded section columns when the local buckling and flexural buckling will occur simultaneously or nearly simultaneously.

1 TEST RESULTS

When the compressive load was increased near the local buckling load expected, the local buckling occurred first at the web of the section and then propagated to the flanges of H-sections and C-sections. Then the significant post-local-buckling strength reserve was observed before the flexural buckling about the minor axis for the H-section and about the unsymmetrical axis for the C-section occurred, respectively. The buckling mode interaction between local and flexural buckling was observed as shown in Fig. 1 before the maximum load was reached.

The experimental local buckling stresses and the ultimate stresses were summarized in Table 1 and compared with the numerical analysis results obtained by the THIN-WALL (1995) and the LUSAS (ver13.4), respectively. The comparison shows that test and numerical results are agreed well except C-1 sections which displayed the premature failure due to local collapse at the loaded column end.



(a) side view (b) front view

Figure 1. Buckled shape of H-section.

Table 1. Ultimate stress and local buckling stress of specimens.

	Ultimate stress (F_{max})			Local buckling stress (F_{cr})		
Specimen	Test (MPa)	Analysis (MPa)	Test/analysis	Test (MPa)	Analysis (MPa)	Test/analysis
H-1-1	198.7	180.6	1.10	185.2	203.4	0.96
H-2-1	170.9	145.6	1.17	143.7	106.5	1.35
H-2-2	182.0	145.6	1.25	155.2	106.5	1.46
H-3-1	158.1	121.8	1.30	121.8	120.9	1.01
H-3-2	165.6	121.8	1.36	128.2	120.9	1.06
C-1-1	120.0	147.2	0.82	66.7	97.9	0.68
C-1-2	106.4	147.2	0.72	73.8	97.9	0.75
C-2-1	155.1	134.4	1.15	128.9	118.7	1.09
C-3-1	116.4	104.4	1.11	86.7	77.1	1.12

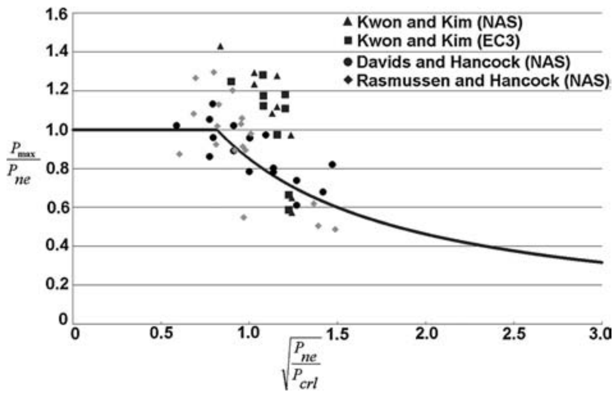


Figure 2. Comparison between DSM and Test Results.

2 PROPOSED DIRECT STRENGTH METHOD

Since the effect of interaction between local and overall buckling of welded sections on the strength for the compressive members may be less significant and the post-local-buckling strength reserve is comparatively smaller than the cold-formed steel section, the modified Winter formulas were proposed for the limiting load.

While the exponent 0.5 as used in Winter formulas is used, the coefficient 0.15 in Eq. (1b) is adopted rather than 0.22 as used in Winter formulas, which reflects a higher post-local-buckling strength reserve in the inelastic buckling range of material for the intermediate length column. The equations for the limiting stress P_{nl} considering the interaction between local and overall buckling for welded sections are given by

For $\lambda \leq 0.816$

$$P_{nl} = P_{ne} \tag{1a}$$

For $\lambda > 0.816$

$$P_{nl} = (1 - 0.15(\frac{P_{crl}}{P_{ne}})^{0.5})(\frac{P_{crl}}{P_{ne}})^{0.5} P_{ne} \tag{1b}$$

where, $\lambda = \sqrt{P_{ne}/P_{crl}}$, P_{crl} = elastic local buckling load and p_{ne} = column design strength.

The Direct Strength Method curves are compared with the tests and numerical results in Fig. 2, which are generalized by the design column design strength according to the NAS (AISI, 2001) and Eurocode 3 (1993), respectively. Since the column design strength is slightly different between NAS and EC3, the abscissa is different according to which specifications are used.

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Numerical analysis of welding considering phase transformation

Shirahata, H
Musashi Institute of Technology, Tokyo, Japan

1 INTRODUCTION

Welding is one of the most fundamental connections for steel structures. Initial residual imperfection such as stress and deflection is one of the most significant phenomena caused by the welding. Many studies on the residual stress for joint strength and fatigue behavior have been carried out.

Some numerical analyses of welding process have been done. Most of the numerical models were relatively simplified. However, welding process includes liquid-solid and phase transformations. A numerical program code of welding was developed in this study. This paper shows the formulation and an example of pad welding.

2 HEAT TRANSFER EQUATION

Heat transfer equation of solids is given as follows:

$$\rho c\dot{T} = \dot{q} + div\,(\,k\;grad\,T) - \rho\sum l_i\dot{\xi}_i \tag{1}$$

where ρ is a density, c is a specific heat, T is a temperature, q is an input heat, k is a heat transfer coefficient. Dot " ." represents time derivative. l is a coefficient related to latent heat by phase transformations. i ξ is a volumetric fraction of the phases. i is an index for phases of steel during the weld process, and takes pearlite, austenite, and liquid.

3 EQUATIONS OF PHASE TRANSFORMATION

Pearlite-austenite and austenite-liquid transformations occur in the heating process. In the cooling process, liquid-austenite and austenite-pearlite transformations occur. Pearlite-austenite transformations in the heating process, fraction of austenite was assumed to be in proportion to the temperature. For austenite-liquid and liquid-austenite transformations, the lever relation was applied. A fraction of pearlite P ξ during austenite-pearlite transformations in the cooling process is given by equation (2).

$$\xi_P = 1 - \exp\left\{\int_0^t f(T,\sigma_m)(t-\tau)^3\,d\tau\right\} \tag{2}$$

4 CONSTITUTIVE EQUATIONS

The total strain velocity ε & is obtained by equation (3).

$$\{\dot{\varepsilon}\} = \{\dot{\varepsilon}^e\} + \{\dot{\varepsilon}^v\} + \{\dot{\varepsilon}^m\} \tag{3}$$

where $\{\dot{\varepsilon}^e\}$, $\{\dot{\varepsilon}^v\}$, $\{\dot{\varepsilon}^w\}$ are elastic, visco-plastic, and phase transformation dependent strain velocity, respectively.

In the numerical analysis, equation (1) was solved first. After obtaining temperature, equations related to fraction of each phase such as equation (2) were calculated. Equations related to stress strain constitutive law were calculated.

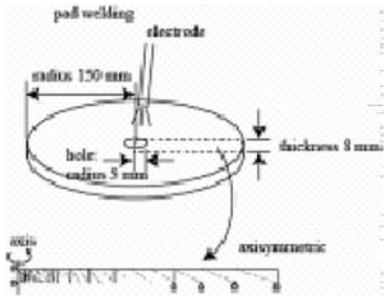

Figure 1. Numerical model of pad welding.

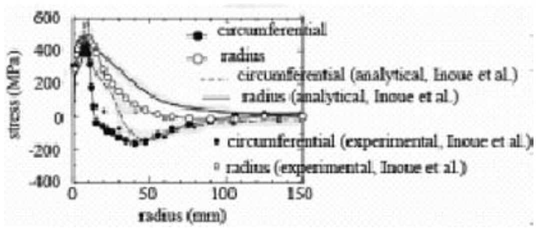

Figure 2. Residual stress distribution.

5 NUMERICAL ANALYSIS

A numerical analysis was carried out. Figure 1 shows the numerical model. The model consists of a disc of which radius was 150 mm and thickness was 8 mm. There was a hole at the center of 5 mm radius. Pad welding was done at the center. The electrode was put at the center for 20 seconds.

Figure 2 shows the residual stress distribution. Figure 2 also includes the experimental and numerical investigations done by Inoue et al. Relatively good agreement could be obtained.

6 CONCLUSION

The numerical code developed in this study showed a good agreement with the previous investigations. For future work, the author would expand the code in order to calculate residual stress and deflection of I-girder and complicated welded joints and low transformation temperature weld material and effect of fatigue life improvement.

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Design guidelines for sole plates in the elastomeric bearing system

C. Joh & Y.J. Kim
Korea Institute of Construction Technology, Goyang, Gyeonggi, Korea

ABSTRACT: Among various types of bearings, elastomeric bearings such as a steel reinforced elastomeric bearing (hereafter elastomeric bearing) and a elastomeric bearing pad are most popular because of their function and cost effectiveness. The elastomeric bearing system has relatively high compressive stiffness while it is flexible in shear to limit the transfer of the shear force detrimental to a pier. It distributes load evenly and absorbs vibrations. It is compact, easy to install, and durable. It requires little maintenance.

The elastomeric bearing system is often composed of an elastomeric bearing, a base plate, and a sole plate. The sole plate makes concentrated load from the superstructure dispersed allowable to the elastomeric bearing in steel plate or box girders. In concrete girders, however, the sole plate spreads the concentrated load from a bearing over the allowable area of the concrete bottom flange.

In spite of important roles of the sole plate in the elastomeric bearing system, the researches on the design of sole plates have been very limited. Most of the researches on the elastomeric bearing system have been focused on the design of an elastomeric bearing itself. As a result, even in AASHTO and Eurocode, the design of sole plates is based on the past experience and a rule of thumb. In practice, the thickness of the sole plate may vary from 13 mm to more than 125 mm.

In this paper, for the reasonable design guideline, the behavior of the sole plate in the steel reinforced elastomeric bearing system is investigated in detail using numerical models (Figure 1).

In the numerical models, to consider practical cases, the sole plate is assumed to be connected mechanically to the bottom flange and the elastomeric bearing is assumed to be vulcanized to the sole plate or the top plate. The superstructure under investigation is a steel plate girder. To simulate critical load condition based on AASHTO, the distributed load that gives 11 MPa (max. allowable vertical stress with shear deformation) to the elastomeric bearing is applied on the top flange. Also, maximum allowable shear deformation (50% of the height of the elastomeric bearing) is applied along the web. Mooney-Rivlin type hyperelastic material model is used for elastomer ($C_{10} = 0.38$, $C_{01} = 0.094$). For steel, young modulus of 210,000 MPa and yielding stress of 327 MPa are assumed. von-Mises yield criterion is used for material nonlinearity.

Three different elastomeric bearings shown in Table 1 are modeled, which represent small (NR1), medium (NR2), and large bearings (NR3). For each elastomeric bearing, the thickness of the bottom

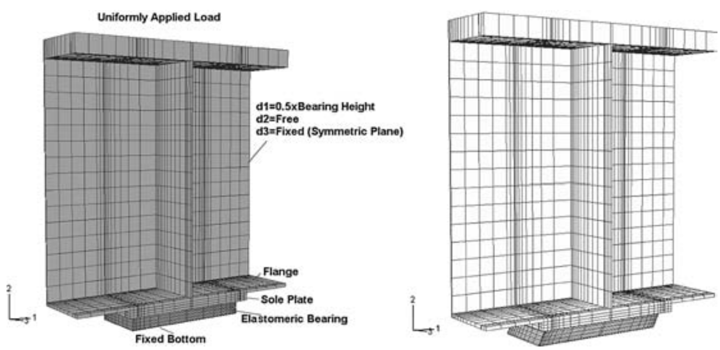
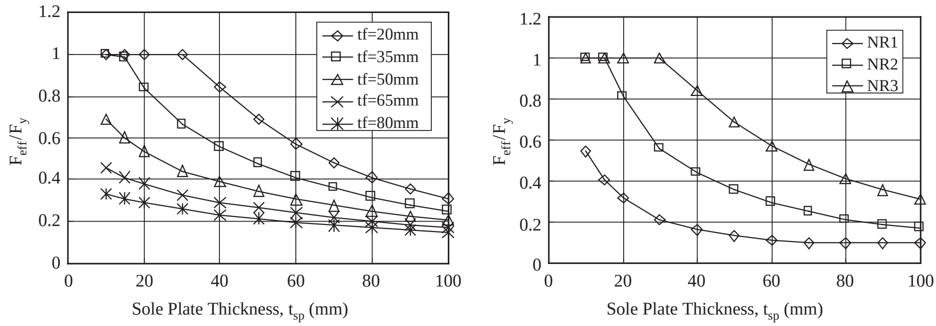


Figure 1. Idealized isolated model and deformed shape (NR3, $t_f = 20$ mm, $t_{sp} = 50$ mm).



(a) NR3 (b) t_f=20 mm

Figure 2. Effective stress (F_{eff}) in the sole plate.

Table 1. Required thickness of the sole plates from Equation 1 and FEM analysis.

t_f (mm)	NR1 (300 mm × 400 mm)		NR2 (500 mm × 600 mm)		NR3 (700 mm × 800 mm)	
	Required t_{sp} (mm)		Required t_{sp} (mm)		Required t_{sp} (mm)	
	Eq. (1)	FEM	Eq. (1)	FEM	Eq. (1)	FEM
20	7 (20)	7 (20)	36	35	105	52
35	3 (20)	0 (20)	15 (20)	10 (20)	43	35
50	2 (20)	0 (20)	8 (20)	0 (20)	24	12 (20)
65	1 (20)	0 (20)	6 (20)	0 (20)	16 (20)	0 (20)
80	1 (20)	0 (20)	4 (20)	0 (20)	12 (20)	0 (20)

flange is varied from 20 mm to 80 mm and the thickness of the sole plate is varied from 10 mm to 100 mm (Figure 1). In total, 135 different combinations of a bottom flange and a sole plate are analyzed. The height of the steel plate girder is assumed to be 1500 mm and the thickness of the web and the stiffener are assumed to be 14 mm, respectively. Upper flange thickness is assumed to be 150 mm to represent the stiffness of composite concrete slab.

Analysis results show that the increase in the thickness of the sole plate and the bottom flange (t_f) decreases the effective stresses (F_{eff}) in the sole plate (Fig. 2a). The results from other bearing models such as NR1 and NR2 show similar tendency. Figure 2b shows the effects of bearing area on the effective stress in the sole plate. The effective stress in the sole plate is proportional to the increase in bearing area. The results from other flange thickness also show similar tendency.

Using regression technique, this relation is derived and simplified as Equation 1 with the safety factor of 1.5 against the yielding of the sole plate.

$$t_{sp} = 1\times10^{-5}\frac{b_w^{\ 3.2}}{t_{bf}^{\ 1.6}} \geq 20mm \qquad (1)$$

where, t_{sp} and t_f are the thickness of the sole plate and the bottom flange, respectively. b_w is the width of the elastomeric bearing representing the effect of bearing area.

Table 1 shows comparison between Equation 1 and FEM results. The Equation 1 shows good agreement with the FEM results in the conservative side.

As a result, in this paper, for the elastomeric bearing system with the sole plate bonded to the bottom flange and the elastomeric bearing, Equation 1 is suggested as the design equation for the thickness of the sole plate.

The sole plate is a simple plate but may affect the performance of the elastomeric bearing system, as discussed above. In practice, however, the sole plate is not designed but chosen based on past experience and a rule of thumb. To guarantee the reliable behavior of the elastomeric bearing system, the sole plate should be design properly.

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Design and experimental analysis of a new shear connector for steel and concrete composite structures

G.S. Veríssimo & J.L.R. Paes
Federal University of Viçosa, Viçosa, Minas Gerais, Brazil

I. Valente & P.J.S. Cruz
University of Minho, Guimarães, Portugal

R.H. Fakury
Federal University of Minas Gerais, Belo Horizonte, Minas Gerais, Brazil

1 INTRODUCTION

This work presents the design of a new shear connector and the corresponding results obtained on push-out tests. This new shear connector consists on a steel rib with indented cut shape that provides resistance to longitudinal shear and prevents transversal separation between the concrete slab and the steel profile (uplift). Adding to this, the connector openings cut make easier the arrangement of transversal reinforcement bars. The installation of the connectors is simple and requires only common welding procedure. Due to its load capacity, the indented connector is able to replace a group of stud bolts. Its structural behaviour is analyzed and compared with other existing connectors, like the stud bolt and the Perfobond. The influence of different geometrical and mechanical aspects on the ultimate load capacity and ductility is assessed. The performed studies indicate that the indented connector presents a good mechanical performance, associated with constructive and economical advantages.

2 STEEL-CONCRETE CONNECTION

The advantages of composite systems are well known and result from the fact that, in these systems, steel and concrete elements can work submitted to compressive and tensile stresses, respectively, situation in which their best behaviour is accomplished. The connection between steel and concrete provides the composite behaviour. The use of shear connectors enhances the development of longitudinal shear forces at the steel-concrete interface.

Several shear connectors have been recently proposed and used in composite structures. However, most of them present important restrictions with respect to fabrication, installation and structural behaviour. The most well-known and used connector is the headed stud, developed during the 40's by Nelson Stud Welding Company (Figure 1a). In the late 80's, the German company Leonhardt, Andrå and Partners developed a new shear connector, called Perfobond. Perfobond consists on a plane perforated steel plate that is welded to the steel beam upper flange (Figure 1b). The main disadvantage of a Perfobond connector is the difficulty to position the slab lower reinforcement, when the steel bars have to cross the connector openings.

This work summarizes the design and tests results for the proposed shear connector, called CR (Figure 1c). CR connector has an indented cut form that constitutes a good alternative to Perfobond connector, because it provides an easier reinforcement bars disposition. It presents a symmetric cut, with trapezoidal saliencies and re-entrant angles, which provides resistance to longitudinal shear forces and prevent the transversal separation between the steel profile and the concrete slab

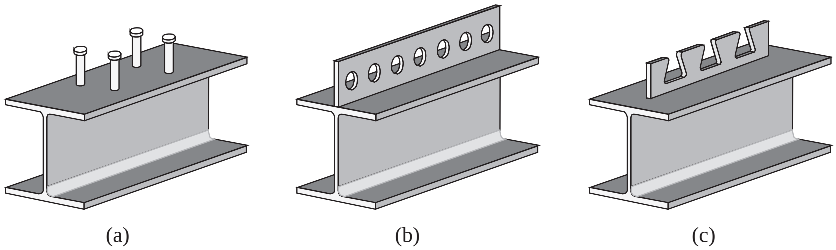

Figure 1. (a) connector *stud*; (b) connector *Perfobond*; (c) connector CR.

(uplift). The concrete positioned inside the connector's apertures works as concrete dowels, with a similar behaviour to the one obtained with *Perfobond* connectors.

Push-out tests were used to study CR connector behaviour, in order to establish the load-slip relation. The experimental tests results obtained for CR connector are critically analysed and compared to the ones obtained for *Perfobond* and *stud*, regarding the maximum load capacity and the connection ductility.

Like *Perfobond*, CR connectors present the following advantages when compared to *stud* connectors: they can be large scale produced, they can assume particular forms and shapes, they are easily welded to the steel profile with no need of special equipment, and the welding task can be performed either at site or at an industrial unit.

3 RESULTS ANALYSIS AND CONCLUSIONS

The CR connector, like *Perfobond*, presents good load bearing capacity after the load peak, which does not happen with the *stud* connector. The increase on the CR connection load capacity is proportional to the concrete strength increase.

The concrete dowel effect is very significant and it is related to an increase in the connection load capacity, in despite of concrete strength. The presence of reinforcement has also an important influence on the connection load capacity.

Tests done with CR connector showed that the average characteristic slip capacity was greater than the 6 mm limit specified by Eurocode 4 (2004), which confirms its sufficient ductility. It was verified that the connection ductility grows up with increasing concrete strength when there is no transverse reinforcement.

The experimental results presented in the bibliography and those obtained during this experimental program put in evidence several important differences between the indented connector, the stud and the Perfobond. These aspects are related to failure mode, maximum load applied during the test and connection deformation capacity.

In relation to the behaviour identified for each connector type failure mode, it has been observed that studs tend to suffer shank shear failure, immediately above the weld. On the other hand, Perfobond does not undergo failure itself, but it tends to cause intense cracking in the concrete slabs. The CR connector presents an intermediate behaviour, since it produces concrete slab cracking associated to some visible deformation of the dents. The CR connector exhibits lower load capacity than a Perfobond connector of similar dimensions.

Several tests performed with CR connectors by the authors of this work and with Perfobond connectors by other investigators showed that both the connection load capacity and ductility are influenced by the concrete strength and the slab transversal reinforcement. Therefore, it is possible to control CR connection load capacity, by properly choosing concrete strength and reinforcement rate. On the other hand, when stud connector failure is governed by shearing, an increase on concrete strength has only a small influence on the connection load capacity.

Usually, CR steel rib connectors show higher stiffness for service loads than studs. The difference between these connectors stiffness is considerable and it is important to emphasize that the elastic range for steel rib connectors is greater than the one observed for studs. In the same way, the slip correspondent to ultimate load in tests performed with steel rib connectors is lower than for studs. The post peak behaviour is characterized by a slower load loss. As the failure does not occur by connector shearing, the final deformation is very large.

The results obtained have shown that the choice of a connector type must take into account the differences in structural behaviour and an evaluation on advantages and disadvantages of its use. These aspects will have direct influence over the structural element response for which the connector is designed and also on the type of loading imposed to the connector along its service life.

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Cyclic loadings on steel and lightweight concrete composite beams

I. Valente & P.J.S. Cruz
Universidade do Minho, Guimarães, Portugal

ABSTRACT: This communication describes the experimental cyclic tests performed on steel and high strength lightweight concrete composite beams, at Universidade do Minho, Portugal. The experimental study involves tests on simply supported beams, all with the same geometrical disposition, supports and materials. Headed studs are used to provide the connection between the steel profile and the lightweight concrete slab. The parameters in study are the stud disposition and the number and range of the load cycles. Two types of stud disposition are chosen: one that guarantees full connection and other that guarantees partial connection. The beams are tested with a six-point loading, uniformly spaced along the beam, and the cycles are controlled by load. The main objectives are: to describe the composite beams behaviour, focused on the connection between steel and lightweight concrete; to analyse the beams load capacity and to analyse the influence of the number and range of the load cycles.

1 BEAMS IN STUDY

The beams are composed by an IPE120 steel profile and a 350 mm × 60 mm lightweight concrete slab. Shear connection is provided with equally spaced shear studs of 13 mm diameter and 50 mm high. The shear connectors distribution is of two types: *Type 1* – total connection, with 8 studs of 13 mm diameter and 50 mm height, distributed in half span of the beam and *Type 2* – partial connection with 4 studs of 13 mm diameter and 50 mm height, distributed in half span of the beam (Table 1). The load configuration corresponds to four concentrated loads, equally spaced of 900 mm along the beam, approximating a uniformly distributed loading (Figure 1).

Four composite beam specimens were tested, in order to evaluate the composite beams cyclic behavior. Of these four specimens, two are submitted to monotonic loading (VM4 and VM6), for evaluation of the connection load capacity and the load-slip curve, and the other two are submitted to cyclic loading (VM9 and VM10).

2 RESULTS FROM TESTS

Beam VM4 shows a bending failure. Concrete crushes initiate in concrete section top fibre, near the load application point. Different from the previous beam, VM6 suffers shear connection failure between the concrete slab and the steel beam. Connector failures are phased, with load capacity losses associated.

Beam VM9 suffered bending failure. As it did not suffer failure during the load cycles, a monotonic loading was applied, until failure was attained. The 11000 load cycles applied to beam VM10 were not enough to provoke failure and therefore a monotonic loading was also applied to this beam after the load cycles. Beam VM10 suffered shear connection failure.

2.1 *Evolution of slip*

The level of load applied to beam VM9 is in a range that fits the elastic behavior of the shear connection. Even so, during the 5000 cycles of a constant load range applied, it is possible to

Table 1. Distribution of stud connectors.

Beam	Connection	Stud distribution
VM4	Total	*Type 1*
VM6	Partial	*Type 2*
VM9	Total	*Type 1*
VM10	Partial	*Type 2*

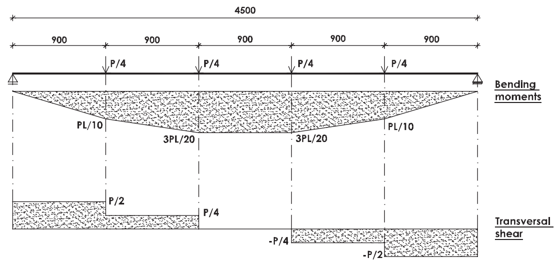

Figure 1. Loading and corresponding bending moment and shear force diagrams.

identify that slip grows with the number of load cycles applied and that this growth is approximately linear. The linear growth rate is equal to 4.33×10^{-6} mm/cycle.

For beam VM10, the rate of slip growth is approximately the same for the first 4500 load cycles. After these cycles, it suffers a small increase until around 7000 cycles are completed. From this moment on, large slip values develop on one side of the beam, while the other side tends to maintain the slip previously achieved. The rate of slip growth until $N = 7000$ cycles is well approximated by a linear law. This linear growth rate is equal to 105.0×10^{-6} mm/cycle.

2.2 *Evolution of strain during the load cycles*

The steel to concrete connection guarantees the total shear force transmission for $0.4M_{max}$, both forVM4 and VM6. The total interaction hypothesis is valid for VM4 even for maximum bending moment. For VM6, the loss of shear connection is verified from lower loads.

There is a small variation on the strain diagram during the load cycles application on VM9. In both cross sections, the variation of slip, *ds/dx*, suffers a small increase during the 5000 load cycles applied, meaning that the evolution of slip has a small influence on the strain diagram.

For beam VM10, the strain diagrams presented show high variation during the load cycle application. There is an important increase on the variation of slip, *ds/dx*, during the 11000 load cycles applied, resulting in an important transfer of stress between concrete and steel sections. At section B-B', maximum strain at the steel section grows to more than twice its initial value, while maximum strain at the top fibber of the lightweight concrete section tends do decrease.

2.3 *Evolution of vertical deflection*

Beams VM4 and VM6 show an initial elastic behaviour, approximate to estimated by an elastic approach. Considering the elastic zone for both types of loading, the total connection hypothesis (VM4) show higher stiffness. A loss of stiffness is verified at each specimen for values over $0.45\,M_{max}$. Before failure, beam VM6 presents higher vertical deformation than beam VM4, when comparing the same level of loading.

The diagram correspondent to beam VM9 is very similar to the diagram presented for beam VM4, which means that the 5000 load cycles could not induce a significant increase of deformation. When comparing with beam VM6, an increase on the vertical deformation is observable for beam VM10 during the 11000 load cycles, leading to an early failure of the beam.

3 CONCLUSIONS

Beams VM4 and VM9 suffered bending failure and beams VM6 and VM10 suffered shear connection failure, as predicted. Beams VM9 and VM10 were submitted to cyclic loads. The load levels and number of cycles applied induced reduced loss of shear connection on beam VM9, while for beam VM10, the loss of shear connection resulted in a reduced maximum resistant bending moment.

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Numerical analysis and assessment of a cable-stayed bridge during construction

A.A. Henriques, R. Faria & J.A. Figueiras
LABEST, Faculty of Engineering of the University of Porto, Portugal

C.M. Félix
LABEST, Institute Polytechnic of Porto – Engineering School, Portugal

ABSTRACT: Monitoring of bridges can provide many benefits to designers, constructors and operators. They are important to build more durable and safer structures. LABEST research unit has been developing and applying this new area of knowledge to different structural systems and real structures, answering to construction companies needs and contributing to cheaper and safer structures that can be built and operated for long periods of time with minimum maintenance.

Interpretation and discussion of the results obtained during a monitoring campaign of a structural system is a difficult task. Numerical models are important tools to improve the application of monitoring systems. They can be used to define adequate algorithms for structural damage detection and to assess accuracy and confidence on results achieved from monitoring. This work presents an evaluation of the accuracy obtained from the new monitoring system set developed at LABEST research unit applied to a cable-stayed bridge built in the city of Oporto (Fig. 1) and designed by *GRID – Consultas, Estudos e Projectos de Engenharia, Lda*. Results will be presented and discussed by comparing the values from the monitoring campaign with the ones from a numerical analysis.

Strains, temperatures, rotations and deflections have been monitored on the bridge deck and on the mast during construction, as well as during the subsequent loading test and current service-life by using a standard system based on electrical sensors providing the automatic and simultaneous interrogation of all sensors, with a minimum human intervention (Félix, 2005). A numerical analysis of the construction was also performed. The inherent numerical program was developed at LABEST, and it is based on finite element techniques using an elasto-visco plastic formulation (Henriques, 1998). The effects due to quasi instantaneous loads applied during the construction phases, such as, application of prestress or deck concreting are emphasized. Thermal effects due to variations of

Figure 1. General view of the cable-stayed bridge (© F. Piqueiro, Foto Engenho, Lda.).

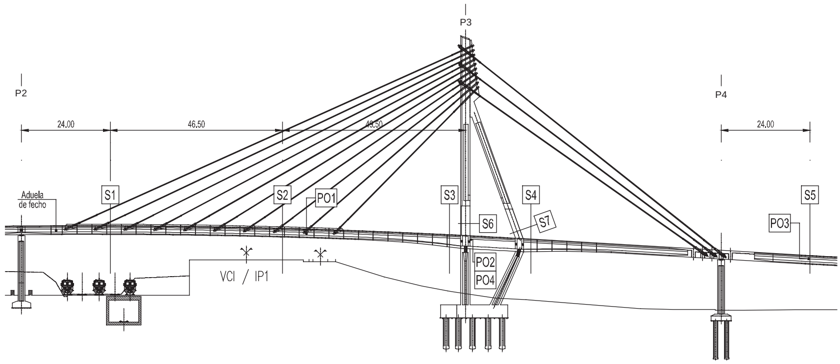

Figure 2. Location of the instrumented sections (S1 to S7) and observation stations (P01 to P04).

temperature and time dependent phenomena will be discussed, taking into account the information obtained from the monitoring campaign.

The monitoring during construction included the instrumentation of five sections at the deck and two sections at the mast (Fig. 2). For the observation of these sections, thirty eight electric strain gages and eleven temperature sensors were embedded into concrete. The equipment is programmable with a predefined time interval between acquisitions; its use is autonomous, allowing the monitoring of the structural behavior for several weeks.

The constructive process was simulated by a finite element code that accounts for the non-linear constitutive relationships of concrete and steel, as well as time-dependent effects like concrete ageing, shrinkage and creep and prestress relaxation (Henriques, 1998; Henriques & Figueiras, 2002). The segmental erection of the bridge was simulated by a discretization with Timoshenko beam finite elements for the reinforced concrete, and axial curvilinear elements for the prestressed cables.

Results obtained from monitoring and numerical analysis were compared, some differences were observed. They can be explained by simplifications introduced in the numerical model, such as the necessity to simulate cast-in-place of girder segments or prestressing operations by instantaneous loads; or by uncertainties concerning the real concrete modulus of elasticity and the prestress actually applied. Despite these differences, results show identical variations and values are similar.

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The collision behaviors between the navigating vessel and the fender systems against the medium collision event

G.H. Lee
Faculty of Ocean System Engineering, Mokpo National Maritime University

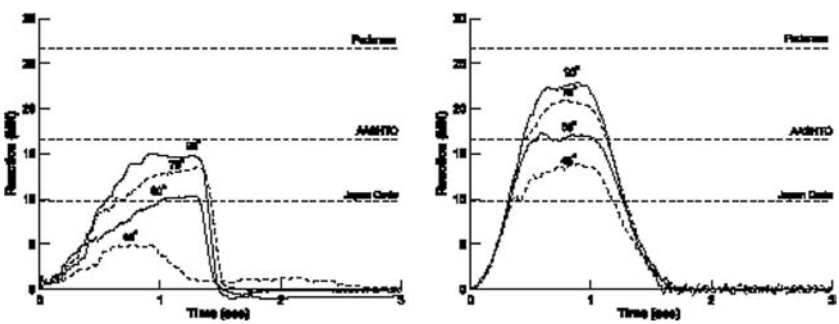
S.L. Lee
Division of Construction Engineering Mokpo National University

J.Y. Ko
Faculty of Ocean System Engineering, Mokpo National Maritime University

ABSTRACT: In this paper, the collision behaviors between the navigating vessel and the fender systems against the medium collision event were studied. The considered fender systems were the steel fender system and the rubber fender system, and it was assumed that these fenders were fixed on the face of bridge substructure. The bow structure of collision vessel that had the standardized dimensions by design code was model by shell elements. The main body of vessel was modelled by the elastic beams that represent the mass distribution, and the buoyancy springs were attached to the main body to represent the change of potential energy caused by buoyancy effects. The analysis were performed by the nonlinear explicit method with the considerations of collision conditions, such as, the collision velocities, the collision angles and the friction coefficients.

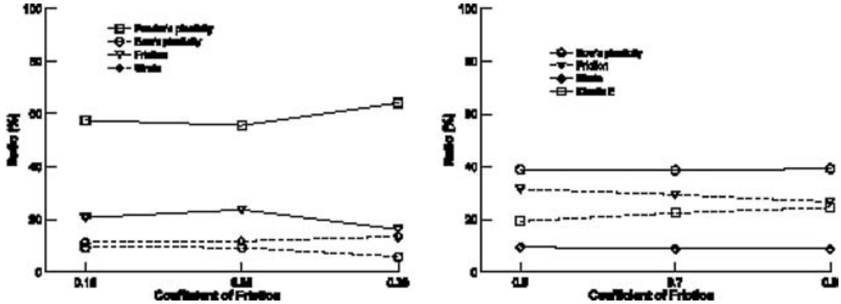
The time histories of the collision forces as the types of fender are presented in Figure 1. The horizontal dashed lines denote the collision forces by some design codes. The collision conditions of these two cases are identical in view points of the design formula, the same collision forces have to be obtained. However, the obtained forces are too conservative (the case of the steel fender) or not conservative (the case of the rubber fender). This means that the conditions of the collision faces are not ignorable in the medium collision events.

The second energy dissipation mechanisms of each case were the friction between the vessel bow and the fender systems. To classify the effects of the friction coefficients, the energy dissipation ratio of the analysis are presented by the friction coefficient in Figure 2. The lines with reverse triangles are denoted the friction energy dissipation ratios. These ratios were not affected by the changes of the friction coefficients. It is worth to note that the code propose the changes of the



a) Steel fender b) Rubber fender

Figure 1. Collision force time history.



a) Steel fender b) Rubber fender

Figure 2. Energy dissipation ratios as the friction coefficients.

energy dissipation ratio by the friction coefficient. This differences are caused from the rigid wall assumption. In the real collision event, the bow of vessel indented into the fender system and locked by indentation. Therefore, the lateral movement of the vessel bow is available only when the lateral collision force vessel is larger than the lateral resistance force of the fender systems. Therefore, the energy dissipation behaviors of the both fender system showed together that the rigid wall assumption is unrealistic in the medium collision events.

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A modern concept of movable scaffolding systems

António Póvoas
Bridge Construction Systems, Portugal

ABSTRACT: The great evolution of pre-tensioning technology allowed structural engineers to approach the design of bridges in a different way, by using larger and wider spans, in order to reduce global costs and construction periods. The mobile equipments have been following this tendency, and some of them may now be used to build bridges with larger spans, in less time, with lower global equipment costs. This article presents a modern concept of such equipments.

1 MOBILE SYSTEMS FOR THE CONSTRUCTION OF BRIDGE SLABS

The most known mobile equipments are divided in four groups such as:

- Mobile formwork running over portal or conventional scaffoldings
- Launching girders
- Travellers
- Movable scaffolding systems

2 THE AP-2005 MOVABLE SCAFFOLDING SYSTEM

In the last 12 years the author is being designing movable scaffolding systems, launching girders, and other mobile and fixed equipments for the construction of bridges, from underwater foundations to cable stayed bridges.

The author has recently developed a new model, which reunites the different types of mobile equipment in a convertible and autonomous equipment, designated as AP-2005.

This equipment is supported on tested solutions from its predecessors, and mainly consists in a powerful equipment that can transform itself along the same bridge, into a movable scaffolding

Figure 1. Vasco da Gama Bridge – Lisbon – 1995

Figure 2. 100 m high Côa Bridge – Portugal – 2002

Figure 3. High speed train Bridge – Barcelona – 2005

system, a traveller for advancing "in situ" casting, or a launcher for both prefabricated segments and concrete or steel beams.

This allows both designers and construction companies to make bridges with mixed structural solutions along its length, using one single type of equipment.

The most used structural solutions for bridges consist in one or a few central large spans with access viaducts.

This equipment allows one phase 72 m span access viaduct, and advancing, 130 m span central viaducts. The spans may be either concrete or composite, with steel beams and concrete slabs.

The equipment is autonomous for the erection of its own supports and operations, ant is capable of performing the full set of operations: – pre-tensioning, opening of the formwork, launching, closing the formwork and preparing for the entry of the steel bars in only 24 hours.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Evaluation of performance on bridges with overloading trucks

Y. Wang
South Africa-China Transport Co-operation Center, Centre for Entrepreneurship, Tshwane University of Technology, South Africa

D. Hu
Chang'an University, China

Z. Liu, H. Xie & J. Liu
Guangzhou Municipal Engineering Testing Center, China

J.P. Grundling
Centre for Entrepreneurship, Tshwane University of Technology, South Africa

ABSTRACT: WANZHAI Bridge over the Diangwan River is located in the Xuzhou City, Jiangsu province of China. The span is 3×30 m composite RC T girders. The substructure consists of abutments, piers and pills. The width of the bridge is as $12.0 + 2 \times 1.5$ m. Design load is Truck-20 and Checking load is Trailer-100.

ZENGBU bridge is located in Guangzhou City, in the Guangdong Province of China. The total span is 35 m + 50 m + 35 m with RC trussed arch. The ratio of height to span is as 1/7. The width of the bridge is 2.5 m (sidewalk) +3.8 m (low speed route) +0.5 m (crashworthy wall) $+2 \times 7.5$ m (express way) +0.5 m (guardrail) +3.8 m (low speed route) +2.5 m (sidewalk). The bridge deck has 3% longitudinal slope and 1% bi-direction transverse slope. Design load is Truck-20 and checking load is Trailer-100. crowd load is 3.5 kN/m^2. The bridge was open in 1980.

This paper introduces the performances of two bridges under overloading conditions. With heavy vehicles two bridges undergo overloads which have passed highway bridges. This paper gives two bridges' investigating data and impact factors. It briefly analyses the reasons of the overloading for those bridges. Some proposals to strengthen these bridges and control overload are raised.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Research on lifetime performance-based bridge design method

J. Peng, X. Shao & X. Jin
Institute of Bridge Engineering, Hunan Univ., Hunan Changsha, China

ABSTRACT: Based on life-cycle cost (LCC) analysis method, this paper presents conceptual framework of LCC-based bridge design method. Namely, taking bridge service level as a basis, the design method minimizes the net present value of life-cycle cost during bridge lifetime as optimal performance objective, and makes use of life-cycle cost analysis method to evaluate bridge design strategy subjected to structural performance constraints whether or not it is an optimum strategy. Life-cycle cost model can be determined by establishing traffic model and bridge lifetime performance prediction model that is associated with bridge serviceability under aggressive environmental attack. Therefore, the hybrid life cycle cost optimization model can be constructed based on the above models. An approach to investigate LCC-based bridge design is proposed in this paper, including systematic method study, parametric uncertainties analysis and risk evaluation. At the same time, structural failure modes and bridge time-variant reliability are researched, and the hybrid life-cycle cost optimization techniques are discussed in the paper. Optimal algorithm is employed as a search method for the optimal design strategy solution, and Monte Carlo simulation is taken as risk analysis approach to account for the uncertainties. The method that stresses on the interaction of lifetime performance prediction, reliability-based life-cycle cost and maintenance/repair interventions has many advantages, such as explicit concept and convenient realization. The related theories developed in this paper can provide conduction for bridge design, construction and lifetime maintenance or replacement actions.

The existing bridge design method in China mainly has the following defects, the first is paying much attention to the initial cost, and ignoring maintenance cost, future repair cost and failure cost, the second is that the reliability of the deteriorating structure and maintenance cost and interaction between the maintenance actions are investigated respectively, and these problems are not discussed from design viewpoint.

Life-cycle cost includes agency cost, user cost and societal cost, and it is not an easy task to establish life-cycle cost models for life-cycle cost-based bridge design method. This is because the LCC models should depend on many uncertain factors, such as structure types, the importance of members, reliability index, deterioration rate, types of maintenances actions, and loading conditions.

The design method of life-cycle performance-based bridge design is proposed in this paper. That is to say, this design method is to decide the optimal design strategy based on the minimum present value of life-cycle cost.

The performance objective of this design method based on life-cycle cost is the minimum long-term investment and the maximum serviceability. That is to say, the minimum of the present value of life-cycle cost-based on structural serviceability constraint.

The life-cycle performance objective can be expressed as follows:

$$\min \quad \mathrm{NPV}; \quad \text{s.t.} \quad \mathrm{BSI} \geq \mathrm{BSI}_{\min} \tag{1}$$

where: BSI = bridge serviceability index; $BSI_{\min}$ = bridge minimum serviceability.

Bridge life-cycle performance is decreasing due to environmental attacks. Serviceability index is always descending because of concrete carbonation and steel reinforcement corrosion.

It is a basis that characteristics and action ways of all kinds of stochastic parameters that affect bridge condition are systematically analyzed. As a whole, there are interior, external and economical factors. Every parameter has its probability density function, and impacts structural performance differently. Bridge life-cycle is divided into design phase, construction phase and service phase.

1. Different factors interact and unite each other, and bridge performance maintenance should have all-round systematic viewpoint.
2. Bridge performance exists in every phase of lifetime, and bridge design should have lifetime viewpoint.
3. Uncertain parameters analysis should be included in bridge life-cycle performance and lifetime reliability estimation.

Significant progresses have been made in the past decade for design and analysis of highway bridge infrastructures based on LCC considerations. Frangopol (1999) developed the basis for cost-effective bridge management incorporating the concepts of lifetime reliability and LCC. Frangopol et al. (2001) reviewed the recent advances in BMS and emphasized the importance of integrating lifetime reliability and costing into life-cycle management of highway bridges. Hassanain and Loov (2003) discussed cost optimization of concrete bridge components and systems with a review of developments in LCC analysis and design of concrete bridge.

Life-cycle cost includes agency cost, user cost and societal cost, and it is not an easy task to establish life-cycle cost models for life-cycle cost-based bridge design method.

The criteria for evaluating the risk associated with each treatment decision are: 1) maximum expected utility, and 2) cumulative risk profile. Maximum expected utility criterion is useful for determining a treatment policy that will provide the best result in terms of overall performance and investment. It is intuitive that the initial treatment decision providing the maximum expected utility is also the decision with the lowest total risk. However, decision based solely on the maximum expected utility value criterion may not be the 'best' policy because the variability of possible decision outcomes in not explicitly evaluated. Thus, cumulative risk profiles are utilized to evaluate decisions according to the distribution of risk and risk dominance criteria.

Future costs are considered in life-cycle cost analysis method, and discount rate is a very important parameter, and sensitive to life-cycle cost analysis.

There is close relationship between maintenance cost and the effect of the intervention on system reliability. For instance, the system reliability is high, and usually no repair will dramatically improve it. If do so, the cost will become high.

A simple sample is used to conduct the design of Bridge abutment by the new bridge design method based on life-cycle cost.

1 CONCLUSIONS

Life-cycle performance-based bridge design method is an innovative design concept, presented in the latest twentieth century. This design method takes the minimum present value of LCC as the optimal objective subjected to performance constraint, and seeks predictive serviceability and maintenance cost during bridge service time. This method not only assures sustainable serviceability of bridge, bur also controls long-term investments and decreases adverse impacts on the society.

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Ultimate strengths of partial composite beams considering long-term effects of concrete slabs

D. Bae
School of Civil & Environmental Engineering, Kookmin University

S.G. Youn
Department of Civil Engineering, Seoul National University of Technology

H.K. Ryu
Department of Civil Engineering, Seoul National University

Y.S. Park
Department of Civil Engineering, Myong Ji University

ABSTRACT: In this paper, experimental and analytical studies on ultimate strengths of simply supported partial composite beams with compact-section steel girders were conducted. Long-term effects were also considered. Four 6.5 m long steel-concrete composite girder specimens were fabricated. To investigate the effects of creep and shrinkage of concrete slabs on composite beams, long-term behaviors such as the changes of deflections, curvatures, and strains were measured for one year. After 400 days, three specimens, which have different degrees of shear connection, were tested to get the ultimate strengths in positive moment region and another one for negative moment failure. Long-term test results were compared with the analytical results obtained by using the age-adjusted effective modulus method (AEMM). Elastic-plastic finite element analysis considering relative slip between slab and girder was also performed to estimate the theoretical ultimate strengths of the specimens. From the analytical and test results, the effects of long-term behavior of concrete, partial shear connection or interaction and material nonlinearity on the ultimate loads of composite beams were evaluated and discussed.

1 INTRODUCTION

In view of composition of steel and concrete, it is necessary to understand the concepts of partial interaction, or the degree of interaction and partial shear connection, or the degree of shear connection. In partially or fully composite beams, the long-term effects of concrete should be considered. Moreover, the effect of creep and shrinkage in composite beams is more significant because deformation of concrete is restraint by steel girder. Deflection, stress resultants and other mechanical behaviour of composite beams at each construction phase considering long-term effects should be also analyzed cautiously. In this paper, experimental and analytical studies on ultimate strengths of simply supported partial composite beams with compact-section steel girders considering long-term effects of concrete slabs were conducted.

2 TEST SPECIMENS

Average 28 days compressive strength of concrete slab was 33 MPa. SM490 steel was used in steel girder which has yield strength of 400 MPa and ultimate strength of above 490 MPa. A diameter of reinforcement arranged in concrete slabs was 13 mm. Four 6.5 m long steel-concrete composite test specimens were fabricated. Concrete slab width and thickness are 400 mm and 160 mm, steel girder flange width and thickness are 150 mm and 9 mm and web depth and thickness are 282 mm and 6.5 mm respectively. The ratio of longitudinal reinforcement was 0.75%. After installation of girder, casting of concrete was conducted without prop and then moisture curing was carried out during 7 days. In all specimens, distributed spacing and number of shear connection are same but a diameter of shear connection was different as 13 mm, 16 mm and 19 mm.

3 LONG-TERM TEST

3.1 *Shrinkage test results*

As the time elapsed, mid-deflection of composite beam increased due to shrinkage of concrete slabs. Also, strain of composite girder was changed. Transverse cracking developed from bottom of concrete slab section to top of slab was mainly investigated. Micro transverse cracking from top to bottom slabs was also observed. Mid-deflection of composite beam and strain of concrete slab were presented. Analytical curve based on AEMM using criteria of concrete in CEB-FIP was compared. In the comparison of test results with analysis for steel girder, the effect of temperature variation could be not neglected. Thus, temperature compensation had been conducted.

3.2 *Creep test results*

Similarly, as the time elapsed, mid-deflection of composite beam increased due to creep of concrete slabs. Mid-deflection and strain of concrete slab was investigated. Analytical curve based on AEMM using criteria of concrete in CEB-FIP was compared with test results. In the comparison of test results with analysis, the temperature compensation was also included.

4 STATICAL FAILURE TEST

4.1 *Ultimate strength*

From the failure tests, load-deflection curves were obtained. Ultimate loads of the specimens calculated by rigid plastic analysis were estimated as 220.92 kN for the positive moment test specimens and 140.31 kN for the negative moment test specimen. It is confirmed that ultimate loads for all specimens were higher than the calculations, although the SM2 and SM3 was designed as partial shear connection composite beams.

4.2 *Inelastic finite element analysis*

In order to estimate the ultimate behaviour of partial composite beams, an elastic-plastic finite element analysis was conducted and the results of analysis works were compared with test results. Additionally, in order to analyze the ultimate strength of composite beams considering construction phases which includes long-term effects, stepwise analysis was executed. The construction phases being included, inelastic nonlinear analysis were conducted to find out ultimate strength of the composite beams. The ultimate strength was evaluated as 240.05 kN from the numerical works where reinforcements were neglected, thus additional load capacity could be anticipated.

5 CONCLUSIONS

From the analytical and test results, the effects of long-term behavior of concrete, partial shear connection or interaction and material nonlinearity on the ultimate loads of composite beams were evaluated. As results of the long-term tests, it is considered that analytical solutions based on AEMM could estimate well the strain of concrete slabs. For the steel strain of girder, temperature compensation should be conducted. From the ultimate load tests, all specimen capacities showed over the ultimate strengths of full shear connection composite beams although the beam was designed as partial shear connection. Thus, it is considered that design concepts using full shear connections could be used conservatively for composite beams. In order to evaluate the ultimate strengths of composite beams more rigorously, additional numerical analysis considering the construction phases was conducted. From the results, if the reinforcements are also included in numerical models, it is considered that more reasonable ultimate strengths of composite beams could be obtained as similarly to the test results.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Busan-Geogje fixed link: Concrete durability design for the bridges and tunnels

C.K. Edvardsen
COWI, Lyngby, Denmark

S.-K. Jeong & J.-C. Kim
DEAWOO E&C, Seoul, Korea

G.-M. Lee
Sungkyunkwan University, Suwon, Kyunggi-Do, Korea

ABSTRACT: The concrete structures of the bridges for the Busan-Geogje Fixed Link (South Korea) shall be designed for 100 years service life.

To achieve the 100 years service life for the concrete structures in the envisaged environment is a very demanding task that can not be fulfilled with traditionally used over-simplistic durability approaches which are based on so-called deemed-to-satisfy rules. Examples of such deemed-to-satisfy rules are requirements to minimum concrete cover, maximum water/cement ratio, and minimum cement content. These traditional durability approaches provide only qualitative definitions of exposure and they fail to define the design life in relation to durability. This makes it important that similar to the current procedures for structural design, a performance based durability designs is incorporated in the present project which takes into account the probabilistic nature of the environmental aggressivity, the degradation processes and the material properties involved.

Therefore, a probabilistic based durability approach has been adopted as part of a rational design basis for the reinforced concrete bridge structures of the Busan-Geogje Fixed Link to document the required service life and the acceptable level of reliability.

The Busan-Geogje Fixed Link project comprises a 8.2 km motorway link from Busan, Korea's southernmost and second largest city, to the island of Geogje. The connection includes a 4 km immersed tunnel – the deepest in the world with 50 metres water depth – and two cable-stayed bridges each 2 km in length. The project is scheduled for completion in 2010. COWI is the leading consultant for both the bridges and the tunnel, DAEWOO E&C is the leading contractor.

The DuraCrete approach developed during an European Research project (1996–1999) and internationally the only available probability-based service life approach, has been adopted as service life design methodology for the reinforced concrete bridge and tunnel structures of the Busan-Geogje-Fixed Link considering the governing deterioration mechanism, i.e. chloride induced reinforcement corrosion. Key durability factors governing the DuraCrete design methodology are the chloride diffusion coefficient and the age factor. Both parameters are functional requirements of the Concrete Specification of the project and have been determined and verified during a comprehensive pre-testing program of concrete mixes performed by DAEWOO Institute of Construction Technology (DICT).

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*Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)*

Quasi-static tests on concrete encased composite columns

C.-S. Shim, Y.-S. Chung, J. Min & J.-H. Han
Chung-Ang University, Ansung-Si, Korea

ABSTRACT: In the design of bridge piers in seismic area, the ductility requirement is the most important factor. In order to enhance the seismic performance of RC columns, it is necessary to make the ductility of columns larger by covering RC columns with steel tubes or confining RC columns by arranging transverse reinforcement such as hoop ties closely. Using core steel composite columns is useful as one of the reinforcing RC columns. In this paper, quasi-static tests on concrete encased composite columns with single core steel or multiple steel elements were performed to investigate the seismic performance of the composite columns. Eight concrete-encased composite specimens were fabricated. The cross-sections of these specimens are composed of concrete-encased H-shaped structural steel columns and a concrete-encased circular tube with partial in-filled concrete. Test parameters were the amount of the transverse reinforcement, encased steel member, and loading axis. Through the tests, it was evaluated the ductility of SRC composite specimens. It has become clear from the test results that encased steel elements makes the deformation capacity of the columns to be large. Based on the test results, design considerations on the details of the reinforcements were proposed.

1 INTRODUCTION

This paper included quasi-static tests for eight concrete-encased composite specimens. The cross-sections of these specimens are composed of concrete-encased H-shaped structural steel columns and a concrete-encased circular tube with partial in-filled concrete. Test parameters were the amount of the transverse reinforcement, encased steel member, and loading axis. Based on the test results, design considerations on the details of the reinforcements were proposed.

2 EXPERIMENTAL WORK

Eight concrete-encased composite columns with aspect ratio of 3.5 were fabricated to investigate the seismic behavior of the composite columns. Three composite columns (QSHS) had a H-shape steel member and two specimens (QSCT) a partially concrete filled steel tube. In order to increase the structural capacity of the composite column, three specimens (QMHS and QMCT) had four embedded steel members. Composite columns with an embedded steel member had 1.63% steel ratio, which is similar with that of the current normal RC piers. Multiple steel member embedded composite columns were designed to have 3.8% steel ratio. One of the main parameters in this paper was the transverse reinforcement. Based on the current bridge design codes, five columns which satisfy the ductile requirements and three specimens satisfying the limited ductility requirements were designed.

In this paper, quasi-static tests on concrete-encased composite columns were conducted and seismic performance was investigated in terms of ductility and energy absorption capacity. it is concluded that for composite columns with low steel ratio:

1. All the specimens showed the flexural failure mode having fracture of main reinforcements in the plastic hinge region when the steel ratio of the embedded steel elements is low.

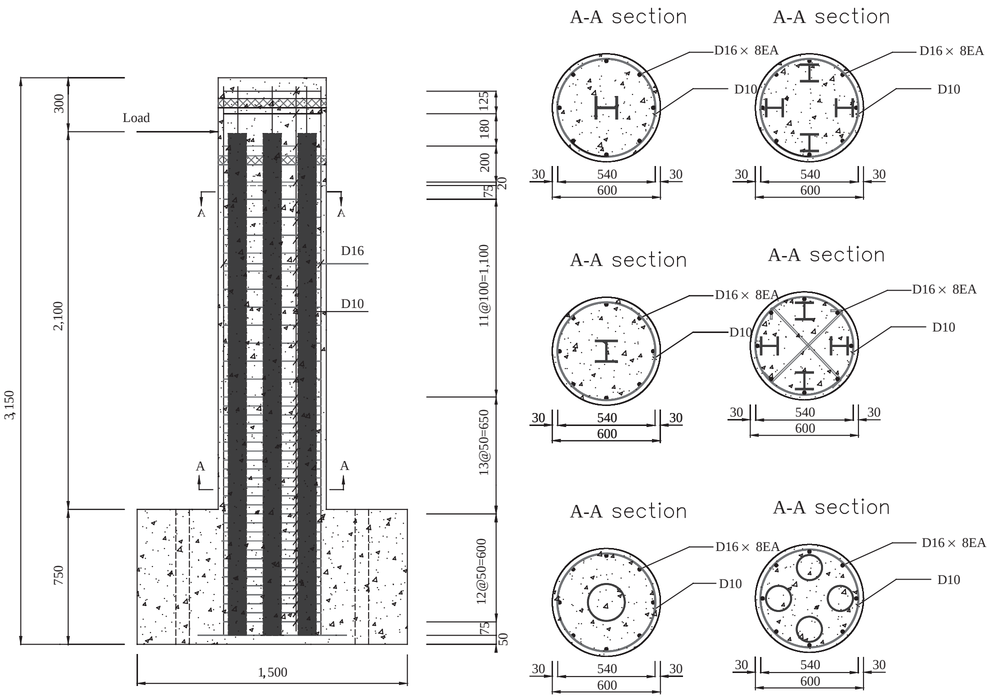

Figure 1. Test specimens cross section (unit: mm).

2. In terms of the ultimate strength of composite columns, the specimens with a partially concrete filled steel tube showed 10.9% higher strength than the specimens with a H-shape steel. However, for multi-steel members embedded composite column, QMCT-BA specimen showed only 4.3% strength increase resulting from the weakness of the composite action between concrete and a steel tube in the plastic hinge region. Therefore, it is necessary to have mechanical connectors for the enhancement of the structural efficiency.
3. It is preferable for composite columns with single steel member embedded to require the transverse reinforcement ratio as the normal RC columns. When composite columns having structural steel ratio less than 2.0% are designed, it is appropriate to evaluate the P-M interaction diagram by concrete-based design codes such as ACI-318 and Eurocode-4.
4. In terms of the energy absorption capacity, composite columns with single steel tube showed higher value than columns with a H-shape steel. However, for multi-element embedded composite columns, specimens with H-shape steels had better performance than that with steel tubes.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Effects of thickness and yield strength of steel on peeling stress

K. Nozaka & T. Furukawa
Ritsumeikan University, Shiga, Japan

K. Suzukawa
Toray Industries Inc., Shiga, Japan

1 INTRODUCTION

In the last several decades, it has been widely recognized that bonding CFRP strip on concrete structures is one of the effective ways for strengthening or rehabilitation because of its high modulus and high strength compared to concrete. However, there are only few cases in which the rehabilitation method by bonding CFRP strips has been applied to existing steel structures due to relatively small number of research works and insufficient understanding on the strength of adhesives in the use of this rehabilitation method. Because the stress distribution in the adhesive layer is significantly complicated, conducting experimental tests would be a reasonable method to investigate the bond strength of adhesive used for rehabilitation of steel members and to confirm the effectiveness of the rehabilitation method.

In this study, steel plates strengthened by bonding CFRP strips, and two steel plates butted each other on which CFRP strips were bonded were loaded in tension; the number of CFRP layers, the yield strength and thickness of steel materials were varied. The strength of adhesive related to the peeling failure of CFRP was investigated.

2 EXPERIMENTAL TEST

Specimen tested in this study were classified in two types, Type A and B; and typical dimensions. The specimen in which CFRP strips were bonded on both sides of a steel plate was classified as Type A, while the one in which CFRP strips were bonded on two steel plates butted each other was classified as Type B. In Type B specimens, a crack in a steel plate was simulated by a slit between two steel plates.

Two types of yield strength (SM400 and SM570) and thicknesses (19 mm and 25 mm) were tested for steel materials. In addition, the number of layers bonded on one side of a specimen was varied, one, two and three. For example, Type A specimen in which one CFRP strip was bonded on each side of a steel plate (SM400, 19 mm thick) was designated as "19-A400-1." A total of 18 specimens were tested in this study.

CFRP strips used in this study were ML520 (2 mm thick, 50 mm width: manufactured by Toray Industries, Inc.). All steel plates were shaped in 50 mm width. The adhesive used in this study was DP-460 which was two-part, room temperature cured epoxy adhesives.

The specimens were loaded in tension by gripping both ends of the steel plates; and loading was applied by displacement control.

3 RESULTS AND DISCUSSION

3.1 *Peeling stress*

The failure mode for all specimens including Type A and B specimens was debonding of the CFRP strips from the steel plates. Based on the test results, it is found that the peeling stress is affected by

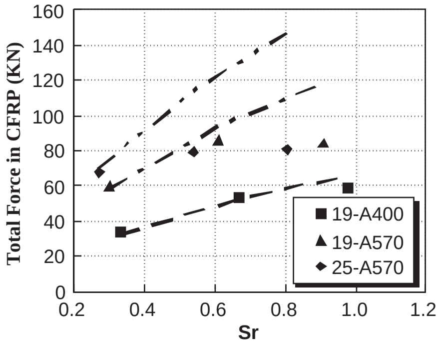

Figure 1. Comparison on total force in CFRP strips.

the yield strength of steel plates; i.e., the peeling stresses are close to the yield strength of the steel plates for specimens with SM400 steel, while some of the peeling stresses are well below the yield strength for specimens with SM570 steel. In addition, increasing *Sr* has an opposite influence on a peeling stress for Type A and B specimens; the peeling stress decreased for type A and increased for Type B as *Sr* increasing. Furthermore, the peeling stress for specimens having the same *Sr* tends to decrease as the thickness of the steel plates increases.

One should be noted is the fact that the peeling stress for Type A and B specimens getting closer as *Sr* increasing. Thus, for specimens with large *Sr*, the peeling stress for Type A and B specimens seems to become an equal value.

3.2 *Prediction of peeling stress for Type A*

As discussed in the preceding section, the peeling stress is affected by both thickness and yield strength of steel plates; thus, an equation for predicting the peeling stress is required to include these parameters. To develop a simple equation to predict the peeling stress, a total force in the CFRP strips at the center of bonded length is considered in this study. In Figure 1, a total force in CFRP strips bonded on one side of the steel plate (vertical axis) is plotted against *Sr* (horizontal axis). As can be seen in the figure, $T_{2\_x=0}$ seems constant at approximately 80 kN for the specimens which failed before steel plates yield. Therefore, it can be said that a peeling failure occurs when total force in CFRP strips bonded on one side reaches 80 kN for materials tested in this study. For other combination of *Sr* and thickness of the steel plate, the following equation, Eq. 1, is proposed to estimate the peeling stress, σ_{p_pre}.

$$\sigma_{p\_pre} = \frac{80000 E_s}{\xi E_c n t_c b_c} = \frac{800000}{t_s b_c} \frac{S_r + 1}{S_r} \tag{1}$$

4 CONCLUSIONS

In this study, two types of specimens were loaded in tension. The number of CFRP layers, the yield strength and thickness of steel materials were varied to investigate the influences on the peeling stress. Based on the test results, a simple equation is proposed to estimate the peeling stress for Type A specimen. Followings are the major findings of this research.

1. Increasing thickness of a steel plate decreases the peeling stress for Type A specimens, and increases the peeling stress for Type B specimens.
2. If the low yield strength steel is used for testing peeling stress, the peeling stress could be controlled by the yield strength of steel. Therefore, high yield strength steel is recommended for testing peeling strength.
3. For Type A specimens, yielding in adhesive layer was observed.
4. For Type A specimens, a simple equation, Eq. (6) is proposed to estimate the peeling stress for different combinations of *Sr* and thickness of a steel plate.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Side-by-side box-beam bridges – design for durability

U. Attanayake & H.M. Aktan
Department of Civil & Environmental Engineering, Wayne State University, Detroit, Michigan, USA

ABSTRACT: Side-by-side box-beam bridges also known as "adjacent box-beam bridges" became popular because of construction and under clearance advantages. In fact, side-by-side box-beam is the bridge superstructure of choice for short and short to medium span bridges. Another significant advantage of this bridge type is that construction is rapid and traffic can be maintained below without interruptions. The side-by-side box-beam bridges are constructed with different design configurations in different states in the US and other countries. In Michigan, it is constructed by placing precast and prestressed box-beams adjacent to each other, mortar grouting full-depth shear keys, applying transverse post-tensioning, and casting a 150 mm single layer reinforced concrete deck.

This article discusses the Michigan side-by-side box-beam bridge design history, durability performance, and other design procedures. The need for developing a rational design procedure based on orthotropic plate behavior for durable side-by-side box-beam bridges is demonstrated.

The conceptual model of side-by-side precast prestressed concrete box-beam superstructure with transverse post-tensioning is a continuous bridge superstructure such as an orthotropic plate. In other words, the practically implementable system of a plate is the side-by-side box-beam with transverse ties. The concept is to place precast prestressed concrete box-beams adjacent to each other and to provide fully interconnected joints between the beams. These joints are expected to transfer flexure and shear loads. This type of interconnection is established by shear-keys, post-tensioning strands/tie-rods, a cast-in-place concrete slab, or a combination thereof. Based on this practically implementable system, the current design procedure was developed defined by empirical relations for the shear key, deck, and transverse post-tensioning details. Since its first introduction in the 1950s in the US, a large number of side-by-side precast prestressed concrete box-beam bridges were constructed.

The American Association of State Highway Officials (AASHTO) Specifications are the primary documents used in bridge design. The discrete box-beam design is based on a fraction of live load (truck load) in addition to dead load due to self weight, railing, and the cast-in-place concrete deck. The live load distribution between the adjacent beams is established based on the connection type between the beams along their sides. Current practice is to use the distribution factors given in AASHTO Bridge Design Specifications. Section C4.6.2.2.1 of AASHTO (2004) specifies that side-by-side box-beam bridge deck can be assumed to act monolithically if beams are sufficiently interconnected. Fully interconnected beam joint is defined as a flexural shear joint. Again, according to Section 5.14.4.3.3 of AASHTO (2004), bridge deck with flexural shear joints should be modeled as a plate. Yet, in the current side-by-side box-beam assemblage design including the transverse connection design an analysis model is not utilized. The design is based on heuristic data and empirical approach (MDOT 2003). On the other hand, *Precast Prestressed Bridge Design Manual* (PCI 2003) proposes a transverse connection design procedure utilizing a grillage model of the side-by-side box-beam bridge. The grillage model based design procedure was originally developed by El-Remaily et al. (1996).

As part of this research, 15 in-service bridges were evaluated. These bridges were selected from a group of 236 side-by-side box-beam bridges that are on the National Highway System (NHS) in Michigan, USA. The field inspection data showed that the predominant distress form observed on all new and old bridges is the longitudinal reflective cracks on the deck surface along the shear keys. The reflective cracking of the deck along the shear keys between box-beams allows penetration of surface water laced with deicing salts through the cracks in a concealed fashion and along the full-length of the beams. Inspection revealed that the box-beams are subjected to repeated and prolonged moisture exposure. Chloride ions diffuse into concrete and initiates corrosion of prestressing tendons and reinforcements leading to delamination, cracking, spall and sometimes broken tendons. Distress specific to the shear keys include longitudinal and transverse cracking and spalling of mortar forming the shear key.

Forensic investigation was performed during demolition of one of the 15 bridges that were inspected as part of this study. The shear key mortar adherence to the girders was poor. This finding is in agreement with the conclusions made by Gulyas et al. (1995) that the shear key failure is due to the poor bond between the substrate and the grout material.

In order to progress towards a theoretical platform, a box-beam assemblage that consists of eight 686 mm × 915 mm beams with 75 mm wide full-depth grouted shear-keys and 150 mm thick cast-in-place concrete deck was modeled and analyzed. The bridge span is 15.24 m with a width of 7.85 m. The assemblage was analyzed only under the dead loads generated by the self weight of the structure. Additionally, finite element analysis of an isotropic plate model was performed with dimensions similar to the box-beam assemblage described above. Deflected shapes of the box-beam assemblage and the plate models are in good agreement with the deflected shape described by the classical theory formulated by Timoshenko and Woinowsky-Krieger (1987). The grillage model proposed by El-Remaily et al. (1996) of the box-beam assemblage was analyzed under uniformly distributed load and compared to the other solutions. The comparisons indicate that the grillage model is not accurately representing the transverse plate behavior. This is because of the limitations due to the assumptions used in transforming a plate to a grillage. This result is in agreement with the findings of Gordon and May (2004).

Based on the studies on side-by-side box-beam design procedures, durability performance, and the structural behavior, following recommendations are made:

1. Grillage method, which is the only available rational analysis model, is not able to predict the transverse behavior of the box-beam assemblage
2. Current transverse posttension design increases the longitudinal deck cracking tendency by increasing transverse camber of the deck
3. Transverse posttension should be provided to compensate for the transverse camber of the bridge deck under dead and live loads
4. Transverse camber varies along the span; hence the transverse posttensioning locations along the beam depth also should vary along the span, to be based on rational analysis
5. Rational design procedure needs to be based on an orthotropic plate model by using the equivalent orthotropic plate rigidities of the box-beam bridge deck assemblage.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Preflex beams: Structural optimization and analysis of economic advantages

C. Mannini & S.G. Morano
Department of Civil Engineering, University of Florence, Italy

ABSTRACT: This paper deals with Preflex beams which are a very particular kind of composite structures, including a precasted concrete bottom flange, which is prestressed by means of pre-deflection of the steel beam. An important variant of this technology, adding prestressing with pre-strained cables to predeflection (Flexstress), is also considered. The concrete bottom flange increases significantly the stiffness with respect to instantaneous loads and it allows very slender designs. The case of a small span rail bridge is taken into account and the beam geometry as well as the material choice are optimized. These innovative solutions are then compared to a traditional composite design with the same slenderness ratio and with an existing rail bridge deck. Relevant cost savings seem to be possible, especially in the Flexstress case.

1 INTRODUCTION

Preflex technology was introduced in Belgium and Japan around the 1950s and then adopted in several other countries (e.g. Dobruszkes, 1959; Boulton & Paul, 1974; Hever, 2001; Jadoul, 2001). The main feature of this innovative structural solution is the presence of the concrete bottom flange which is poured after the predeflection of the steel beam and then prestressed when the load is released (Fig. 1). An important variant of this technology is the Flexstress beam (e.g. De Keyser et al., 1990), in which the predeflection is substituted or accompanied by pre-strained and sometimes even post-tensioned prestressing cables. The concrete bottom flange increases the stiffness of the structure and it allows to conceive very slender designs. Nowadays this technology, combined with the use of modern materials, seems particularly interesting to realize small-span rail bridges when clearance limitations impose to reduce as much as possible the height of the deck (Mannini, 2002). Nevertheless for the durability of the structure it is very important to avoid cracking, accurately accounting for the prestressing losses due to viscous effects. In this work the modular-ratio method described in Morano & Mannini (2006) is applied.

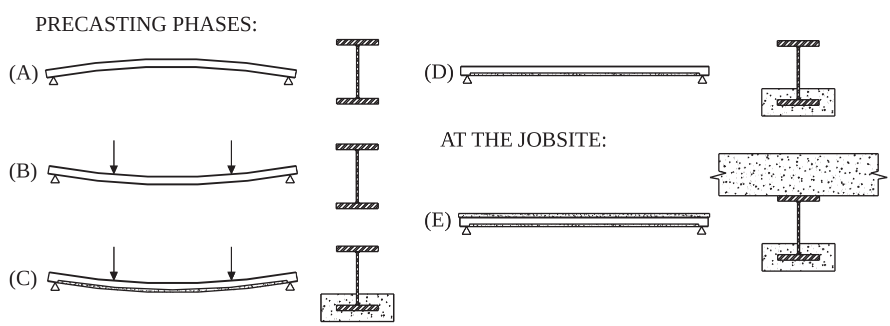

Figure 1. Phases of construction of Preflex beam: A) assemblage of the steel beam; B) predeflection of the steel beam; C) pour of the concrete flange; D) release of the composite beam; E) pour of the concrete deck slab.

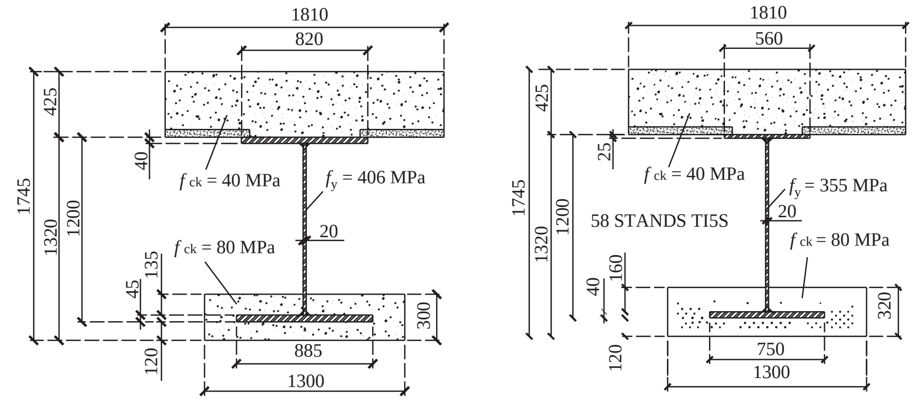

Figure 2. Optimized Preflex (2a) and Flexstress (2b) cross-sections (in mm).

This paper deals with the structural optimization of the Preflex solution for a 36 m-span rail bridge deck characterized by a slenderness ratio L/H $\cong$ 20, where L is the span length and H the deck height. The objective function is the structural steel quantity per unit deck surface, which is also used to compare the Preflex solution to a traditional composite design with the same slenderness ratio. The choice of materials plays a key role in this analysis and the use of high-strength concrete and high-yielding steel is necessary in order to achieve a performing design. Other design guidelines are highlighted too. Figure 2a shows the optimized Preflex cross-section. Significant economic advantages are found when strict limitations are imposed to deflection (e.g. in the case of three-or-more-span viaducts) but some evident limits of this technology suggest the use of pretensioning together with predeflection to prestress the concrete bottom flange (Flexstress). In this case the precasting phase becomes more complicate and a very important additional design variable, that is the amount of pre-strained cables, is introduced and has to be optimized too. Nevertheless special structural steel is no longer needed and the material savings seem to be largely rewarding for the precasting complications. Figure 2b shows the optimized Flexstress beam.

Finally the Preflex and Flexstress solutions are compared to a traditional composite design with the same slenderness ratio and to a much less slender existing composite deck. Market prices are introduced and all the structural deck components are taken into account. This economic analysis is affected by several uncertainties since the concrete bottom flange precasting costs are very difficult to estimate. Further investigations on this issue would be very useful. Nevertheless the present results show how Preflex and Flexstress technology combined with the use of modern materials can be very interesting for some practical applications.

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Characteristics of 3-D FRP sandwich panels for transportation infrastructure

E.M. Reis & S.H. Rizkalla
North Carolina State University, Raleigh, NC, USA

1 INTRODUCTION

The use of sandwich structures is rapidly growing to construct cost effective, low weight and high performance structures. New materials typically bring new challenges to designers utilize these new materials. Conventional sandwich structures consist of a metallic skin sheets and polyurethane foam core material as an insulating layer. These conventional structures have been used very widely for structural components. However, the durability of those panels become degraded due to delamination at the bonding interfaces, corrosion of the face sheets and stiffness degradation through aging of the foam core over time (Zenkert 1997). This paper introduces an innovative use of glass fiber reinforced polymer (GFRP) sandwich panels as an alternative and solution to these problems. The 3-D FRP sandwich panels presented in this paper consist of two GFRP laminates separated by a foam core where top and bottom GFRP layers are connected together with through thickness fibers as shown in Figure 1. The use of light-weight foam core serves to place the stiffer GFRP face sheets further from the neutral axis and therefore increasing the flexural stiffness and strength. The through thickness fibers increase the shear stiffness of the panel and delay the delamination between the plies of a composite laminate. The panels are fabricated using pultrusion and the through thickness fibers are injected during the pultrusion process.

2 MATERIAL PROPERTIES

Fundamental material properties in tension, compression, flexure and shear are evaluated experimentally.

Tension specimens, having different number of plies and different configurations of through thickness fibers, were tested according to ASTM D3039 to evaluate the in-plane tensile properties of the face sheets of the various 3-D FRP sandwich panels.

Shear specimens were tested in shear to evaluate the influence of the through thickness fibers on the core shear modulus of the 3-D FRP sandwich panels. Shear tests were conducted according to the ASTM C273 for sandwich panels.

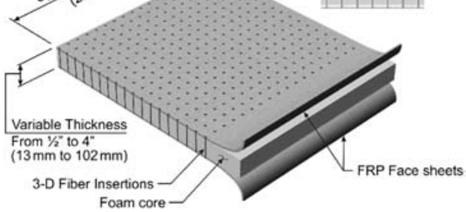

Figure 1. Schematic of 3-D FRP sandwich panel.

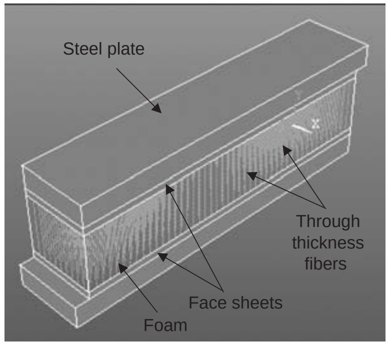

Figure 2. Finite element modeling of shear test.

The flatwise compressive properties of the sandwich panels were determined according to the ASTM C365.

FRP sandwich flexural specimens having different through thickness fiber densities, patterns, and different thicknesses were tested according to ASTM C393. For each type of the 3-D FRP sandwich panel, several specimens were tested using variable span length. The elastic modulus and the shear modulus were determined by the simultaneous solution of the complete deflection equation for each span.

3 ANALYTICAL STUDY

Two sandwich theories were discussed on the behavior of FRP sandwich panels, elementary sandwich theory and advanced sandwich theory. The elementary sandwich theory follows the same principle of the beam theory, it accounts for the shear stress and shear deformations. Based on advanced sandwich theory the local bending stiffness of the faces has an effect on the shear deformation of the core. The faces reduce the shear deflection at the expense of the additional bending moments and shear forces induced into the faces. Analysis results showed that prediction based on the advanced sandwich theory matched very well with the test data and provided better results than the prediction based on elementary sandwich theory.

The shear behavior of the sandwich panel was modeled using ANSYS, a finite element analysis software, as shown in Figure 2. The objective is to determine the initial shear modulus of the 3-D sandwich panel for a given overall and facing thickness, through thickness fiber density and configuration. Based on the analysis in this study it could be concluded that the finite element model is capable to predict the linear part of the behavior prior to cracking of the foam.

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Mechanical properties of HPC and SCC cured in mass structures

M. Kaszynska
Technical University of Szczecin, Poland

ABSTRACT: The objective of this paper is to present the results of analysis of hydration heat and mechanical properties (compressive strength, tensile strength and modulus of elasticity) of early age high performance concrete and self-consolidating concrete, hardened inside of massive structures.

The construction of bridge substructures, including foundations, abutments and piers, often involves the large volume of concrete. Bridge owners become more aware of maintenance and repair problems and there are more strict requirements regarding durability and long-term performance. Therefore, there is a need to identify the most efficient strategies for control of long-term performance of materials. The quality of concrete often depends on its early age characteristics. Therefore, this study is focused on early age properties of new generation high-performance concrete materials.

The research results presented in this paper are related to HPC and SCC mixtures made using Portland cement CEM I 42.5 (European Standard EN 197-1:2000). For the considered mixtures, a superplasticizer and pozzolanic additives (fly ash or silica fume) were used. Two different superplasticizers were considered: modified polycarboxylic ether Sika ViscoCrete 3 for SCC and Sikament 400/30 for HPC.

The measurements of the self-heating temperature of concrete carried out during construction of structures of this type indicate that inside concrete blocks exist quasi-adiabatic curing conditions. In view of this, the assessment and analysis of cement hydration heat in concrete and its compressive strength, tensile strength and modulus of elasticity were carried out in separate investigations in adiabatic conditions. This gave an upper assessment of the values of heat emitted in the concrete. The measurements were carried out in a special calorimeter apparatus, the schematic diagram of which is shown in Figure 1.

To achieve the highest possible accuracy in measuring the temperature rise in the concrete during curing, the cylindrical specimens with diameter $\Phi = 240$ mm and height $h = 300$ mm were taken for investigating the heat of hydration. Upon stabilization of the temperature within a concrete sample, it was slowly cooled down to the ambient temperature, at 20°C.

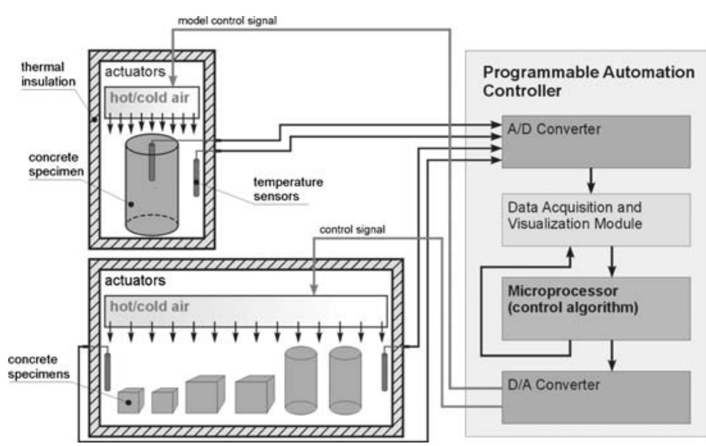

Figure 1. Schematic diagram of the calorimeter apparatus.

Table 1. Test results of mechanical properties of HPC and SCC.

Age [days]	Compressive strength f_{cm} [MPa] lab 20°C	adiabatic 20°C	adiabatic 35°C	Tensile strength $f_{ctm,sp}$ [MPa] lab 20°C	adiabatic 20°C	adiabatic 35°C	Modulus of Elasticity E_{cm} [GPa] lab 20°C	adiabatic 20°C	adiabatic 35°C
Self-consolidating concrete									
1	6.49	29.7	46.3	0.27	2.51	3.03	19.48	29.45	–
3	49.7	60.7	68.7	4.01	3.97	4.82	31.39	33.55	36.23
5	56.53	65.0	75.5	4.18	4.39	4.94	32.31	34.67	–
28	61.8	72.1	76.0	4.88	5.50	5.08	35.40	34.09	37.38
High-performance concrete									
1	36.3	54.9	58.2	3.48	4.58	3.99	25.56	33.73	36.08
3	57.13	80.93	68.0	4.33	4.63	4.16	29.94	34.85	32.80
5	63.85	83.97	69.43	4.50	5.07	4.60	32.49	33.93	–
28	85.77	87.0	72.37	5.60	5.84	4.90	35.81	32.25	33.36

Compressive strength, tensile strength and modulus of elasticity were tested on cubes and cylindrical samples stored in the calorimeter container at a temperature controlled by the temperature of the sample used for testing hydration heat in adiabatic conditions. The mechanical properties of concrete were determined after 1, 3, 5, 28 days of curing in conditions which simulated the curing conditions in a massive structure and in laboratory conditions

On the basis of the temperature vs. time curves recorded during the tests, the amounts of heat of hydration of cement in concrete Q(t) and the values of the source function W(t) (rate of heat evolution) were calculated. In case of HPC the source function W = dQ/dt reaches its maximum values in mixtures prepared at 35°C but the greatest amount of heat is generated during cement hydration in the mixtures prepared at 10°C. In case of SCC the source function W = dQ/dt reaches its maximum values after longer time than in the case of the HPC. The large amount of retardation is observed in the SCC than in the HPC.

A comparison of the results of the compressive strength, tensile strength and modulus of elasticity of HPC and SCC of samples cured in laboratory conditions (lab) and in conditions simulating the actual hardening process in massive structures (adiabatic) for initial temperature $T_o = 20°C$ and $T_o = 35°C$ are presented in Table 1 .

Investigations indicated a large influence of the initial temperature of the concrete mixture on the evolution of the heat emission process and on the growth of mechanical properties in high-performance concrete and self-consolidating concrete.

After 28 days of curing under adiabatic conditions, the HPC show similar values of compressive strength, tensile strength and modulus of elasticity for initial mix temperature $T_o = 20°C$. The highest initial temperature of concrete mixture $T_o = 35°C$ worsens the hydration process and reduces the subsequent concrete strength. In view of the influence of self-heating in high-performance concrete on its strength in structures, it is most advantageous to use concrete mixtures with their initial temperature lowered and concreting during low ambient temperatures.

In the case of SCC with a high amount of fly ash, the higher values of mechanical properties after 28 days of curing in the initial stage under conditions simulating curing in massive structures has been observed. A high temperature of the self-heating influences positively the fly ash reactivity and in the course of the pozzolanic reaction in self-consolidating concrete, a higher strength increase rate of concrete is observed. This suggests a higher activation energy for the pozzolanic than for the cement hydration reactions.

The sustained period of the elevated temperature action in the course of setting and hardening of concrete in mass structures is a decisive factor affecting the development of mechanical properties of concrete at an early-age, and also its mechanical properties at latter ages.

On the basis of the results obtained, the relationships between the quantity of the hydration heat emitted in concrete and its strength at a given moment of setting have been determined.

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Durability design criteria for the Reno Bridge

G. Furlanetto, L. Ferretti Torricelli & A. Marchiondelli
SPEA Ingegneria Europea s.p.a., Milan, Italy

ABSTRACT: For a long time the upgrading of the Italian highway network both at National and International level is in progress, by focusing the attention on the global quality of the infrastructures. In addition, a growing sensibility towards the achievement of durability requirements has been developed in both design and management of new constructions, as well as in developing and improving repair and rehabilitation techniques of existing structures. The importance of the durability performance is particularly emphasized when essential maintenance interventions should be avoided during the expected service life, as required for strategic structures like bridges and viaducts.

In according with this recent trend, the Reno bridge design (Figure 1), in the respect of several constraints of different nature, developes with the objective to provide a structure that is mechanically efficient, safe, cost saving, technically feasible, durable, and also complying with more general requirements, like aesthetics, human comfort and acoustic needs.

The main aspects that contribute to characterize the Reno bridge design consist in the choice of a continuous concrete box girder beam for the deck, the adoption of the incremental launching for the construction method and, finally, the realization of the V-shaped framed system for passing over the river. In addition, a specific maintenance program will be also applied as usual for the standards of the owner Autostrade per l'Italia. In the paper the design choices related to materials, structural scheme and construction process of the Reno bridge are described in detail by highlighting their crucial role in achieving high level durability requirements. In the following some of these aspects are briefly recalled.

The Reno bridge (Figure 2) is made by a curved continuous prestressed concrete box girder deck subdivided in nine spans for a total length of 572.75 m. In the first six spans ($32.75 + 5 \times 45$ m), the deck is simply supported by reinforced concrete vertical piers. For the remaining three spans ($90 + 135 + 90$ m), the bridge consists of a framed system where the deck is clamped to two V-shaped piers, each one formed by four inclined arms converging in the same foundation. Each arm is realized by an outer steel shell with elliptical cross-section, properly strengthened by internal ribbed trusses and filled by reinforced concrete.

For the construction the incremental launching method is used (Figure 3). The choice of adopting this method allows to achieve several advantages. In fact, the construction process, carried out completely without falsework, allows to pass over the Reno river and the close main road without problems. The concentration of the fabrication yard behind one abutment also guarantees the highest

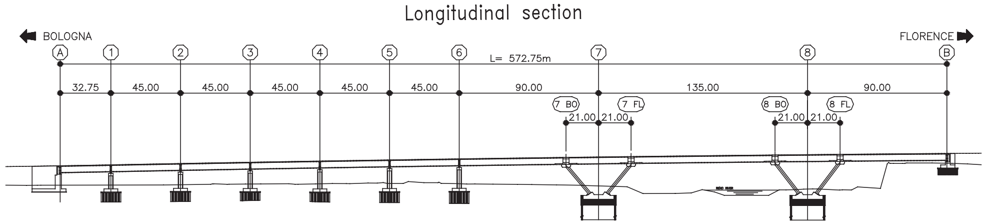

Figure 1. Longitudinal profile of the Reno bridge.

Figure 2. Views of the Reno bridge.

Figure 3. The Reno bridge during launching.

standards of quality by highly mechanizing the construction process, minimizing transportation distances and so allowing time and cost reduction.

In addition, the box girder deck solution has several advantages with reference to durability requirements. In particular, the continuous superstructure allows to facilitate the internal inspections of the box girder in order to control any eventual visual anomaly. The deck longitudinal prestressing consists of post-tensioned tendons injected by vacuum grouting technique and characterized by length of 45 m for the typical spans and by a maximum length of 135 m for the long spans. The use of oversize tendons allows to minimize the number of anchorage zones.

Finally, the design criteria which motivated the choice of the particular V-shaped framed scheme for passing over the river bed mainly emerged from the need of complying with several constraints of different nature, as well as client imposed performances, and more general satisfaction requirements related to users' comfort. In particular, besides to structural efficiency, special attention has been devoted to bridge durability in order to easily allow ordinary preventive maintenance interventions and to avoid the essential maintenance ones.

Measurement and monitoring

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Suitability of portable electrochemical techniques for determination of corrosion stage of concrete structures in on-site conditions

R. Bäßler, A. Burkert & G. Eich
Federal Institute for Materials Research and Testing – BAM, Berlin, Germany

ABSTRACT: Within several EU-Projects different monitoring systems have been evaluated regarding their suitability for reduction of the inspection and maintenance costs as well as the traffic impairments.

They partially dealt with portable techniques for assessment of reinforcement corrosion. During this work potential field mapping and a portable equipment based on the galvanostatic pulse method (GPM) were tested and compared in different situations at laboratory and on-site conditions.

This paper presents results and analysis of the GPM measurements performed in laboratory and on-site conditions in comparison to results of potential mapping. Additionally results of average corrosion rates determined by weight loss and galvanostatic pulse technique were compared. Special attention was paid to the comparability of instrument readings to real behavior. Finally the necessary precautions which need to be taken when the on-site data are used for service life prediction of structures are discussed considering environmental and technical effects.

Generally steel is protected permanently against corrosion by the alkaline pore water environment. At unfavorable conditions (carbonation, chloride ingress) the passive layer on the steel surface can be destroyed. First the resulting corrosion products could be incorporated in the pore structure of concrete without significantly visible changes at the concrete surface. Later secondary damages caused by corrosion, like cracks and delaminations at the structure, can occur due to volume increase of corrosion products.

In order to initiate necessary rehabilitation measures at the right moment from the safety aspect as well as the economic point of view non-destructively determined information on the current corrosion behavior of the reinforcing steel have a high importance. For that various electrochemical measurement techniques are available, like measurement of corrosion potential determination of short-circuit currents on corrosion cells, and external controlled electrochemical investigations.

Because in passive conditions the rebar potentials can fluctuate in a wide range depending on different parameters. In some potential areas a clear classification of active (corroding) or passive conditions is not possible. In such cases measurements by Galvanostatic Pulse Method (GPM) might be supportive. By this technique a short anodic DC-current impulse is applied to the reinforcement using a counter electrode. Evaluation of the potential change, recorded simultaneously at the steel, enables a much better classification regarding the situation at the reinforcement. However the applicability is being discussed controversially.

By measurements in laboratory conditions a very good evaluation of the reinforcement corrosion behavior could be shown. In potential ranges, where a clear assignment to passivity or activity could not be made, a better evaluation could be achieved using galvanostatic pulse measurements (GPM). The comparison of results obtained by regular pulse measurements for a long time period to the measured mass losses shows a sufficient correlation. It can be stated, that GPM is suitable for evaluation of the corrosion rate on small specimens. Due to the unknown actually corroding area severe misjudgments of the corrosion rate can result, if a clear localized corrosion attack occur.

On the test plate also a good qualitative evaluation of the corrosion stage could be achieved using potential mapping. Here it turned out that relatively small corrosion spots only can be detected by a narrow measurement grid.

The detection of polarization resistance by GPM did not lead to meaningful results. An evaluation of corrosion behavior is therefore impossible. So the usability of GPM in on-site conditions is not given. Only for the determination of concrete cover resistance the GPM can be used. Here the results could clearly be related to the initialization spots, and a good reproducibility of the results could be achieved. As expected, the results were not essentially influenced by the pulse current height.

Concluding it can be stated that potential mapping can be performed independent on the moisture situation on the surface. However at each measurement point uniform conditions needs to be assured. Measurements on almost dry surface only allow an evaluation of potential differences but not of the reinforcement potential itself. If the real reinforcement potential should be evaluated a sufficient moisturization of the surface has to be provided. The achievement of stable potentials has to be checked by permanent potential measurements before starting the actual potential mapping. Usually 20 to 30 minutes should be sufficient. During the whole measurement a uniform moisturization situation has to be provided. For longer measurements, where quick drying out could occur, a remoisturization might be necessary.

For GPM-measurements stable conditions by a sufficient moisturized surface is absolutely essential. Such measurements are only meaningful if it can be assured, that no potential shift occur during the measurement. A complete moisturization of the whole surface is not required. Only the respective measurement point has to have potential stability and comparable conditions.

Due to the differences of gathered own and literature results it is necessary to evaluate the limitations on real structures. Right now there is a big skepticism in terms of correct applicability of corrosion rate measurement devices. These techniques are being evaluated on park decks and bridges as well as in special laboratory approaches.

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Detecting wire breaks in a prestressed concrete road bridge with continuous acoustic monitoring

S. Fricker & T. Vogel
Institute of Structural Engineering, ETH Zurich, Zurich, Switzerland

ABSTRACT: In recent years much experience has been gained with continuous acoustic monitoring of monostrands in flat slab buildings and cables of cable stayed bridges. On the other hand there is little experience with monitoring of post-tensioned bridges with bonded tendons due to the complexity of the structures and the lower energy released in a wire break. To evaluate the diagnostic performance of acoustic monitoring taking of bonded tendons the monitoring system SoundPrint has been installed on a small road bridge with two spans of 30 m length each. This bridge, the Ponte Moesa was built in 1952 and is one of the first prestressed concrete bridges in Switzerland. In the longitudinal direction it is prestressed with 84 continuous tendons and 21 additional tendons at the middle support. Insufficient grouting and de-icing salts among other causes have led to deterioration and wire breaks.

1 INTRODUCTION

Acoustic monitoring can help the engineer to assess the present status of a bridge and can also assist in deciding whether costly maintenance is required. Until recently, continuous acoustic monitoring of large structures was not practicable because of the high costs. Nowadays, the development of powerful and cheap computers, sensors and software as well as fast data transfer through the internet enable new potentials. The monitoring system SoundPrint was installed on the Ponte Moesa, a small bridge in the south of Switzerland to evaluate the use of continuous acoustic monitoring in practice.

The bridge as well as the installation and calibration are described. In the following sections the detected signals, the monitored spontaneous wire breaks and the blind testing are discussed. Then the acoustic monitoring is compared with conventional invasive examination and half-cell potential measurement to evaluate the applicability of acoustic monitoring to bridges.

2 OBJECTIVES

In recent years much experience has been gained with continuous acoustic monitoring of monostrands in flat slab buildings and cables of cable stayed bridges. However, there is little experience with monitoring of post-tensioned bridges with grouted tendons. The released energy in fully or partially grouted tendons is likely to be smaller and therefore events are more difficult to detect than wire breaks in unbonded tendons. To evaluate the diagnostic performance of continuous acoustic monitoring for grouted tendons, a monitoring system was installed on a small road bridge and has been in operation since June 2004.

The aim of the installation is to test the diagnostic skills of the system on an existing object with ambient noises, e.g. from traffic.

Additionally, gaining experience for the adequate future use of permanent acoustic monitoring on different types of bridges is an important objective.

3 PILOT-OBJECT PONTE MOESA

The Ponte Moesa, built in 1952 in the canton of Grisons, Switzerland, spans the River Moesa and each of the two spans is 30 m long. The cross-section shows a hollow-core slab with a depth of 0.70 m at midspan and 1.30 m at the middle support.

In the longitudinal direction, the bridge is prestressed with 84 tendons. Every tendon contains 12 wires of 7 mm diameter each. There are 21 additional tendons above the pier. Insufficient grouting and de-icing salts, among other causes, have led to deterioration and wire breaks. During a periodical inspection in March/April 2001, out of 25 inspected ducts 13 were not grouted at all, 9 were poorly grouted and only three were in a proper condition. Some of the wires were completely corroded.

4 INSTALLATION AND CALIBRATION

The SoundPrint system was developed in Canada; in North America the system is commercially available to monitor monostrands in flat-slab buildings. Sensors, attached to the structure, detect and localize acoustical waves induced by wire breaks and transferred through the structure. Since these waves have distinguishable characteristic features, irrelevant data can be filtered out. The analysis and interpretation are performed using problem-oriented software. The localization of the events is done by geometrical triangulation. This enables a visualization of wire breaks and their clustering and thus to evaluate the deterioration process of the bridge.

The signal from each sensor is recorded whenever a pre-defined threshold is surpassed. Then the software filters, stores and transfers the data to the analysis center in Vélizy/Paris via the internet.

During the calibration phase, the hardware is tuned to give a corresponding and predetermined response. Using the calibration impacts, both the actual location (as measured on site) and the calculated location (from the signals) are compared to get an idea of the location accuracy of the set-up.

During data processing there are three levels of filtering. The first level is the threshold that sets the minimum value for triggering. The second level is based on a real time analysis of the characteristics of the signal by the software of the acquisition unit and leads to automatic classification. This eliminates obviously unwanted data, so that only data of probable interest is sent to the processing center. The third level, extracting the data of interest, is done by specialists in the data processing center in Vélizy.

5 BLIND TESTING

The blind tests on fully grouted tendons were carried out by the Institute of Structural Engineering of ETH Zurich. To artificially create wire breaks for the blind testing an electrolytic corrosion-cell was developed. This made it possible to create wire breaks without any announcing noises from cutting or grinding. The basic idea of the corrosion cell is to create a saline environment where forced by the voltage of the battery an accelerated corrosion takes place. To corrode only one wire, wedges were driven into the bundle to separate a single wire.

Two blind tests were conducted. Depending on the applied current to corrode the wire up to failure, it took 22 to 29 hours.

All detected and located wire breaks during the monitoring are shown and listed by date. It is remarkable that apart from the wire breaks No 2 to 4 and 15 all events concentrate in one area. Comparison of Monitoring and Other Inspection Methods.

From the results of a visual inspection, the wire breaks would have been expected in the region with bad concrete surface and honeycombing. Quite in contrary to the expectations the wire breaks occurred in a zone with a proper concrete surface.

On the other hand the results of a half-cell potential measurement made in October 2004 show that the localized wire breaks match with the results of the half-cell potential measurement. Most of the wire breaks occurred in the regions with increased or high likelihood of corrosion.

To confirm the detected wire breaks, two invasive inspections were executed during monitoring. The location and size of the openings were determined from the monitoring. Three wire breaks with bright fracture surface in a highly deteriorated zone were found. In the opening, two tendons, containing 24 wires altogether were in a severe condition. Only three wires with reduced cross-section were still acting, all the others were broken.

6 RESULTS

During the monitoring from June 2004 to November 2005, 13 spontaneous wire breaks have been detected, classified and localized. Three of those were confirmed by an invasive inspection. The accuracy of the localization for the spontaneous wire breaks ranged from 10 to 60 cm.

Additionally, two artificial wire breaks in fully grouted tendons were performed by catalytic corrosion cells as blind tests. Both were detected, classified and localized. The accuracy of the localization for the two artificial wire breaks ranged from 16 to 66 cm, respectively. Also irrelevant signals and ambient noises such as noises from expansion joints, bearings, traffic, construction activity and other ambient sources were detected, classified and filtered out.

Due to the results of the acoustic monitoring the owner decided to examine the bridge. A half-cell potential measurement was executed and additional inspection recesses opened. Highly deteriorated regions were found. The result of the invasive inspections and the percentage of the remaining cross-section of the wires for each opening will be shown.

As a consequence of the monitoring and the further assessment the owner decided to replace the bridge in 2006. The new bridge will be built on the upstream side. Additional supports in each span were erected in November 2005 as an immediate safety measure.

7 SUMMARY

The diagnostic performance of the continuous acoustic monitoring system SoundPrint was evaluated with blind testing. Even in a noisy environment the system was able to identify and locate wire breaks in fully grouted and partially grouted tendons. Invasive inspections confirmed spontaneous wire breaks that were also detected during the monitoring. The results of half-cell potential measurements matched with the monitoring. Almost all located spontaneous wire breaks concentrated in one area with higher likelihood of corrosion.

On the basis of the data obtained during the monitoring, it can be concluded that continuous acoustic monitoring is able to record, analyze, classify and locate wire breaks in grouted and partially grouted tendons. It is also considered capable of filtering out of events that are not evident for the structural capacity of the bridge.

8 OUTLOOK

The continuous acoustic monitoring of the Ponte Moesa will be continued. More wires might break spontaneously. The replacement of the bridge provides the opportunity to inspect the bridge by intrusive methods. This will be done by cutting the bridge into pieces and examine the cross sections. The tendons can be accessed and all the detected wire breaks can be confirmed. Furthermore the localization accuracy will be checked and the different influences on the localization will be evaluated. These informations would be difficult to get, if the bridge had to sustain in service.

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Study of masonry arch bridge limit states with acoustic emission techniques

A.K. Tomor & C. Melbourne
University of Salford, Salford, UK

ABSTRACT: Masonry arch bridges form over 40% of the European bridge stock, 60% of which are over 100 years old. With regards to the significantly increasing levels of loading and volumes of traffic, establishing the safe limits of loading are becoming more and more urgent.

Determining the life expectancy of masonry arch bridges is clearly necessary for managing the aging bridge stock. Acoustic emission is increasingly widely used for monitoring concrete and metallic bridges however, its application for masonry bridges is currently very limited due to their heterogeneous nature. The paper reports the use of the acoustic emission technique in an attempt to identify the onset of the ultimate and serviceability limit states for brickwork masonry arches. A series of 3 m span two-ring and 5 m span three-ring arches have been tested at the University of Salford under static and long-term fatigue loading to assess their endurance limit and modes of failure.

Masonry arches can fail a number of ways. Failure can be caused by exceeding the ULS or the SLS at any location of the structure. Instead of having one ULS and SLS for the entire bridge, an ULS and SLS needs to be defined for each structural member under each type of stress condition (e.g. radial compression, radial tension and radial shear). As a result the safe working limit is defined as the SLS of the SLS of the weakest structural member. The acoustic emission technique has shown that break-down of the material can initiate from as low as 20–30% of the ULS, much lower than the generally excepted 50% fatigue limit in design and assessment codes. Further exploration of the USL is necessary.

The acoustic emission technique also allowed the various stages of the failure mechanism to be identified with, e.g. crack development, crack opening and stress redistribution. This may provide useful information in terms of the condition of the structure.

Exceeding the SLS and overloading the arch can result in sudden failure under fatigue loading with no previous indication of damage as has been found by several large-scale tests. This is of particular interest for bridges under traffic loading as no information is currently available of the SLS of such bridges. The recorded AE absolute energy can be able to inform whether the released energy is stable or increasing and imminent failure can be excepted.

In order to represent bridges in the traffic network that have usually suffered some form of damage, acoustic emission technique has also been tested on badly damaged arches. Although distinguishing increased acoustic activity in such cases may be difficult due to excessive background noise, with the help of AE absolute energy data gradual break down of the structure may be identified.

Acoustic emission technology therefore shows great potential to warn of residual damage and crack propagation occurring to masonry arches, even to those in poor condition. The technique shows great potentials for condition assessment and monitoring of masonry arch bridges in the traffic network.

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System for monitoring of steel railway bridges based on forced vibration tests

J. Bień, P. Rawa & J. Zwolski
Institute of Civil Engineering, Wroclaw University of Technology, Poland

J. Krzyzanowski, W. Skoczynski & J. Szymkowski
Institute of Production Engineering and Automation, Wroclaw University of Technology, Poland

ABSTRACT: A team of Wroclaw University of Technology (WUT) is developing a monitoring system based on forced vibration tests. The tests, repeated periodically, have as a main goal to identify and monitor the modal parameters of the structure. It is expected that on the basis of the changes of construction modal parameters some conclusions concerning structure condition can be drawn. Results of the modal tests of bridges can be also used in FEM models updating, in the serviceability and fatigue analyses. As a bridge excitation source a special eccentric-mass exciter has been built and tested in laboratory. A dedicated software created for test control, data acquisition and processing has been also described in the paper.

1 VIBRATION TESTS IN BRIDGE MONITORING

Monitoring of civil engineering structures based on forced vibration tests have been started in early 80' of the last century. Since this time the main aim of the monitoring process is to deliver information to owner about structure condition. It is done by systematic periodic checking of physical parameters of the structure. Changes of modal parameters (natural frequencies, damping and mode shapes) identified from the test data is believed to be good global descriptor of technical condition that can be converted into structural model (stiffness, mass and damping matrices of structure's FEM model).

Civil Engineering Institute together with Production Engineering and Automation Institute of WUT participating in 6th Framework Program of EC titled "Sustainable Bridges – Assessment for Future Traffic Demands and Longer Lives" is working on a monitoring system for railway bridge structures based on forced vibration tests. As a source of excitation an unbalanced rotational mass exciter has been constructed together with a dedicated supporting frame structure to enable performing the tests of railway bridges (Figure 1). Special software MANABRIS has been also designed and implemented for automation of testing control as well as data acquisition and processing.

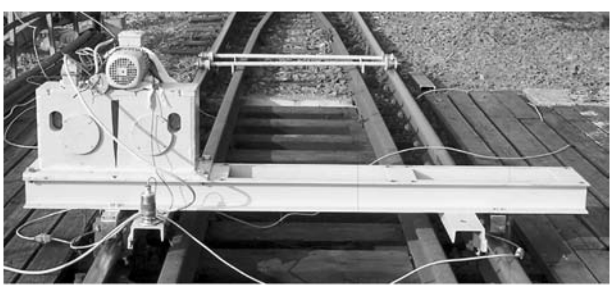

Figure 1. Exciter during vibration test of railway bridge.

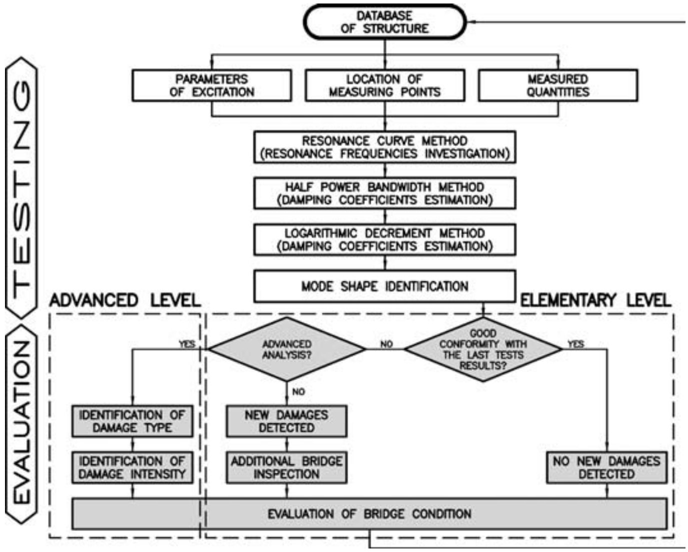

Figure 2. Procedure of bridge condition monitoring.

2 MONITORING PROCEDURES

Monitoring procedure consists of initial test and series of systematic tests during bridge service. The main goal of the initial procedure is to identify modal parameters of the structure as intact or on the beginning of the monitoring system applying.

The main requirement according to the monitoring procedure (Figure 2) is to perform all tests in the same manner and in the same conditions (as close as possible) as the initial test and to check if significant changes in modal parameters of the structure occurred. On the elementary level a basic analysis is only performed to investigate whether the structure is damaged or not. If changes of identified modal parameters are observed the advanced level analysis should be performed in order to locate the suspicious regions of possible damages.

The shaded items in Figure 2 indicate tasks which can be supported by a specialized expert system. After processing two sets of results (from an initial test and from a monitoring test) some conclusions related to changes in technical condition of the analysed structure might be suggested by the expert tool. In this analysis changes of resonance frequencies, changes of vibration amplitudes at the resonance frequencies, changes of mode shapes (and their curvatures) and also changes in damping coefficients should be taken into account.

Presented system is proposed for systematic monitoring of bridge technical condition for fast and relatively cheap identification of deteriorated structures. In case of significant changes of construction modal parameters a system operator will be able to make a decision on additional detailed or special inspection.

3 SUMMARY AND FUTURE WORKS

All components of the developed system for condition monitoring of railway bridges has been verified and tested in laboratory with regard to its usefulness and efficiency in the system. The tests have been also carried out in order to confirm the system stable and failure-free work.

To verify the stability of the system working in field a pilot test of steel railway bridge have been conducted. Results of the test showed efficiency of the exciter in inducing stable vibration of the structure and proved usefulness and safe work of the control software MANABRIS. Field test showed also needs of improvements in the exciter construction as well as in functionality of the control software.

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Wavelet-based impact acoustic method for detecting interfacial separation of steel-concrete composite bridge

B.F. Yan
Institute of Bridge Engineering, College of Civil Engineering, Hunan University, China

A. Miyamoto
Department of Computer Science & System Engineering, Yamaguchi University, Japan

X.Y. Zhou
College of Civil Engineering, Central South Forestry University, China

ABSTRACT: NDT techniques such as impact-echo and impact acoustic available for the integrity assessment of concrete bridge can be employed to locate void, cracks, honeycombing, and delamination in concrete decks and slabs, as well as other concrete structure. These two methods have the similar theoretical background, i.e., both of them are performed by introducing a stress pulse impact source (such as a ball drop or hammer strike) at the surface of a structure and monitoring the stress waves with a transducer (impact-echo method) or microphone (impact acoustic method). Stress waves propagate into the structure and are reflected by the external surface and existing voids or delaminations, and thus to create a resonance condition which can be observed by a spectrum analysis of the acquired waveforms. The resonance frequency provides critical information about the structure geometry, soundness of the structure, and the acoustic behavior of interfaces between materials (Sansalone et al 1989; Sansalone et al 1997). One advantage of these two methods is that access to only one surface of a structure is needed, making them suitable for rapid assessment of structure.

The processing of the stress waves is generally implemented in the frequency domain. As a method for evaluation of the relative integrity degree of concrete structures, the relative change in the frequency content of the measured signal compared with the signal from the sound parts of the structure indicates the presence of a fault (such as void and interfacial delamination), variation of structure geometry, or a modification in concrete characteristics, or all three. Other than frequency domain property, some other soundness indices (such as amplitude ratio between input and output, averaged center frequency) are developed and implemented via statistical analysis to extract valuable information for evaluation (Haya et al 2003; watanabe et al 2004). Practically, there are numerous factors (such as interface condition, material property, local vibration of measured specimen, environmental noise contamination) affect the identification accuracy.

For the purpose of extracting valuable information representing the structural integrity from the acoustic signature, more advanced analysis model and signal processing technique require careful dealing. The study places emphasis on using the impact acoustic method based on wavelet transform to detect the interfacial separation of the steel-concrete composite slab. a time-frequency-energy wavelet spectrum for a real signal derived from an impact test on the steel-concrete slab with a separation (void) at steel-concrete interface is given. As shown in Figure 1, a simplified conceptual model can be constructed by categorizing the frequency contents into three parts, i.e. *zone A, zone B, and zone C*:

Zone A: reflecting characteristics of the boundary condition of the part of the concrete slab. The echo of the *P*-wave generated from the reflection by the bottom of the slab, and the resonance frequency indicating the slab thickness can be explicitly observed from the wavelet spectrum. Note that if there is inner void in the concrete slab part, the corresponding resonance frequency can be found in the higher frequency zone.

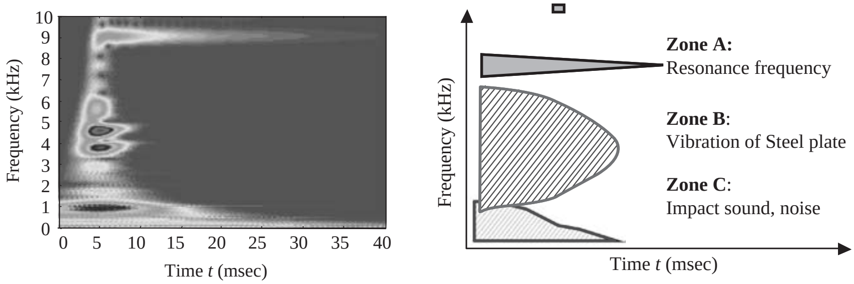

Figure 1. Illustration of the simplified conceptual model for the impact acoustical signal processing based on wavelet transform.

Zone B: representing the local vibration response of the thin steel plate where void or interfacial separation exists. The frequency content of the steel plate under impact excitation is depended on its size and boundary condition.

Zone C: located at the low frequency zone. Including all kinds of low frequency sound information such as environment noise contamination, low frequency part of the impact acoustics caused by shock between the hammer and steel-concrete composite slab.

From the wavelet spectrum, the signal characteristics in the time and frequency domains and its energy distribution can be extracted as the features for further pattern classification. The frequency domain, as above proposed, can be separated into three characteristic zones. The time domain information indicates the duration time of the decaying frequency components, which to some extent reflects the amplitude of impact force, material properties, internal flaw type, etc. In this study, the energy distribution of the frequency contents at zones A, B, and C, as an effective indication index, is extracted from the wavelet spectrum and adopted as the input to the classifier for training and testing.

A field experimental test on a composite slab was conducted for verification. The artificial interfacial separation between the steel and concrete is simulated by styrene foam block and rubber sheet with different sizes and embedded in steel-concrete interface before casting concrete to simulate the interfacial separation and water invading. It is concluded that: a) the resonance frequency of *P*-wave and its decay time history can be easy identified from the wavelet spectrum. b) the energy distribution of the impact acoustic signature is deeply affected by the size and type of the embedded material. When the installed material is rubber sheet (simulating water invading) or the size of the styrene foam block (void size) is so small that after impact the vibration frequency of the corresponding local steel plate are beyond 10 kHz, the steel-concrete interface separation can not be detected, and the energy distribution is similar to the intact specimen. Moreover, it is difficult to distinguish the difference between the intact specimen and the rubber sheet invaded specimen from the frequency domain. c) the frequency components distribution (or density) is a valuable indicator that contains information about the size of the local vibration plate, which indirectly reflects the extent of the steel-concrete interfacial separation.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

A neural-network-based system for Bridge Health Monitoring

T.K. Lin
National Center for Research on Earthquake Engineering, Taipei, Taiwan

K.-C. Chang
National Taiwan University, Taipei, Taiwan

C.C. Chen
Yunlin University of Science and Technology, Yunlin, Taiwan

C.Y. Chen
National Center for Research on Earthquake Engineering, Taipei, Taiwan

I.J. Tsai
National Taiwan University, Taipei, Taiwan

ABSTRACT: Earthquake has been the most threatening disaster to civil structures. The damage of infrastructures not only causes the loss in economic activities but the life and properties of people. For that reason, research on applying instrumentation on structures to monitor their characteristic during and after earthquakes is always an important issue. By the development of system identification in monitoring the vibration trait of buildings and bridges, some theories and methods has gradually become matured in the last few decades.

In order to record the earthquake data in Taiwan, the Central Weather Bureau (CWB) of the Ministry of Transportation and Communication (MOTC) has instrumented sensors in free field, buildings and bridges among Taiwan to record their response during major earthquakes. For the part of bridges, sixteen bridges have been chosen as the demonstration examples and a safety monitoring and seismic evaluation system has been established. The database collected by the system can offer abundant information for researchers. The practical situation of bridges can be reflected by the data and a proper modification of design code can be determined.

Traditionally, the identification methods were developed under frequency domain. However, two close frequency modes can not be effectively separated by frequency-domain-based method when noise is contained in the measured data. To solve this problem, the discrete-time domainbased identification technique has been applied to civil engineering in the last two decades. In most methods, structures are considered time-invariant. Namely, parameters of structures are assumed to be constant during the whole time history.

However, structures might be damaged or with nonlinear behavior during earthquake. To this goal, neural networks, proved to have outstanding performance in complex problems, was integrated into identification systems. The adaptability and fault tolerance of neural networks have made them good candidates in dealing with data of uncertainty and incompleteness and identification of nonlinear systems under major earthquakes may be implemented by neural networks (Adeli et al. 1995) (Masri et al. 1992) (Masri et al. 1993).

A bridge health monitoring system based on neural network technology is proposed in this paper. In order to identify the nonlinear behavior of structures, a NARX system is trained from data collected in major earthquakes. The relationship between the input and output channel can be reflected by the weighting of the neural network and the fundamental period of the structure can then be derived. By applying the system to bridges, the multi-support characteristic can be analyzed and the combination of specific frequencies causing resonant phenomenon can also be

obtained. The result would be an important basis for verifying the source of damage behavior on structures.

To demonstrate the performance of the proposed system, a bridge in the second southern freeway in Taiwan is used. By data collected from two large ground excitations, the NARX system with a structure of two input nodes and one output node is established to evaluate the property of the bridge. The input channels are the signals from the pile-cap and the output is the response of the web on the middle span. Analytical results of different methods including transfer function, ARX model are also compared with the proposed neural-network-based system to evaluate their efficiency in health monitoring.

The result has shown that besides identifying the fundamental frequency of structure, the proposed neural-network-based system can also be successfully applied in bridge health monitoring after major earthquakes. The combination of specific frequencies causing resonant phenomenon is clearly shown in the kernel transformation diagrams and damage on structures could be alleviated. More information can be studied from the complex high-order kernel transformation.

The capability of the NARX system in dealing with nonlinear structure would be another research focus. By the proposed method, bridges with nonlinear bearing such as lead rubber bearing or visco-elastic dampers can be precisely monitored during major earthquakes. The behaviour of these elements under specific time history can be used to assess the performance of these equipments and the bridge design code can be revised basing on the practical condition of bridge structures.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Distributed strain measurement in steel slab-on-girder bridge via Brillouin optical time domain reflectometry

F. Bastianini, F. Matta, N. Galati & A. Nanni
Center for Infrastructure Engineering Studies, University of Missouri-Rolla, MO, USA

ABSTRACT: Fiber optic sensors are emerging as effective alternatives for the health monitoring of civil structures. Distinctive advantages over conventional electronic sensors are the immunity to electromagnetic interferences, the dielectric performances, the high survivability in aggressive environments and the high degree of miniaturization.

Single-mode fiber optic sensors based on spontaneous Brillouin scattering add the unique feature of measuring distributed strain and temperature profiles along structural members. Measurement is based on the correlation of strain and temperature with the frequency shift of the Brillouin backscattered light induced by a pump light pulse launched into an optical fiber. The technique has significant potential for the structural health monitoring of bridges and, in perspective, of integrated transportation infrastructures. However, very few field applications have demonstrated its feasibility on large-scale structures.

The paper presents a pilot application of Brillouin Optical Time Domain Reflectometry (BOTDR) for the extensive strain measurement along four steel girders of a major highway bridge located in Osage Beach, Missouri, USA. Bridge A6358 is built with five continuous symmetrical spans: the two external are 44.8 m and 56.4 m long, respectively, while the central one has a length of 61 m, resulting in a total bridge length of 263.4 m. Each internal support consists of reinforced concrete (RC) bents supported by two RC circular piers having a 1.8 m diameter. The cross section comprises five composite, equally spaced, steel I-girders acting compositely with a 216 mm thick RC deck, with an out-to-out deck and clear roadway width of 12.4 m and 11.6 m, respectively.

Measurements were made during a diagnostic load test that was conducted to assess the safety of the newly constructed bridge while acquiring the distributed girder strains under controlled load conditions. In addition, deflections at discrete locations were measured using a high-precision, non-contact automated total station (ATS) system that was previously evaluated with respect to Linear Variable Differential Transformers (LVDTs) in similar applications.

A 1159 m long optical circuit was installed on the web of four girders at different depths, along up to two continuous spans, using an on-purpose designed aluminum/FRP inspection cart moving along the bottom flanges of two adjacent I-girders. The sensing circuit comprised bare optical cables and a custom-made "smart" glass Fiber Reinforced Polymer (FRP) tape with embedded optical sensors for strain measurement and thermal compensation. The specific correlation coefficients between the Brillouin frequency shift and the strain/temperature in each fiber type were preliminarly evaluated in the laboratory. The optical attenuation of the BOTDR circuit was kept within a value of 6 dB in the first 1026 m, thereby enabling to measure strains with a minimum accuracy of $\pm 40\,\mu\varepsilon$ on a length resolution of 2 m, using a AQ8603 optical strain analyzer unit set for 20 ns laser pulses. During the diagnostic load test, the data acquired from the AQ8603 unit were processed in real-time through a proprietary software in order to compensate for thermal deformations.

Three-dimensional linear-elastic finite element analysis was performed to estimate the theoretical response of each girder. The numerical results approximate the ideal bridge response assuming full contribution of secondary structural members to the transverse load distribution, thus resulting in lower bound strain profiles. Hence, the FEM was calibrated based on the results of ATS deflection measurement in order to provide a reliable benchmark to evaluate the performance of the BOTDR

system. Upper bound strain profiles were also computed via one-dimensional beam analysis as per the AASHTO LRFD Bridge Design Specifications.

Despite the non ideal working conditions during circuit installation, which may affect the measurement quality, the experimental strain profiles accurately described the global bridge response under typical load test conditions. In case of steep geometrical discontinuities, a higher degree of accuracy was observed at strain levels above 100 $\mu\varepsilon$, which may suggest the use of pre-straining fixtures. The system performance was poor when bare fibers with several sharp bends had to be used, and in the circuit portion where a significant optical attenuation was detected.

The outcomes of this pilot project demonstrate the practical potential of BOTDR sensors systems for the distributed strain measurement in large-scale bridge structures, and support the need of further research to improve and refine the technology, as well as reduce strain analyzer and specific equipment costs.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Data processing for safety control of bridges in real time

V. Marecos
ISEL, Lisbon, Portugal

L.O. Santos
LNEC, Lisbon, Portugal

F. Branco
IST, Lisbon, Portugal

ABSTRACT: This paper describes the developed methodologies for data processing of bridge monitoring data in real time, applied to vertical displacement data on Salgueiro Maia Bridge.

1 INTRODUCTION

The observation of structure's behaviour, especially related to long-term monitoring, has had large improvements with the introduction of new techniques for automatic acquisition of measurements. Automatic acquisition gives the possibility of reading several equipments continuously, simultaneously, without the presence of operators, plus the capability of establishing a remote connection to a centre of studies through phone, radio, GSM or Internet.

For a better use of the automatic acquisition and considering the new developments associated to sensors based on recent technologies, and considering also the upgrading of storage and acquisition equipment, it became necessary the development of new methodologies for data processing.

For the data management and analysis, it is needed to get reliable measurements from each sensor and information on the relations between different sensors and on the evolution of the data with time.

Some of this information is now possible to get directly from the data-loggers with the definition of routines that are introduced on the acquisition program. This way they are done immediately after the readings, and it is possible to repeat some measurements if any anomalous data is detected. These procedures include validation of signals, measurements and tendencies.

On the other hand, to guarantee the observation of the structure in real time, alert and alarm levels have to be established to control, permanently and in situ, the development of selected variables, allowing a more efficient evaluation of safety conditions of the structure.

2 INTEGRATED MONITORING SYSTEM

The Integrated Monitoring System is divided in three sub systems: Data Acquisition and Processing in real time, Data Transfer and Data Management and Analysis.

In data acquisition sensor signals are read and stored with the data-logger which makes it possible to have measurements from all the sensors distributed along the structure simultaneously, allowing the right comparison of the data because they are referred to the same moment in time. Measurement's rate can be specified according to the variability of the data that we are observing. The data-loggers can be linked to each other's building a local network that is commanded by the master data-logger.

After acquiring the data, the data-logger starts the analysis by checking if the value read is under the alert or alarm level and if not, unchains the alert or alarm procedures. Those procedures include

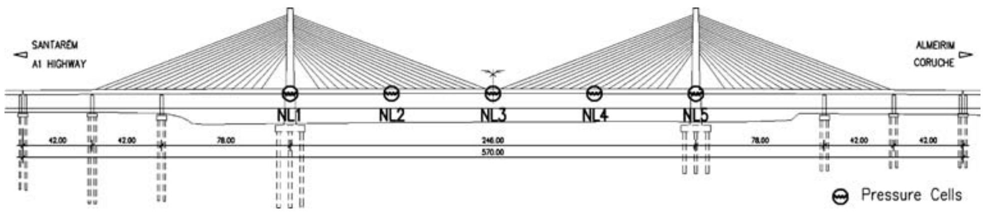

Figure 1. Longitudinal section of Salgueiro Maia cable-stayed bridge and hydrostatic levelling system.

the increase of readings frequencies, but also the sending of messages that informs the technician in charge of the anomalous data, allowing him to do more detailed analysis.

In order to obtain reliable data for analysis it is necessary to validate the records by verifying if they are within the limits defined and eliminating erroneous ones.

The verification of validated records may be done using estimated data from correlations between several measurements or applying measured input of the bridge to the analytic model and calculating the expected value of the variables monitored.

3 SALGUEIRO MAIA BRIDGE

The Salgueiro Maia Bridge is a cable-stayed bridge and has a total length of 570 m divided by seven spans with a main span of 246 m (Figure 1).

The masts, with a height of 50 m above the deck, are in reinforced concrete up to the level of the first cables, and from there on have a steel-concrete composite structure. The two masts are monolithic with the deck. The stay cables support the bridge deck and are anchored at the masts and at the deck axis.

A hydrostatic levelling system associated with pressure cells was implemented to measure vertical displacements of the main span of the bridge deck.

The pressure cells are located at piers P4 and P5, at mid-span and quarter-spans (Figure 1), the deposit is situated at pier P4. The relative vertical displacements are calculated with the variation of cells pressures using cells NL1 and NL5 as the fixed references.

The validation of the data was done using the modified Z-Score test, with S_n for scale estimation. The values marked as an outlier were eliminated and afterwards the median of the remaining records of the sample was calculated.

The vertical displacements of a bridge depend of several factors, such as traffic loads, temperature, rotation of piers and masts, cable forces, creep or shrinkage. The estimation of vertical displacements was done using its correlation with air temperature.

4 CONCLUSIONS

New developments in sensors and the upgrading of storage and acquisition equipment lead to the necessity of new methodologies for data processing.

For data management and analysis procedures it is needed to have reliable measurements from each sensor and also from the relations between different sensors.

This paper shows the efficiency of modified z-score method, with the use of S_n to estimate scale, on the validation of vertical displacement.

With the use of measured air temperature it was possible to forecast the displacements in order to create ranges for verification of measured values.

For future analysis the use of combine correlations between other monitored data and also the use of a finite element model would result on better predictions for measured data that could reduce uncertainty margins.

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New method for detecting & measuring cracks on concrete using fiber optic sensors

A.D. de León & P.J.S. Cruz
University of Minho, Campus Azurém, Guimarães, Portugal

K.T. Wan & C.K.Y. Leung
The Hong Kong University of Science and Technology

ABSTRACT: Advances in the production of optical fibers have made possible the recent development of innovative sensing systems for health monitoring of civil structures. The main reasons for this development are the reduced weight and dimensions of fiber optic sensors, the strong immunity to electromagnetic interference, the improved environmental resistance and the scale flexibility for small-gage and long-gage measurement. This paper provides an overview of the challenges in developing a new fiber optic sensor that can be employed to monitor flexural and tensile cracks on RC structures. The methodology in detecting and localizing the formation of flexural cracks in various locations and sensor's capability in measuring a range of crack widths is demonstrated through testing of instrumented RC beams subjected to sustained and repeated loading.

1 INTRODUCTION

The identification and evaluation of degradation of the structural performance is an important criterion for maintenance intervention on new bridges, and to repair or upgrade those already built. Nowadays in practice, the identification of deterioration in Reinforced Concrete (RC) structures is often executed visually, and when it is necessary a deeper investigation using various instruments will be performed. Usually the principal parameters to be determined in the evaluation of RC structure include: cracking, spalling, deflection, stains, erosion, and corrosion. The cracking can accelerate steel reinforcement corrosion in the concrete structure which is inciting spalling and also stains. In practice, this is the most commonly used parameter in inspections to propose maintenance intervention and preventing moisture infiltration.

In current design procedures attenuation and control of cracking is done for obtaining acceptable appearance and for long-term durability of concrete by maintaining adequate distribution of cracks and reasonable limit on crack widths. In bridge structures cracks need to be repaired if they reduce the strength, stiffness, or durability to an unacceptable level, or if the function of the structure is seriously impaired. In practice the reparation of cracks is done after the cause is established, and determined whether the observed cracks are indicative of current or future structural problems, taking into consideration the present and anticipated future loading conditions. This involves first a field observation to identify the location and extend of cracking, then a review of drawings, specification, and construction and maintenance records.

Collection of crack data is difficult and time consuming because a manual and expensive survey has to be involved in the process, which always requires the intervention of specialized equipment and operators. Due to the nature of the subjective survey, it is very difficult to obtain results that are accurate, repeatable, and reproducible. Thus, there is a need to automate the cracking survey process to improve safety and achieve more objective and consistent data of structure cracking. Today, the implementation of a statistical system to detect, locate, and measure the flexural and tensile cracks in any RC structure is one of the greatest difficulties encountered. In the future, it can be possible to establish a fracture control procedure in the design, fabrication, and maintenance of elements in bridges. If it is properly implemented, the fracture control plan can be capable to ensure the global safety of the RC structures during its service life based on the theory of structural reliability using random variables as the width and spacing of cracks.

A sensor for the reliable detection and monitoring of cracks in concrete structure was first proposed by researchers at MIT and Brown University (Leung et al. 2000). This technique does not require prior knowledge of the crack locations, which is a significant advantage over existing crack monitoring techniques. Moreover, several cracks can be detected, located and monitored with a single fiber. However, the crack direction is needed to measure the crack opening. An ideal application of the sensor, to monitor flexural cracks in bridges, was recently proposed (Olson 2002). To achieve the requirements in the monitoring of cracks on bridges the sensor was improved by researchers of the University of Minho, in Portugal, and of the Hong Kong University of Science and Technology, in China (Diaz de Leon et al. 2004).

The work presented herein introduces the development of the new distributed fiber optic crack sensor for RC structures. The sensor construction is simple and practical for applications to large RC elements. The capability of the sensor in monitoring the formation of flexural cracks, and measure the cracks widths through testing of instrumented RC beams subjected to sustained and repeated loading is demonstrated.

2 RESEARCH SIGNIFICANCE & OBJECTIVES

The monitoring of cracks on RC structures requires sensing at multiple points; therefore many sensors are normally required. The spatial resolution of each measurement should be small, and within a few centimeters (as minimum it is assumed to be the cover thickness) so that formation of a crack in various locations of a structure could be detected. The practical approach for crack sensing involves the development of a distributed fiber optic sensor with the capability of making measurements from only one side of the structure. The sensor can be employed to detect, locate, and measure the crack widths of flexural and tensile cracks on RC structures. The basic principle of operation for the crack sensor is based on intensity variation of the optical power within the optical fiber due to microbending in the fiber by the initiation and opening of cracks. Flexural and tensile crack widths can be expected to double with time for members subjected to either sustained or repeated loading, depending on the environmental conditions. Based on the crack control design procedures the minimum and maximum resolutions required to measure crack widths are functions of the tolerable crack widths versus exposure conditions in reinforced concrete. In view of the fact that it is desirable to detect and measure a crack width ranging between 0.2 and 1.0 mm, the sensitivity of the sensor needs to be sufficiently high. However, for the optical sensor studied in this work, to detect many crack requires that the sensitivity not to be too high. Otherwise, only a limited number of cracks could be detected and monitored since the dynamic range of any Optical Time Domain Reflectometer (OTDR) is not unlimited.

The primary objectives of the work were: 1) To examine the sensor's methodology in detecting and localizing the formation of cracks in various locations; 2) To evaluate the sensor's capability in measuring a range of crack widths with respect to the sensitivity and control of the sensor response; and 3) To demonstrate the applicability of the sensor in monitoring flexural cracks on RC beams subjected to sustained and repeated loading.

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Computer benchmark for static and dynamic damage identification in bridges

A. Del Grosso & F. Lanata
Department of Structural and Geotechnical Engineering, University of Genoa, Italy

ABSTRACT: Structural Health Monitoring can be defined as real time structural condition monitoring based on feedback information from the structure during its service life, in terms of response and performance under operational and environmental conditions. While there have been many works on SHM in relation to mechanical or aerospace structures, monitoring of civil structures is still at a preliminary phase. In order to prevent failures of civil structures, SHM systems have been recently developed and applied to an increasing extent. At the core of any health monitoring system stays the ability to automatically identify structural damage, considered as a weakening of the structure that negatively affects its performance. Damage identification means, at the simplest level, determining that some type of damage has occurred. At a deeper level, it includes damage localization and finally the assessment of the extension and severity of damage.

The majority of the methods reported in the literature have been focused on dynamic-based techniques, but, in the last few years, also static-based procedures have gained increasing importance. With a dynamic-based approach, location and severity of damage is identified by measuring a change in the global dynamic properties of the monitored structure. The dynamic response is usually compared with the results from a field-calibrated simulation model.

On the contrary, static-based techniques allow damage identification by measuring changes in the static structural response. The measured quantities are typically displacements or strains under environmental and applied thermal loads. This type of monitoring needs the realization of complex permanent instrumental monitoring systems, enabling in principle a continuous evaluation of the actual safety conditions of structures. For this purpose, only long periods of observation seems to give reliable information.

The paper describes the features of typical data flows produced by a continuous monitoring system both in the static and dynamic approaches and provides an advance to an algorithmic treatment of these data. The application of some algorithms will also be discussed, showing advantages and disadvantages of the proposed approaches.

The simulation system has been employed on a two span continuous beam. A MATLAB® based program has been used for the simulation. The simulated structure is supposed to be equipped with a set of displacement sensors. The virtual installed sensors are chosen with the aim to simulate a monitoring system composed by fiber optic sensors with a distributed configuration. Sensors are located at the mid point and near the supports of each span, in symmetric positions, so as to cover almost completely the beam.

Measurements deriving from the virtual monitoring are the elongation and the shortening of the sensors (often called deformations for simplicity), and they are relative to their initial length in the reference state. The monitoring system is supposed to be designed in order to give the long-term response of the structure. For the static approach 4 deformation measurements in a day are supposed, according to the temperature measurements, for a total period of about 8 years; for the dynamic approach the monitoring of the structure at regular intervals of 330 s before and after damage initiation is considered.

Finally, simulated measurement errors have been generated and superimposed on the measured displacements.

With the aim to study the static structural response under ambient excitations, a thermal load has been applied to the simulated structural model. Thus, two different temperature distributions have been simulated: the first one has to be applied to the lower surface of the beam and the second one is supposed to be applied to the upper surface, in order to have thermal gradient and consequent deformations in the modelled beam.

For the dynamic excitation, a white noise has been generated in order to study the structural response under ambient vibrations.

The benchmark has been completed with the introduction of traffic loads. A simulation has been conducted taking into account the possible presence of the weight in motion in terms of the different traffic during the day and of the probability distribution of the traffic weights.

The intensity of the daily traffic loads has been extracted from the available traffic data. The simulation is able to control if there is a moving load (successful event) on the simulated bridge during each of the six daily measurements.

After the study of the structural model in the healthy state, different damage scenarios for the beam have been simulated. The damage is supposed to affect one or more elements of the mesh, also in different positions along the beam. However, the extension of the damaged zone can only be an integer multiple of the dimension of a single element of the mesh. The severity of damage can range between 0 (no damage) and 100% ('complete' damage, i.e. no flexural rigidity).

The finite element model has been finally solved for both the static and the dynamic methods, in order to have the nodal displacements corresponding to different damage scenarios. The procedure is able to simulate the true damaged beam behaviour. The applied loads are the same for all the simulated cases for each approach.

A selection of the algorithms proposed in the literature has been first applied to the simulated data, in order to identify the most suitable damage detection algorithms. First of all, simple predictive models have been applied to the simulated structural response, in order to extract a trend with specific threshold limits characterizing the normal operational life of the structure. Any deviation from this behavioral range could be associated to a damage occurrence. Also, wavelet algorithms have been chosen because they can be considered as the evolution of the Fourier transform capable of giving information on the localization of damage. Here the wavelets have been used to filter the data and evaluate the influence of traffic loads in the long-term time series.

Based on the results of the study, it is the Authors' opinion that damage sensitive features and the corresponding statistical classification schemes must be developed on a case-by-case basis taking into account the specific properties of the structure being monitored and the specific properties of damage that is to be detected. The classification procedure must be trained with data from the actual system to set up appropriate threshold levels sensitive to damages, and that do not produce false positive indications of damage.

The main objective of the work was is detect anomalies in the long-term time series deriving from a continuous structural monitoring. Damage threshold to be identified had to be relatively small, in order to detect damage that doesn't appear by visual inspection. As a consequence, a benchmark study has been generated for studying the structural response of a particular class of structures like bridges. Different damage detection algorithms can be applied to the simulated data in order to test the more suitable procedure for the given application. The extension of the benchmark study to analyze the response of other structural typologies is actually under development.

In this paper only two simple signal processing techniques have been applied, predictive models and wavelets. The influence of traffic loads on the long-term structural response has also been investigated. Predictive models detect damage when the structural response overcomes a pre-defined threshold level, defined from the analysis of structural behaviour in the previous undamaged state.

Minimum threshold level for the identification of damage changes using different algorithms and different input data, above all in presence of moving loads on the structure. As a consequence, a pre-processing algorithm using wavelets has been applied to the initial displacement time series for filtering the effects of traffic loads. The technique has revealed very effective to improve damage detection with the simple algorithm of predictive models.

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A real scale PC bridge for testing and validation of monitoring methods

H. Budelmann, K. Hariri & A. Holst
Technical University Braunschweig, Germany

ABSTRACT: Within the Collaborative Research Center (CRC) "Live Cycle Assessment of Structures via Monitoring" at the Technical University Braunschweig funded by the German Research Foundation, several projects are dealing with the monitoring of reinforced and prestressed concrete structures: development and testing of new sensors for concrete degradation and for corrosion and fracture of prestressing elements, of adaptive prognostic models for concrete degradation and of a reliability-based system assessment tool.

The condition of tendons needs to be assessed on the basis of the following questions: Where exactly are the tendons located? What is the present tensile force? Is there any damage? Are there any fractures? Can protection against corrosion still be guaranteed? Is there any corrosion activity? If so, how was it caused? There are also a number of additional questions, but they are of minor importance. For this purpose, several sensors were developed in order to solve these problems at the iBMB of the Technical University in Braunschweig in the last decade.

The validation of the sensors occurred on simple elements in the first step. Those elements were then extended to real scale reinforced members for the purpose of improving innovative sensing methods. These building substitutes allowed us only the verification of sensing mechanisms for defined boundary conditions.

In order to get a significant input especially regarding reliability analysis, one needs real scale substitute buildings with large scale action on specific members. This is the reason why it was decided to design and build up a new real scale building with the shape and mechanisms of a bridge. This contribution will report on the new designed real scaled PC bridge.

The designed structure provides typical structural situations in a realistic manner. It consists of a double-T cross section with a height of 80 cm and has a total length of 17 m with a span of 13 m and a cantilever of 4 m as depicted in Figure 1. In longitudinal as well as in transverse direction the bridge is prestressed, with internal or external tendon profile, with or without bond. So prestressing techniques as known from conventional motorway bridges have been included. Traffic load will be applied via hydraulic actuators respectively via additional prestressing an deadman. The design of the bridge had to take these factors into account. It was designed for 4 different static systems. The abutments are movable in order to create those different states.

Within this structures several faults and weaknesses are produced. Furthermore devices are supposed for chemical and mechanical attacks, leading to degradation processes. The weak points are hollow members and ungrouted parts of ducts with a local attack of chloride on the strand as described above. Some tendons will be fractured by drilling and by means of electro-chemical corrosion via applied currents. Other factors regarding degradation of the superstructure are honeycombs, cracks in concrete to allow a faster penetration of degrading agents like chlorides or NH_4SCN.

Several new sensor techniques regarding durability affecting parameters and monitoring of prestressed members have been developed and used in the last years within the framework of the Collaborative Research Center 477 in Braunschweig. Table 1 shows an overview of the investigated parameters as well as the measurement principle which were used to ensure the measurement of those parameters as e.g. strain of steel fracture.

Besides LVDT to measure the deformation of the whole building and magnetoelastic sensors for the measurement of forces in the longitudinal strands as depicted in Figure 1, fiber optic sensors

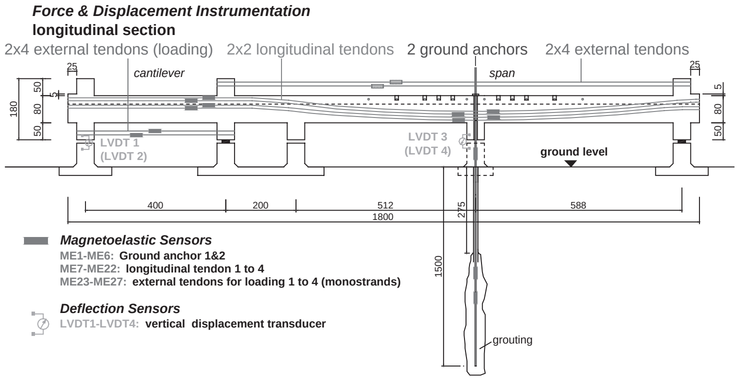

Figure 1. Longitudinal section of the experimental bridge “Concerto” and force and displacement instrumentation.

Table 1. Overview of sensor development.

Measurement parameter	Measurement principle	Ref.
Steel Strain	Magnetoelastic Method	Budelmann et al. 2004
Steel Fracture	Electromagnetic Resonance	Wichmann et al. 2004
Moisture, pH, Chloride	Solvatochromic Dye/Fiber Optics	Flachbarth et al. 2004
Concrete Corrosion	Concrete Corrosion Cells	Schmidt-Döhl et al. 2004
Steel Corrosion	Electromagnetic Resonance	Budelmann et al. 2004

are included in the transverse prestressing members. Permittivity sensors were installed to measure the dielectric constant of concrete in order to perform accurate high-frequency fracture assessment. The HF-coupling is very crucial. For each tendon 2 coaxial cables are included. One is fractured after 6 m, the other one lies in the structure over its whole length. The strands are coupled each from one side (the monostrand builds the ground). Besides those sensors chemosensors were also installed. Chemical sensing via optical fiber as described in Flachsbarth et al. 2004 is built in the superstructure. Own corrosion sensors were also mounted. Some prototypes of a miniature plastic board sensor are mounted on concrete spacer for deep depended monitoring of concrete resistance in order to detect corrosion activity.

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MEMS-based sensor networks for bridge stability safety monitoring during flood induced scour

J. Isley II & M. Saafi
NSF-REU in Nanotechnology for Civil Infrastructure Safety and Security, Alabama A&M University, Normal, AL, USA

J. Julius
Department of Electrical Engineering Technology, Alabama A&M University, Normal, AL, USA

ABSTRACT: Bridge scour is the leading cause of bridge failure in the United States. Scour is credited with 95% of bridge collapses that take place in the United States. Because these damages occurs under water and not visible, they went undetected thereby jeopardizing the safety and security of the users. Current techniques for scour detection are extremely labor intensive requiring heavy and special equipment to operate and to transport from bridge to bridge making it expensive to use for a large number of bridges. To overcome the shortcomings of current scour detection techniques, the Center for Transportation Infrastructure Safety and Security at Alabama A&M University is currently developing an innovative, low cost and high resolution technique to remotely and continuously monitor and detect scour at bridge piers and abutments. The proposed technique uses low cost PFOSs and MEMS-based electronics and will be installed around bridge piers and abutments to detect and monitor scour depth. The proposed monitoring system uses an array of polymer optical fibers connected to specially configured MEMS-based electronics e.g., switches, phototransistor, LED, amplifier, detector and multiplexing system to detect and monitor the change in the media. The reflection mode of the optical fiber will be used to detect scour and erosion near bridge support. The principle of the detection system is shown in Figure 1. As shown in Figure 1-a, a MEMS light source generates a light intensity and this light intensity travels in the fiber optic and then reflected back from the end of the fiber. The reflected signal is received by a MEMS-based detection and signal processing system to display the sensor response. The reflected signal depends on the medium in contact with the end of the optical fiber and after calibration this property will be

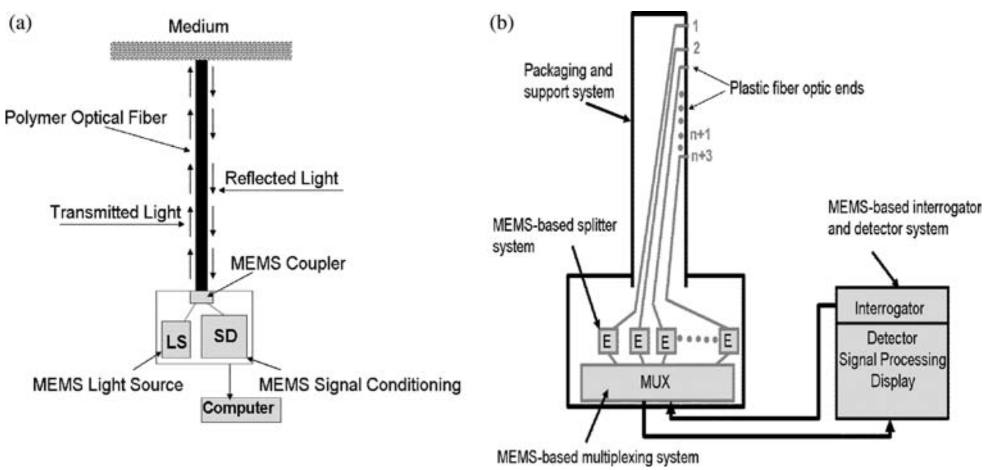

Figure 1. Principle of MEMS/PFOS sensing system.

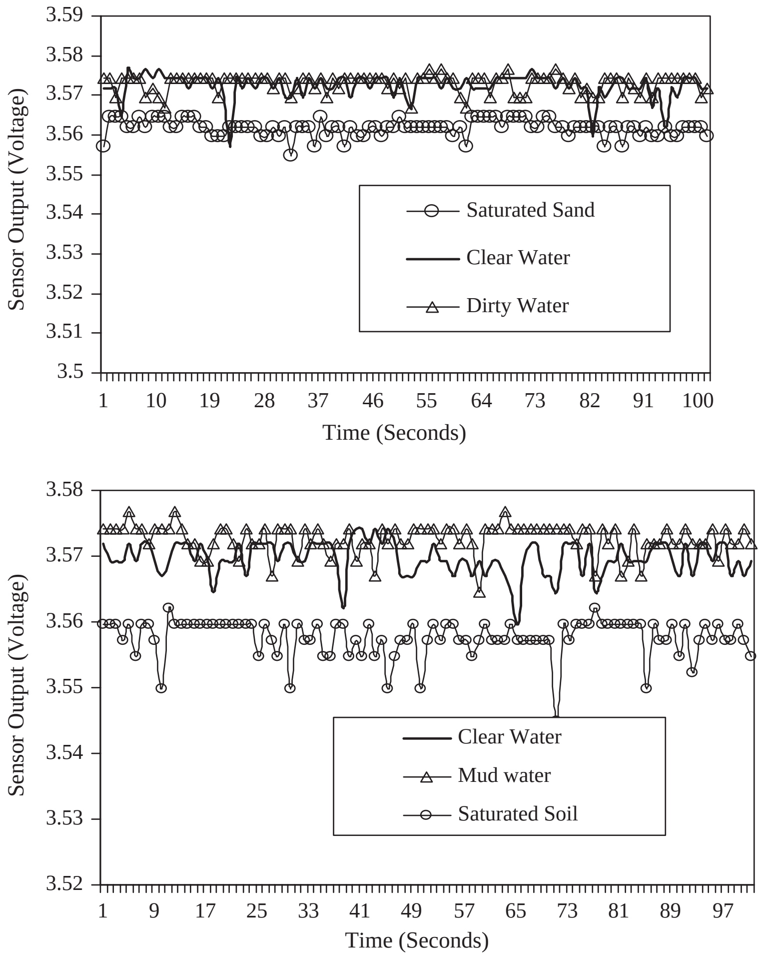

Figure 2. PFOS response when embedded into sand, caly, mudded and clear water.

used to detect scour. For example if the end of the optical fiber is in contact with sediment, the sensor will exhibit a signal reflecting the presence of sediment. When a scour occurs and the sediment is eroded, the sensor displays a different reflection signal indicating that water has replaced sediment and thus scour has occurred and by using an array of these sensors, the depth of the scour can be determined.

As shown in Figure 1-b, the optical scour monitoring system consists of a vertical array of numerous single point polymer optical fibers embedded into a packaging support with fiber ends extending from the support and exposed to the media such as sediment, water, etc. The support also houses the multiplexing and splitter systems. The MEMS-based multiplexer permits each of the optical fibers and its associated optical splitter to share common interrogator and detector systems in a pre-specified sampling sequence. When the sediment (sand, clay or gravel) is eroded, the ends of the optical fibers in the array display a different reflection or transmission coefficient indicating that water has replaced sediment. By knowing which of the optical fiber ends in the array is indicating the changed reflection or transmission coefficient, an estimate of how much scour has occurred is provided. During operation, a remote light source signal is passed through the multiplexer to each of the optical splitters. The return reflection signals from the end of the optical fiber is then fed from the optical splitters to the multiplexer from which it is sent to a remote optical detection and signal processing system. The interrogator and detector will be miniaturized using MEMS technology to obtain a compact and low-cost chip. The number of optical fibers in the array depends on the required resolution.

Figure 2 shows the effect of each medium on the sensor response for the sand and clay respectively. As can be seen, the PFOS distinguishes saturated soil from mudded and clear water, indicating that the response PFOS changes when a soil medium is replaced by water as in the case of a scour process. However, the experimental results show that the PFOS embedded into clear and mudded water exhibited approximately the same response. This is mainly attributed to the test procedure where it was found difficult to produce mudded water in a small container. More tests are currently underway using a large scale hydraulic channel to further evaluate the effect of different mediums on the sensor response.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Acoustic emission analysis techniques for wireless sensor networks used for structural health monitoring

M. Krüger
TTI GmbH – TGU Smartmote, Stuttgart, Germany

C.U. Grosse
Department of Civil and Environmental Engineering, University of California, Berkeley, USA

J.H. Kurz
Institute of Construction Materials, University of Stuttgart, Germany

ABSTRACT: Continuous structural health monitoring provides data from inside a structure and will help engineers to better understand the structural performance and to predict the durability and remaining life time of civil engineering structures. Failure of steel tendons and concrete cracking are important aspects of structural health and maintenance that could be determined by acoustic emission analysis techniques. Wired systems for acoustic emission analysis are still used to monitor structures today, but their installation is relatively expensive, vulnerable and time consuming. In this paper, acoustic emission techniques are presented that could be implemented into competitive wireless sensor networks. The acoustic emission analysis based on onset detection of the acoustic wave is one of the most common techniques for the localization of acoustic events. However, the onset detection requires high accuracy in time synchronization as well as high sampling rates of 100 kHz or even more. Both, high sampling rates and the high time accuracy require a lot of power. However, if wireless sensors will be used, power efficiency is very important. For that reason the array beamforming technique is presented in this paper. Array beamforming for acoustic emissions means that the direction of the approaching elastic wave is determined with an array of sensors. The theoretical background of this technique as well as first results obtained from tests with test specimens are shown and discussed in this paper.

1 LOW POWER ACOUSTIC EMISSION TECHNIQUES

At the Institute of Construction Materials two different techniques for acoustic emission analysis using wireless sensor systems are under investigation, which are: i) localization of acoustic events based on arrival times of the acoustic wave at multiple sensors, ii) localization of acoustic events using array beamforming techniques.

The acoustic emission analysis based on arrival times of the acoustic wave at multiple sensors is required for the 3D localization of acoustic events. One problem is that for the arrival time detection high accuracy in time synchronization as well as high sampling rates of 100 kHz or even more (up to several MHz) are needed. Both, high sampling rates and the high time accuracy require a lot of power if it will be used in wireless sensor networks. Consequently lifetime of such a system is limited due to power restrictions. Nevertheless this technique could be used sufficiently in combination with wired monitoring systems at large structures like bridges.

1.1 *Array Beamforming for Acoustic Emission Localization*

Array beamforming for acoustic emission localization means using an array of sensors for acoustic emission localization, i.e. the direction of the approaching elastic wave can be determined. For a sensor array numerous sensors are placed at discrete points in a well-defined configuration. The superior signal detection capability of arrays is obtained by applying "beamforming" techniques, which suppress the noise while preserving the signal, thus enhancing the signal-to-noise ratio. Furthermore, arrays provide the station-to-event azimuth called back azimuth. The array methods presented here require the assumption that a plane wave is arriving at the array. The wave must have traveled a certain distance when this assumption is applied. The direction of a propagating elastic wave can be described by the vertical incidence angle, and the back azimuth which is measured relative to the reference sensor of the array.

In practice, not the incidence angle is used but the slowness which is the inverse apparent velocity of the wavefront crossing the array. The apparent velocity is a constant for a specific ray traveling through a material. The components of the slowness vector can be expressed as functions of the back azimuth and the incidence angle.

Sensor arrays as presented here are used for the separation of coherent signals and noise. The basic method to separate coherent and incoherent parts of a signal is array beamforming. Array beamforming enables the determination of the back azimuth of the incident wave. One sensor is chosen as a reference sensor and all parameters are taken relative to this sensor. For most applications all sensors are in the same horizontal plane. Then the vertical component of the slowness vector is zero. Beamforming uses the differential travel times of the plane wave front due to a specific slowness and back azimuth to individual array stations. Therefore, the most important point during array beamforming is to find the best delay times for shifting the individual signals. If the single-sensor recordings are appropriately shifted in time, all signals with the matching back azimuth and slowness will sum constructively.

Acoustic emission localization using a sensor array means determining the back azimuth of the propagating elastic wave. The true beam can only be calculated for the correct back azimuth. Any delay time for each sensor can be calculated by multiplying the coordinates of each sensor with a slowness vector. Since the slowness is a function of back azimuth and incidence angle it is possible to calculate the true beam by beamforming calculations on a grid of different slowness values. The maximum energy indicates the true beam since only the coherent signals interfere constructively. Searching for the true beam can be performed in the time domain called beam packing and in the frequency domain called frequency wavenumber analysis.

If more than one array is used a two-dimensional localization of the source of the incident wave is possible by calculating the point of intersection of at least two back azimuth lines. The advantage of such a localization procedure is that no time consuming onset determination is needed. The signal to noise ratio is increased significantly and the applicability of the principle has already been demonstrated in many fields of application. Due to the long travelpath the signals do not have high frequency content any more. Therefore, as first tests showed, the principle works with even low sample rates. This enables the use of highly automated and wireless systems which have to work energy-saving.

2 CONCLUSIONS AND OUTLOOK

Wireless sensor networks using MEMS technologies could enormously reduce the costs for structural health monitoring to just a small percentage of a conventional cable connected monitoring system. This will increase its application and thus more detailed information from the structural and its condition could be obtained. Therefore engineers will be enabled to use more precise information for the structural analysis, repair dimensioning and life time prediction. For that reason first prototypes of wireless monitoring systems were developed. The next steps are to look for efficient data reduction methods as well as for sufficient algorithms for signal analysis, so that the large amount of measured data is reduced to only a few relevant data items. Then structural health monitoring will be intrinsically efficient.

Two techniques for acoustic emission analysis seem to be sufficient for the use for wireless sensor networks for monitoring large engineering structures. First is the localization of acoustic events based on arrival times of the acoustic wave at multiple sensors and the second is the localization of acoustic events using array beamforming techniques. It was shown that both techniques could be used to detect concrete failure as well as steel rupture. However, array beamforming techniques are more useful for wireless sensor networks due to moderate power consumption.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Ground anchorage tension force monitoring by using magnetostrictive method

E. Yanagisawa, O. Nakade & S. Oka
Mitsubishi Heavy Industries Ltd., Hiroshima, Japan

K. Ideue
Ryomei Engineering Co., Ltd., Hiroshima, Japan

K. Onishi
West Nippon Expressway Company Limited, Takamatsu, Japan

T. Nagashima
Formerly Mitsubishi Heavy Industries Ltd., Hiroshima, Japan

ABSTRACT: For long term monitoring of the ground anchorage tension force, this paper presents a new way of monitoring by using the steel ring and magnetostrictive method. The steel ring is installed at the top part of anchorage, stress of steel ring is measured by magnetostrictive method, and tension force of ground anchorage is evaluated. This system only needs steel ring with paint, and no electrical wiring sets on the field. More than two years field test proves the accuracy and durability of the proposed monitoring system.

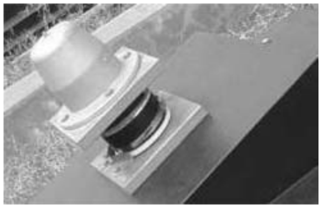
1 INTRODUCTION

The ground anchoring method using tension bars or wire rope has widely applied against landslip. During the long term use, the force of ground anchoring tension bars or wire rope decreases or increases, in case of the movement of ground and deterioration of the anchorage device itself. Therefore, to stabilize the ground by ground anchorage method, tension force monitoring of anchorage in service is important. In the present state, lift-off method and load measurement by using load cell are common way of tension force monitoring. The disadvantage of these methods is short of mobility due to the necessity of heavy crane, low endurance of electrical wiring of the equipments for long term use, respectively.

To solve these problems, this paper presents quite a new tension force monitoring method using steel ring and the magnetostrictive method. In general, magnetostrictive method is able to measure the stress of steel non-destructively, without removing paints. So, if the steel ring is installed at the top part of anchorage and measures the stress of this ring by magnetostrictive method, the tension force of ground anchorage is evaluated without load cell and lift-off method. This system only needs steel ring with paints on the field, and no electrical wiring is left. So this system has long life span not less than the anchorage lifetime, and the load can be monitored until the ring is rot away.

2 FIELD TEST

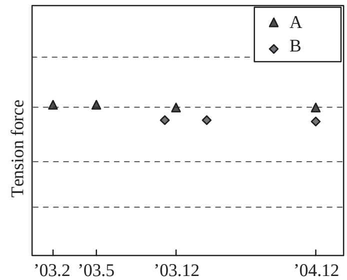
The rings were set to the land slope in the field, and conducted the long-term field test. The shape of the steel ring is important to improve the accuracy of the tension force estimation, because proper height, almost equal to the ring diameter, is needed to smother the effect of the distributed load. So, to determine the shape of ring, preliminary FE-analysis were conducted. On the other hand, the



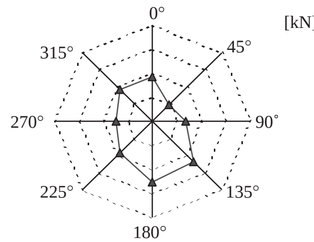
(a) With covered byrubber

(b) Measurement surface

Figure 1. Setting conditions of the steel ring.



(a) The change of tension force in two years



(b) Load distribution

Figure 2. Two years load measurement results.

material of the ring needs to have sufficient strength, good magnetostrictive sensitivity, and low residual stress. SCM440 quenching and tempering steel with 30S shot blast condition can realize these requirements.

The shape of ring and setting up condition are shown in Figure 1. The ring is covered with rubber, which is bound with the stainless steel band, to maintain the normal condition of the paint. Two years have passed since the test began, but no crack and no rust can be seen on the steel ring. The painting of the ring keeps good condition enough to measure the stress by magnetostrictive method, after two years have passed in the rainy and sunny environment.

Two years load measurements results are shown in Figure 2. The tension force shows no change in two years as shown in Figure 2(a). The deviation of load distribution is able to check in the measurement. Figure 2(b) reveals the load distribution of the ring. The load is slightly moved to 225 degree side. So, it can be seen that the load level of measured anchorage has no change, although it has slightly deviated load. After the measurement, the ring was covered again by rubber, as shown in Figure 1, until the next measurement.

Weather proof acceleration test also prove fur more life span. Considering the acceleration ratio of the test, the estimated life of the paintings is about 40 years.

Furthermore, to improve the utility of the proposed tension force monitoring system for the practical measurement, multi functional steel ring, which is hybrid of the proposed steel ring and strain gage, and correction way of the machinery dependency of the magnetostrictive stress is also proposed and certified. These improvements enable to measure in short term, such as daily and hourly, and mass production of the ring.

3 CONCLUSION

For long term monitoring of the ground anchorage tension force, this paper present a new way of monitoring by using the steel ring and magnetostrictive stress measurement method. This way of monitoring only needs steel ring with paint, and no electrical wiring sets on the field. Field test reveals that the proposed system has enough accuracy and durability for practical use.

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Monitoring an interstate highway bridge with a built-in fiber-optic sensor system

R.L. Idriss & Z. Liang
Civil Engineering Department, New Mexico State University, Las Cruces, NM, USA

ABSTRACT: Using High Performance Concrete (HPC) in prestressed concrete girders has enabled engineers to design bridges with longer span lengths and fewer supports, shallower sections, and increased girder spacing, which can decrease the fabrication, transportation, and erection costs of the bridge. However, despite the wide current use of HPC, there are still factors that need to be further considered. The design of a prestressed concrete girder is highly dependent on the amount of prestress loss expected over a period of time. Current methods to empirically estimate these losses were developed for conventional concrete and have not yet been modified for HPC. Those methods need to be reevaluated for HPC. Another factor that needs further consideration when using HPC is camber. Camber is an important serviceability consideration in the design of prestressed bridge girders. Both material properties and structural parameters influence girder camber. Current simplified methods of camber prediction, such as the Precast/Prestressed Concrete Institute (PCI) multiplier method and the PCI improved multiplier method were developed for regular strength concrete and need to be further evaluated for HPC.

An optical fiber monitoring system was designed and built into one span of the five span high performance prestressed concrete I-10 Bridge over University in Las Cruces, NM. The spans have lengths of 42.14, 30.94, 40.70, 40.70 and 40.31 meters (138.25, 101.5, 133.5, 133.5, and 132.25 feet). Each span consists of six 1.37 meter (54 inch) tall prestressed concrete spread box-girders. The girders are constructed of high-strength prestressed concrete with minimum 28-day strength of 68.9 MPa (10 ksi). A total of 72 long-gage (2 m long) deformation sensors, along with 36 thermocouples were embedded in the prestressed concrete girders. The monitoring equipment and deformation sensors were manufactured by the SMARTEC co. The system is based on a low-coherence interferometry in long-gage optical fiber sensors. Sensors were installed along the bottom and top flanges, at mid-span and quarter spans. Pairs of crossed sensors in a rosette configuration were embedded in the webs at the supports. The embedded sensors measured temperature and deformations at the supports, quarter spans, and mid-span. The sensors were used to measure girder camber, as well as the short and long-term losses in the prestressing cables throughout beam manufacturing, bridge construction, and service.

Several methods were used to estimate the prestress losses: the PCI general method, the ACI-ASCE method, the LRFD method, and the LRFD lump sum method. None of these methods were developed specifically for high-performance concrete. The prestress losses calculated using the sensor measurements were compared to the estimated losses.

The sensor measurements were also used to calculate the average curvature of the cells, which in turn were used to construct the curvature function of the whole girder. The deflection of the girder was then produced by double integration of the curvature function. Along with the cambers obtained from sensor measurements, cambers were also measured using a self-leveling laser on site. Sensor measurements agreed well with the laser measurements. Camber was then calculated using the PCI bridge design manual multiplier and improved multiplier methods. At transfer both PCI methods closely agree with the results based on sensor measurements. Both PCI methods overestimated the camber at erection. In the PCI methods creep of the concrete is primarily responsible for the long

term camber growth. HPC creeps less than regular strength concrete. This could explain why the girders had much less camber growth at erection than what was expected by the PCI methods.

The research leads to the following conclusions:

- The PCI general, ACI-ASCE, LRFD, and LRFD lump sum methods are all very conservative in estimating the prestress losses of HPC girders.
- The PCI multiplier method and the PCI improved multiplier method over-predicted the camber at erection. Both methods factor in camber growth due to creep. The lack of actual camber growth can be attributed to the lower creep exhibited by HPC.
- Equations need to be developed to better estimate camber and prestress losses for HPC girders.

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Monitoring of fatigue crack by field signature method

K. Oku
Osaka University, Osaka, Japan

K. Arita
Atlas Co. Ltd, Chiba, Japan

Y.-C. Kim
Joining and Welding Research Institute, Osaka University, Osaka, Japan

1 INTRODUCTION

FSM (Field Signature Method) was developed as a method for detecting reduction of thickness owing to erosion and corrosion in steel structures non-destructively with applying an electric potential difference method. Applying FSM to monitor initiation and propagation of fatigue cracks is considered.

In this paper, first, fatigue tests using small specimens are carried out. It is investigated whether initiation of fatigue crack can be detected by FSM or not. Next, it is also investigated whether propagation of fatigue cracks contained in a large-scale specimen can be detected by FSM or not.

2 EXPERIMENTS

2.1 *Monitoring by FSM*

In FSM, a number of sensing-pins (called as pins hereinafter) are attached in a lattice on a monitoring area (that is, the position where initiation of fatigue cracks is expected and the neighborhood of fatigue cracks). Then, measuring an electric potential difference generated between two sensing-pins (called as a pair hereinafter) by impressing direct pulse current among pins at any intervals, initiation and propagation of fatigue cracks is monitored.

In this paper, the change obtained from an electric potential difference is expressed by converting Field-signature Coefficient value (called as FC value hereinafter).

FC value of Pair A at time, i, can be obtained from the folowing equation:

$$FC = ((Ai/Bi)(Bs/As) - 1) \times 1000(\text{ppt})$$

where,
As : a potential difference of Pair A at the starting,
Bs : a potential difference of a reference pair at the starting,
Ai : a potential difference of Pair A at time, i
Bi : a potential difference of a reference pair at time, i

2.2 *Detection of fatigue crack initiation*

Figure 1 shows the outline of a small specimen. Material is SM490.

Loading condition is tensile axial loading (stress ratio is less than 0.1) in the experiments.

For identifying initiation of fatigue cracks, strain gauges are attached at the toe of boxing welds (both sides). The side attached the sensing-pins is defined as an obverse.

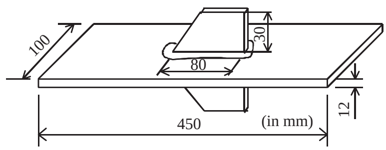

Figure 1. Outline of specimen.

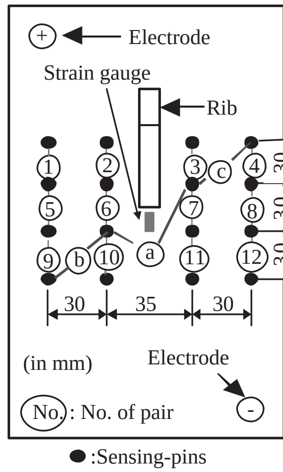

Figure 2. Position of sensing-pins and electrodes.

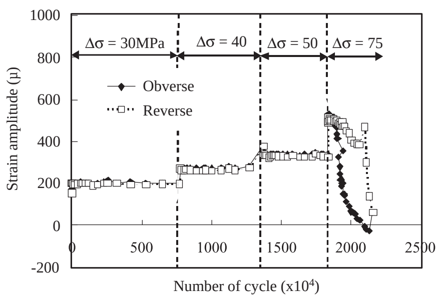

Figure 3. Results obtained by strain gauge.

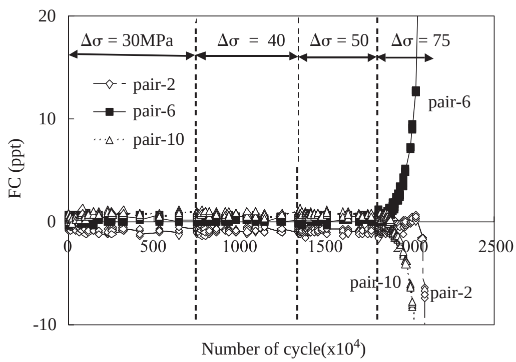

Figure 4. Results monitored by FSM.

Figure 2 shows the attaching positions of the electrodes and pins.

The experiment is carried out amplifying the stress amplitude step-by-step. Measuring strain amplitude obtained from the attached strain gauges, an electric potential difference generated between the pairs (from 1 to 12) shown in Fig. 2 is measured at regular intervals.

3 RESULT OF THE EXPERIMENTS AND CONSIDERATION

Figure 3 shows the strain amplitude in each number of cycle obtained from strain gauge is attached at the toe of boxing welds. From the measured results by strain gauge, it is elucidated that the fatigue crack is initiated at stress amplitude 75 MPa.

Figure 4 shows FC value (0 kN at applying the static load) of each pair in number of cycle measured by FSM. FC value of each pair dose not change at all within the region of stress amplitude 30–50 MPa. On the other hand, FC value of each pair changed when stress amplitude becomes 75 MPa.

Figure 5 shows again the monitored result at the region of stress amplitude 75 MPa shown in Fig.4. At first, FC value increases near the position (Pair-6 and Pair-7) of initiation of fatigue crack

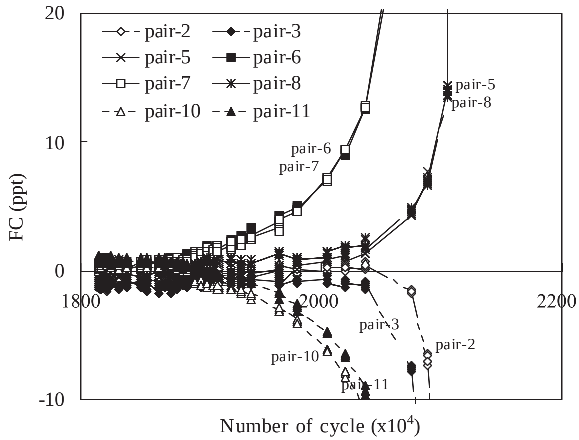

Figure 5. Magnified figure of monitored results.(Stress amplitude 75 MPa)

(at boxing welds) and FC value decreases in the direction of propagation of crack (Pair-10 and Pair-11) next. Therefore, it is elucidated that the position of crack initiation and the direction of the propagation can be identified by noting the position of the pair of which FC value increases or decreases.

4 CONCLUSIONS

(1) The measured FC value did not change at all unless fatigue crack were initiated at the monitoring area. Initiation of fatigue crack could be identified by the change of FC value.
(2) Noting the positions where FC value of pair changed, the position of fatigue crack initiation and the direction of propagation could be identified by FSM.
(3) Fatigue cracks less than 1mm, which could not be confirmed by visual observation, could be detected by FSM.
(4) Fatigue crack generated on the reverse of specimen could be detected by FSM from the obverse. Therefore, the fatigue crack generated on the reverse of specimen or inside of welds, which cannot be confirmed by visual observation, could be detected by FSM.
(5) Propagation of fatigue crack (length is 0.1 mm) passed through thickness could be detected by FSM.
(6) The propagation length of fatigue crack and FC value was good correlation. using this relation, the length and the propagation velocity of fatigue crack could be found.
(7) The measured FC value was not influenced by the opening displacement.
(8) Propagation of fatigue cracks could be detected without the influence of vibration of members.

From the results, initiation and propagation of fatigue crack could be monitored with good accuracy under live loads on real bridges in sites.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Multiplexed fibre Bragg grating sensor system for bridge monitoring applications

Y.M. Gebremichael, W.J.O. Boyle, J. Leighton & K.T.V. Grattan
School of Engineering, City University, Northampton Square, London, UK

B.T. Meggitt
EM Technology, London, UK

ABSTRACT: A major concern in today's strategically important civil structures, such as rail bridges, is condition assessment of the existing infrastructures for ever increasing usage demands including increased loading and vehicle speed in addition to damage, aging or environmental deterioration. In this respect, the incorporation of resident, cost effective damage detection and monitoring systems is an important consideration, both to detect and track structural damage that may eventuate as well as to monitor the effectiveness of any subsequent repairs that may be carried out in an efficient and effective manner. The principal objective of this current research is to develop an advanced fibre optic monitoring system that will provide enhanced knowledge of the integrity of large strategic civil engineering structures, such as rail bridges, in order to assess their structural worthiness. Such integrated monitoring systems will also provide a means of evaluating the in-service performance of the structure for enhanced operational use in a safe and predictable condition.

Existing methods for monitoring such structures are severely limited by the available technology, relying on electrical strain sensors and load cells that are time consuming to install and require a large number of electrical connections and thus complex cabling. These systems are also very difficult to embed during either the construction or any subsequent repair processes that may be carried out. Consequently, much monitoring is usually undertaken by periodic visual inspection or through the use of conventional strain gauges, which are not well suited either for long-term use or in harsh environments (e.g. during exposure to chlorides).

Fibre Bragg grating sensor based monitoring systems on the other hand offer an enabling technology that can be used for online in-situ monitoring with unobtrusive, chemically inert and electrically immune sensing operation. To exploit this technology for monitoring applications in the remote, harsh environments of civil infrastructure, a multipoint and online monitoring capability with high data measurement repeatability is essential to allow for the continuous determination of the strain-dependent wavelength shift and to create a remotely accessible database providing a record of the condition of the structure over periodic test periods or during its operational lifetime. This can provide invaluable information on the structure in relation to the stress, shrinkage, creep, dead loading, post tensioning and structural degradation manifested by the appearance of cracks, fissures and corrosion. Further, such monitoring can permit the opportunity to record the static and dynamic loading history that is essential in for determining controlled maintenance procedures and schedules and for structural design assessment. This provides a powerful means to enhance the service quality, performance and thus improve the safety of the structure in a continuous fashion, both during construction and throughout its lifetime. This latter aspect is especially important when the structure is approaching its designed life-span or following unusual phenomena such as subsidence, earthquakes, accidental impacts, high winds, fire and flood in order to avoid catastrophic failure events.

A portable and compact multiplexed FBG sensor demodulation unit utilising a 10 mw broadband light source centred at 1550 nm has been designed and developed specifically for long-term field application use. The laser diode is driven by a stabilised current source and feedback controlled

thermo-electric controller to ensure good stability of its output power and spectral profile. Spatial multiplexing is achieved with an 8-way passive splitter for an 8-channel system utilising a scanning Fabry-Perot filter for wavelength division multiplexing (WDM) interrogation technique to de-multiplex the return signals from the array of gratings used as the strain sensitive devices. The instrumentation is capable of interrogating four sensors per channel for high level strain measurements (each capable of measuring $>\pm 2,500\,\mu\varepsilon$), although up to ten sensors can be configured along each fibre consequently reducing the maximum strain measured. The return signal from the serial gratings on each channel is detected via 8 parallel low noise (30 dB SNR) detectors. The detector circuits developed consist of two stage amplifiers with transimpedance and amplification/filtering stages of a bandwidth of 10 kHz. The detected Gaussian peak analogue signal is then sent to a plug-in data acquisition board (ADwin-light-16) with analogue inputs and outputs. The data acquisition board is designed around a SHARC DSP processor with its own local memory for fast data acquisition (conversion time of $10\,\mu$s), on-line processing (processing of each measurement can occur immediately after acquisition) and control of the scanning Fabry-Perot filter in real time under Windows. The board comes with proprietary real-time software (ADbasic) which allows programming mathematical operations and functions which are executed immediately after each sampling step. The Adwin DSP board also generates a ramp signal to drive the tuneable WDM filter as well as simultaneous data capture from the detection channels at every scan voltage. The DSP temporally resolves the optical spectra received from each of the 8 channels used, for every scan of the Fabry-Perot cavity and thus the position of each peak is identified. In addition, post processing and data visualisation in the PC can be handled with many popular programming environments including Visual Basic and LabVIEW. To achieve linear drift free wavelength and thus strain and temperature readings, the measurements are made relative to the two thermally stabilised (to 0.15 pm) reference Bragg gratings which are assembled in a thermally controlled package for the rejection of common mode noise and multiplexing non-linearity, as well as wavelength scaling in a differential measurement. The reference gratings are also used to eliminate any long term or short-term thermal or shock vibration-induced drifts arising from the scanning filter. An accurate detection of the peak position in the wavelength domain is the key to accurate strain differentiation. Peak location involves least square curve fitting or threshold level detection around the peak from which the centroid of the reflected grating spectra is calculated. The system designed is capable of measuring multipoint data using for potential use in strain mapping across a large structure with a short term resolution of $\pm 2\,\mu\varepsilon$ and bandwidth of 100 Hz. Comparative measurements with resistive strain gauges showed close correlation highlighting the performance of the system. Long term test results also show that the strain resolution of the system at full bandwidth is less than $\pm 5\,\mu\varepsilon$.

In most monitoring applications, Bragg grating sensors are either surface bonded to existing structures or embedded within the structures during manufacture. Several bonding adhesives and techniques of attachment have been investigated to optimise the strain transfer and to achieve high repeatability and sensor integrity in long-term measurement use. Cynoacrylate type adhesives achieved near to 100% strain transfer in a manner that was consistent over many tests with no degradation of the sensors or the bonding mechanisms employed. Prior to attachment, the acrylate coating on the Bragg gratings was stripped off to enhance the bonding to the surface, which was cleaned by using an appropriate solvent. Furthermore, tensile test results on post embedded array of Bragg grating sensors into carbon fibre structures, in an attempt to create a smart sensing strips or strengthening rods, have shown good results correlating with surface attached electrical strain gauges. Fibre Bragg grating sensors can also be embedded into carbon composite structures of various thicknesses during the composite pultrusion stage. Such robust sensor packaging will result in preassembled sensors that could routinely be used in harsh environments, large concrete or masonry structures with ease.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Field observations on concrete box girder railway bridges

L.O. Santos, J. Rodrigues & X. Min
LNEC - National Laboratory for Civil Engineering, Lisbon, Portugal

1 INTRODUCTION

The Corona and Espinhaço de Cão Bridges, integrated in the railway line linking Lisbon and Algarve, were submitted to static and dynamic tests to evaluate its structural behavior under static loads and its dynamic characteristics (frequencies, mode shapes and damping ratios).

Corona Bridge has a total length of 208 m, divided in three intermediate spans with 48 m and two extreme spans with 32 m. Espinhaço de Cão Bridge has only four spans with a total length of 133 m. These bridges are similar structures although with a different number of spans. In fact, both bridges have a prestressed concrete box girder deck, 2.5 m height and 8.1 m width, rectangular piers with 3.50 m × 2.00 m and the same type of bearings, which are fixed for all horizontal displacement at the South abutments and allow longitudinal displacements at the other supports. Piers and abutments are in reinforced concrete with footing foundation.

After a brief description of the bridges, this paper presents the finite element models used to analyze both bridges and the experimental procedures adopted in the field observations, including the use of a hydrostatic levelling system to measure vertical displacements during the static tests and the output-only modal identification techniques used to analyze the data obtained in the dynamic tests. The experimental results achieved are compared with the numerical values evaluated with a finite element model of each bridge.

2 STATIC TESTS

The static load tests were performed in two phases: with only a locomotive and with a train composed by a locomotive and six ballast wagons with a total load of 3126 kN. These loads were placed in 12 positions, at Corona Bridge, and 8 positions at Espinhaço de Cão Bridge, in accordance to the load plan that maximizes the most important effects in the structures, however without inducing unwanted situations of early cracking in the concrete.

During the test, vertical displacements and rotations were measured at several sections. In order to measure the most reliable and redundant data, different types of sensors were installed.

Vertical displacements were measured at mid spans of major spans by an upgraded hydrostatic levelling system associated to pressure cells. Some of these sections were also instrumented with traditional mechanical deflectographs.

In both bridges, transverse and longitudinal rotations were measured by electric clinometers located at the top of piers P1, P2 and P3. Two others clinometers were placed close to pier P2 at the third span. At one of these sections a mechanical air-bubble clinometer was also used.

Three automated data-acquisition systems *DataTaker DT515* were used to read data from the hydrostatic levelling systems and from the electric clinometers, allowing an effective control of experimental data in real time.

Extensive experimental data was obtained during the static test. From all these data, some illustrative results are presented, including some influence lines, achieved by placing only the locomotive in different positions along the deck, and deck deformations due to the load train.

Figure 1 presents the influence line of the rotation at the top of pier P2 of Corona Bridge. In general a good agreement between experimental values and computed ones was achieved. It is also

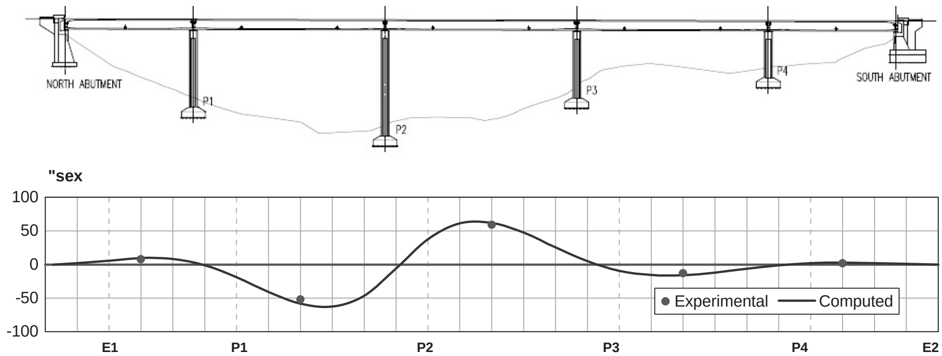

Figure 1. Corona Bridge: influence line of the rotation at the top of pier P2.

relevant the good agreement between values measured by different devices (hydraulic levelling system and deflectographs or electric and air-bubble clinometers).

3 DYNAMIC TESTS

The dynamic tests consisted in the measurement of accelerations in the structures induced by the wind (ambient vibrations) and by trains crossing the bridges in a normal operation condition (including active tilting trains with speeds of 220 km/h). No restrictions were imposed to the railway traffic in order to perform the dynamic tests. An identification of the natural frequencies of the structures was performed from the measured accelerations as well as an analysis of the railway traffic induced vibrations.

The dynamic tests were performed with 15 Kinemetrics ES-U force balance accelerometers, signal conditioning equipment constructed at LNEC and data acquisition hardware and software from National Instruments.

In Corona Bridge the dynamic tests were conducted in three set-ups and vertical and transverse accelerations were measured in 30 sections of the bridge deck. The ambient vibration data was acquired during about 22 minutes using a sampling frequency of 1000 Hz. The railway traffic induced vibration records were also acquired with a sampling frequency of 1000 Hz.

In Espinhaço de Cão Bridge a similar testing procedure was adopted, however only two set-ups were performed with vertical and transverse accelerations being measured in 21 sections of the bridge deck.

4 CONCLUSIONS

The experimental results obtained in the static and dynamic tests have a good correlation with the analytical values computed by FE model. In a general way, the experimental results obtained in the tests of the two bridges are similar, confirming, however, that the Espinhaço de Cão Bridge has a largest stiffness as foreseen by the finite element models.

The hydraulic levelling system associated with pressure cells proved to be an accurate way of measuring vertical displacements in box-girder bridges.

The experimental data obtained about the structural behaviour of these bridges is an important contribution to the characterization of their actual condition at the beginning of their lifetime.

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Assessment and condition monitoring of a concrete railway bridge in Kiruna, Sweden

O. Enochsson, L. Elfgren, T. Olofsson & B. Täljsten
Luleå University of Technology, Div. of Structural Engineering, Luleå, Sweden

B. Töyrä, A. Kronborg & B. Paulsson
Banverket (The Swedish Railway Administrator), Luleå, Sweden

ABSTRACT: A two-span railway concrete trough bridge over Luossajokk in Kiruna in northern Sweden has been studied. The owner wanted to increase the axle loads from 250 to 300 kN in order to reduce freight costs for iron ore. Examples are given of methods used and results obtained from the assessment where bending, shear and fatigue were studied. Material properties, loads and load carrying capacity were evaluated using deterministic and probabilistic methods. It was shown that the bridge could carry the higher loads with a safety index $\beta > 4.7$ for reasonable assumptions of the load distributions. A measurement system was installed to check the actual level of critical strains and the worst positions of the train. Results are also given from a condition monitoring program 2001–2006, launched to periodically check the development of strains with time.

1 INTRODUCTION

The bridge is situated on the Iron Ore Line "Malmbanan" in northern Sweden and passes over Luossajokk in Kiruna. The railway line is mainly used for transportation of iron ore from northern Sweden to Narvik and Luleå on the costs of Norway and Sweden, respectively. Here the owner wanted to increase the axle loads from 25 to 30 tons to reduce the iron ore transportation costs, Paulson & Töyrä (1996).

In this paper, examples are given of methods used and results obtained from a bridge assessment in Sweden. Bending, shear and fatigue are studied for a two-span railway concrete trough bridge. Material properties, loads and load carrying capacity are evaluated using deterministic and probabilistic methods, Enochsson et al (2002). Results are also presented from a condition monitoring program used to check the actual level of critical strain, the worst positions of the train and the development of strains with time, Enochsson et al (2003, 2006).

A recalculation according to the design code, BV Bärighet (2000) showed that the increased axle load would exceed the yield limit in the reinforcement. Before any decision was taken regarding strengthening or replacing of the bridge an assessment with probabilistic methods was carried out.

An outline of the bridge is shown in Figure 1. The bridge is a two-span reinforced trough bridge built in 1965. The mid-foundation is a concrete wall from the same period, whereas the end-foundations are stone walls that were constructed when the line was built in around 1890. A first assessment of the capacity showed that there were three sections where the capacity was too small: (1) in the top of the short span the bending capacity was too low in the longitudinal direction, (2) in the bottom of the short span the bending capacity was too low in the transverse direction, and (3) close to the mid support the shear transfer was insufficient between the beams and slab. It was decided to carry out strain measurements in these sections to check the real influence of the loads, see Figure 1.

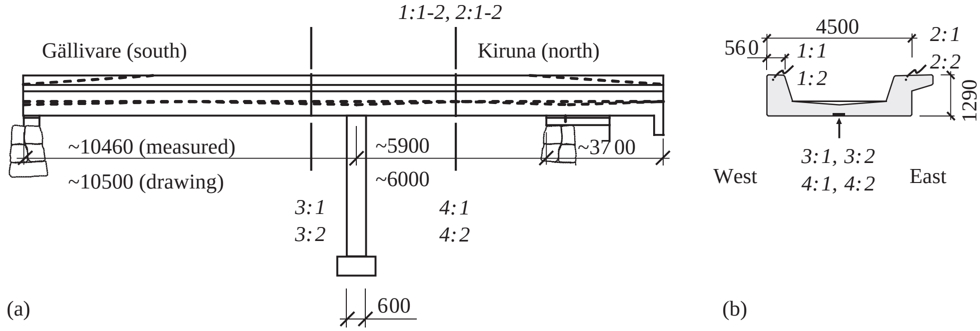

Figure 1. Bridge over Luossajokk in Kiruna in northern Sweden; a) elevation and b) cross section with location of strain gauges.

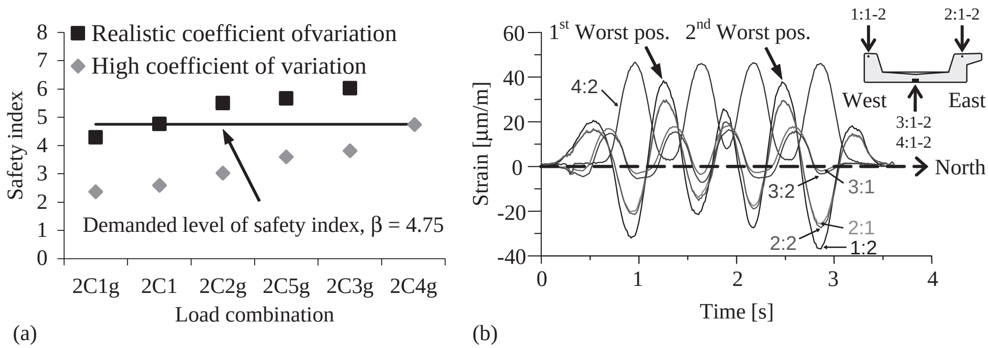

Figure 2. a) Evaluated safety factor β for different combinations of loads, and b) typical results of strain measurements of locomotive "IORE" with the higher axle load (30 tons). The locomotive is driving towards Kiruna town (north).

2 ASSESSMENT AND CONDITION MONITORING

A probabilistic evaluation of the safety was made according to a First Order Reliability Method, FORM, using the program VaP, see Schneider (1997) and VaP (1999). The assumptions regarding types of distributions, mean values, m, and coefficients of variations, $v = s/m$, were chosen using Diamantides (2001) and JCSS PMC (2001). Different combinations with varying assumptions and loads were tested, see Figure 2a.

The actual strain level and its development with time are periodically measured since 2001. It is shown that a structural health monitoring program can be a very useful and powerful tool in the assessment work, see the result from the evaluation of the worst traffic load in Figure 2b.

3 CONCLUSIONS

The assessment of the load-carrying capacity of the bridge has shown that the bridge can carry the increased loads without strengthening. There are many assumptions that could be worth to investigate further e.g. the influence of train velocity (dynamic load factor), break and acceleration loads and uneven temperature, see Enochsson (2002). A structural health monitoring system has been installed and the bridge has been successfully monitored from Luleå University of Technology during the period 2001–2006, see e.g. Hejll (2004), Enochsson et al (2003, 2006).

The result shows that the bridge behaves linearly for an increase of the axle load from 25 to 30 tons and that there is a very small time dependent influence. However, there is quite a big variation during the year depending on the influence of increased stiffness of frozen ballast during the winter period.

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Fuzzy-based variable gain approach for controlling cable-stayed bridges

S.-Y. Ok
Korea Bridge Design and Engineering Research Center, Seoul National University, Seoul, Korea

K.-S. Park
Department of Architechtural Engineering, Dongguk Uviv. Seoul, Korea

C. Chung & H.-M. Koh
School of Civil Engineering, Seoul National University, Seoul, Korea

ABSTRACT: Due to the vulnerability of cable-stayed bridges to dynamic loads such as earthquakes and strong winds, study on vibration control system for the cable-stayed bridges has been of great interest for the past few decades (Dyke 2003). The dynamic response control of the cable-stayed bridges provokes mutually conflicting response control problem. In particular, the strongly coupled model behavior and the complex cable-deck-tower connection configuration further increase the complexities of the response control problem. Viable control strategy for the cable-stayed bridges should thus account for the trade-offs existing between their dynamically induced responses. As one of the gain scheduling scheme (Parlos et al. 2001), variable gain feedback control approach is presented in this paper. The problems with the conventional gain scheduling scheme is: (a) it is difficult to select a set of discrete control gains and determine the proper switching condition among the gains for complex structure such as cable-stayed bridge; (b) degradation of control performance sometimes occurs due to control chattering caused by frequent discontinuous switching. In order to overcome these shortcomings, we present fuzzy-based variable gain approach, which is easy to select a set of discrete control gains, unnecessary to determine the switching condition among the gains, and able to modulate the continuously varying dynamic gain. The proposed technique is a hybrid method with a two-layer topology in which a lower layer consists of several conventional controllers, and a second supervisory layer possesses a fuzzy inference mechanism for endowing the controller with intelligence. Several controllers in the sub-layer are separately responsible for the individual response selected among the multiple target responses of the cable-stayed bridge to be controlled. Thus, each sub-controller is independently designed to efficiently reduce the selected response only. Then, a fuzzy supervisor determines the contribution level of each sub-controller through fuzzy inference mechanism to intelligently manage the overall enhanced control performance of the control system. To investigate the effectiveness of the proposed approach, two kinds of vibration control problems have been dealt with: the seismic response control of cable-stayed bridge and the wind-induced vibration control of cable-stayed bridge under construction. Example designs and numerical simulations of conventional linear quadratic Gaussian (LQG) control system and new fuzzy-based variable gain control system have been performed for the sake of comparison of control performance.

The results of the robust performance between the LQG control system and two FSC systems under uncertainties in the bridge model and seismic excitations are compared. In numerical simulations, three historically recorded ground motions are used as input excitations, i.e., El Centro earthquake, Mexico City, and Gebze earthquakes. The peak accelerations of the earthquake records are 3.4170, 1.0432 and 2.5978 m/s^2, respectively.

From the simulated results, the two FSC systems successfully maintain an overall robust performance under the presence of uncertainties in bridge stiffness as well as the magnitude of the seismic

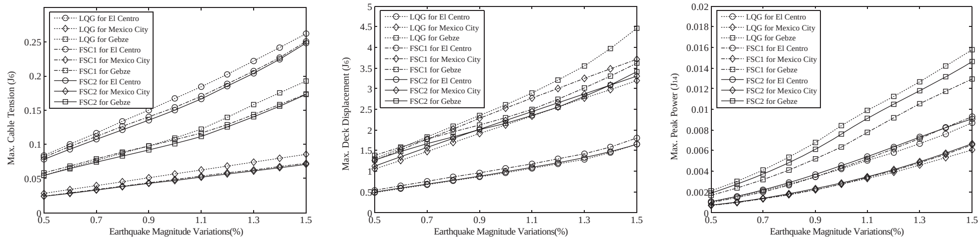

Figure 1. Robust performance of two control systems against variations in the magnitude of excitation.

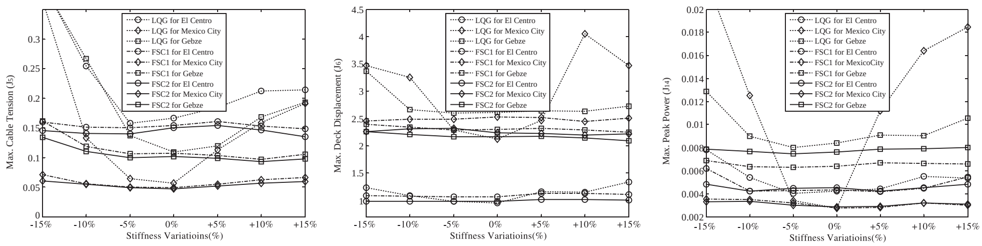

Figure 2. Robust performance of two control systems against variations in stiffness of the bridge model.

Table 1. Control performance of both systems subject to self-excited and randomly generated wind loads.

Performance ratio (%)	02 Cable model		06 Cable model		12 Cable model	
($J_{unc} - J_{LQG\text{ or }FSC}/J_{unc} \times 100$)	LQG	FSC	LQG	FSC	LQG	FSC
Maximum variation in cable	37.49	38.47	70.73	71.33	26.77	29.05
tension	11.59	13.86	57.23	58.50	47.91	49.58
Maximum longitudinal	27.87	28.14	71.28	72.76	38.80	40.99
displacement at the top tower	29.90	31.39	48.35	48.99	39.67	41.07
Maximum longitudinal	27.78	27.94	71.15	72.72	37.98	39.47
acceleration at the top tower	34.66	36.04	55.47	56.32	42.49	43.55
Maximum vertical	29.11	29.71	71.27	72.70	37.70	39.96
displacement of the deck	29.28	30.55	48.91	49.59	40.13	41.54
Maximum vertical	24.94	19.17	71.34	72.99	38.23	34.42
acceleration of the deck	8.88	8.11	61.55	61.63	55.05	54.64

*First line is for the case of the self-excited wind and second one is for randomly generated wind.

event, while stiffness perturbation in the bridge model leads to a significant amplification in the seismic responses of the cable-stayed bridge for the LQG system. In particular, the outstanding robust performance of the FSC2 system demonstrate that the tuning process of the fuzzy rule is able to further enhance the seismic performance of the fuzzy-based variable gain approach for a cable-stayed bridge subject to earthquake excitations. Therefore, the FSC system clearly guarantees excellent robust performance against the uncertainties in the bridge model as well as seismic events, especially through the fine-tuning process of the fuzzy rules.

Table 1 represents the simulated results of both systems when the cable-stayed bridge is subject to the self-excited wind loadings and a set of 100 randomly generated wind loadings. All the values in the table are the ratio of the maximum controlled responses to the maximum uncontrolled responses. Therefore, the control system exhibits better performance as the values of the performance ratio increase. Even though the performance of FSC system is a little inferior to that of LQG system for a few cases of the maximum vertical accelerations of the deck, the FSC system shows better control performance for the rest of the wind-induced responses than LQG system. Furthermore, the maxim level of total power required by FSC system is 0.768 kW, while LQG system requires 0.783 kW. Hence, the presented approach guarantees the overall enhanced control performance for the wind-induced cable-stayed bridge under construction.

The comparative results show that the fuzzy-based variable control approach can efficiently reduce the dynamic responses of cable-stayed bridges while maintaining lower level of control efforts, and clearly guarantee the robust performance against the uncertainties of bridge model and seismic events involved. Therefore, it is concluded that the proposed technique is an effective control strategy for the vibration control of cable-stayed bridges.

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Development of safety warning system for infrastructures

J.S. Lee & G.H. Juhn
Korea Infrastructure Safety and Technology Corporation, Goyang, Korea

ABSTRACT: Everyone knows that structural monitoring system is very useful tool for the measurement of structural behavior and the evaluation of structural condition. Because most of the collapses of the infrastructures are occurred progressively, the monitoring system should catch small abnormal behavior and give warning sign to mankind before the collapse. Up to now, so many monitoring systems were developed and installed in the large infrastructures such as long-span bridges, buildings, dams, tunnels, etc. Most of those systems are very complex and expensive because many sensors and data loggers should be installed in the site and unique software should be developed to handle the data loggers. The aim of these systems is focused on the observation of various structural behaviors and on the verification of design parameters as well as on the safety warning.

During the last 10 years, many sophisticated monitoring systems were installed in the cable bridges, named Namhae, Dolsan, Jindo, Olympic, Seohae, etc, in Korea. Recently, many other cable bridges are under construction and are scheduled to be constructed. Same as those bridges in use, the monitoring systems is scheduled to be installed in these new bridges. Common features of all these monitoring systems are that lots of sensors, such as strain gauges, displacement sensors, tiltmeters, accelerometers, etc, were installed in various bridge members, the unique software was developed for each bridge site and the maintenance office is located near the bridge site where one or more special engineers are charged with the maintenance of the monitoring system. This type of monitoring system is appropriate for the large cable bridges which were constructed at a high price and have structural complexity and importance. Many engineers in the construction companies and installation companies have been trying to catch the various structural behaviors and to find the accurate sensors which can catch structural behavior precisely.

We had thought of watching the structural integrity of infrastructures with simpler monitoring system and had focused the target on mid and small size infrastructure, especially on the old and weak ones. Finally, we developed an integrated monitoring system which consists of the simple and rugged hardware systems and the software watching the conditions of the infrastructures simultaneously in same screen map through the internet.

This system is composed of two parts. One is hardware which is installed in the field. The other is software which is gathering data from loggers in the filed, processing data for evaluating the condition of the infrastructure and displaying the condition in the screen map.

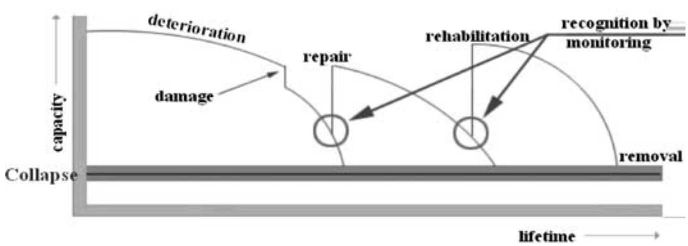

Figure 2. The concept drawing of safety warning by monitoring.

Figure 6. The monitoring system of Kia Bridge.

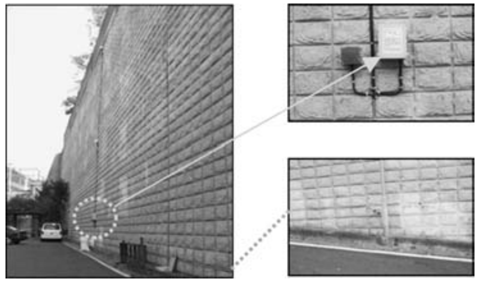

Figure 7. Monitoring system of the retaining wall of Ace Apartment in Ilsan.

Figure 9. Main page of the software.

Figure 11. General information about the infrastructure.

We assumed the collapse mechanism of each infrastructure for the disasters which could affect the safety of the infrastructure. We had studied which behavior we have to measure in order to watch the safety of the infrastructures with this early warning system.

We surveyed and chose the sensors and loggers for this system. We tested the accuracy and the long-term stability of the equipments we chose in the field. It should operate independently and stably in any hazardous environment. So, we chose the wireless CDMA modem for data transmission and the solar cells for the power supply.

The software is developed internet based. Basically, this system is developed for non-specialist who is charged with the maintenance of the infrastructures in local government and local agency of Ministry of Construction and Transportation. A person in charge can use this system as a useful maintenance tool because he can store the result of periodic inspection and make inspection report easily. He can watch the circumstances of the infrastructures, such as damage of structure, traffic condition, water level under bridge, etc, with webcam.

We have a plan of establishing Infrastructure Safety Surveillance and Evaluation Center (ISSEC) in KISTEC. We want lots of old and weak infrastructures to be included in this new system after surveying all of the infrastructures in Korea. We also want to perform continuous monitoring and watch the safety of them in ISSEC.

For the past several years, we had performed the feasibility study with the long-term monitoring system in the field and developed the software which could evaluate the structural integrity of the infrastructures and could watch the safety of the entire related infrastructure easily and simultaneously.

We believe that it is effective and economic way to watch the safety of the infrastructures with this system. This system will be helpful to prepare the instant measures against disaster during and after the event. We will continuously upgrade this system with high-tech equipments and new technology. We hope that lots of old and weak infrastructures will be included in this system and will be being watched by this system for the safe use of those infrastructures.

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Development of strain sensor holders to be applied to the monitoring of metallic structures

B.J.A. Costa, A.O. Diamande & J.A. Figueiras
LABEST, University of Porto, Faculty of Engineering, Porto, Portugal

C.M. Félix
LABEST, Institute Polytechnic of Porto, Engineering School, Porto, Portugal

ABSTRACT: The Laboratory of Structures (LABEST) of the Faculty of Engineering of Porto University (FEUP) has been engaged, in the latest years, in assessing the structural performance of several steel structures. The observed bridges and viaducts structural typology is very rich, gathering recently constructed viaducts and centenary bridges, single and multiple span decks, truss and steel plate girders, metallic and masonry piers. Figure 1 illustrates some monitored structures.

The monitoring systems implemented on those structures made it possible to appraise and control the structural effects of the ongoing construction works, but also allowed to evaluate the structural conformity during the required load tests, carried out before their official opening or the strengthening and rehabilitation process took place. In the early ages of service, the systems have permitted to collect data related to the structural response under the environmental actions and load traffic.

Typically, in this kind of applications it becomes very helpful to keep some samples of the original structural elements of the metallic structures in laboratory, installed with sensors equal to the ones applied on the bridge which is under observation, in order to verify the existence of any phenomenon of sensor degradation with time, afterwards helping to clarify the measurements taken on the site.

Despite the well known good results obtained with the classical application of electric strain gages on metallic surfaces, there are intrinsic problems related to its ordinary procedure, such as the gentleness of some operations, common adverse environmental conditions, weak accessibilities to the points to be instrumented, time consumed on the site and tight construction schedules to attend. To overcome these difficulties, a new based electric strain gages sensor holders was developed, regarding the already existing know-how of embedded fiber optic Bragg grating sensors in carbon fiber reinforced polymer (CFRP).

To ensure the suitable longevity for any monitoring system, special care has to be taken in the sensor installation, mainly if they are directly exposed to the environmental conditions, just as the surface mountable strain sensor holders applied on steel bridges are.

Figure 1. Structures observed with monitoring systems. Left: Pinhão bridge, northern Portugal; Center: Luiz I bridge, Porto; Right: Andresas viaduct, Porto.

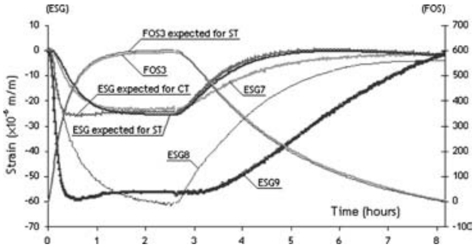

Figure 2. Data collected by the sensors during the temperature cycle.

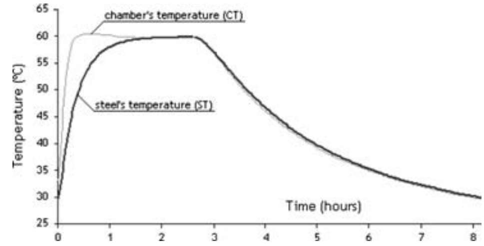

Figure 3. Temperature evolution in the chamber and in the steel.

In this paper, the procedure and the several cares taken during "in situ" sensor installation and protection are also described. The cover applied to the surface mountable sensors provides mechanical protection through the cork layer, which also avoids direct insolation over the sensor surrounding area.

The protection resin and the paint enable the resistance against a variety of environmental factors, such as UV radiations, rain and moisture. Thus, the durability is ensured and the intrusion level introduced on the readings is minimum. Also, the installation overall appearance is very discreet.

During the design stage of the sensor holders one primary geometric parameter that needed to be established was the CFRP length. Not only the thickness and deformability of the gluing resin are mandatory in the adequate transferring of deformation from the material to the sensing element (strain gage or Bragg grating), but also the composite length has an important role in that phenomenon. On the other hand, attending the mechanical properties of the material used to embed the sensing elements, it is convenient to know the deviation introduced on the readings collected by the sensor holders, since the surface mountable sensors after being glued to the steel form a local reinforcement on the point to be measured.

In order to appraise these effects a three-dimensional finite element model was developed, to simulate the behavior of a steel bar instrumented with a sensor holder, under a uniform uniaxial load applied to the edges in the longitudinal direction.

The laboratory tests carried out allowed to appraise the sensor holders behavior, when structural elements instrumented by them were subjected to different actions. These experiments also enabled to validate the installation and protection procedures considered. On the other hand, some tests made possible to verify sensor holder dimensions, estimated from numerical analysis previously made.

Often, not only during its service lifetime, but also when the structures are constructed, it is important to know the stress overall evolution on bridges. Thus, the strain average level experimented by the observed points can be significant and have a strong long-term nature, demanding a good behavior of the glue to creep.

The temperature is one of the most important actions that rules the bridge structural behavior throughout its normal service life. This environmental load acts through daily cycles but also exhibits seasonal cyclic variations. To appraise the performance of the sensor holders used in the monitoring systems and the efficiency of their protection, thermal variations were applied to samples recovered from the observed structures.

The hygroscopicity of composite materials made of epoxy resins is well known. Since the sensor holders and their protections are made of such materials, it is essential to appraise the effect of moisture on the readings experienced by the sensors. Consequently, the effectiveness offered by the adopted protections is mandatory.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Design and installation of the optic based monitoring system applied to the Luiz I Bridge

B.J.A. Costa & J.A. Figueiras
LABEST, University of Porto, Faculty of Engineering, Porto, Portugal

C.M. Félix
LABEST, Institute Polytechnic of Porto, Engineering School, Porto, Portugal

ABSTRACT: The Luiz I bridge is a unique example of the Portuguese industrial history in the late XIX century. It was designed by Teophile Seyrig, a Belgian engineer disciple of Gustave Eiffel, and constructed by the Société Anonyme de Constructions et des Ateliers de Willebroeck. The structure has been in continuous operation since October 1886.

The bridge is constituted by a metallic double hinged arch, supporting simultaneously two decks at different levels over the river crossing. The arch has a parabolic geometry both in vertical and plan views, presenting 172 m long and 45.1 m of maximum height, with a variable apparent vertical thickness from 16.7 m near the supports to 7.1 m at the crown. Two continuous truss girders, attached to the crown arch and sustained by roller supports over seven piers and the abutments, materialize the different 13 spans of the upper deck, performing a total length of 391.25 m.

The lower deck, 8 m wide and 174.3 m long, is supported at the ends and suspended from the arch by four ties 36 m apart. The several cross sections of the truss elements composing the bridge structure were achieved by using one or more steel plates of different thickness, joined with angles and rivets.

Despite some operations of rehabilitation and maintenance or minor changes suffered to accommodate the passage of new types of vehicles, the structure has been in continuous operation since its conclusion date in October 1886. However, lately a strengthening and rehabilitation process took place on this bridge, in order to allow the integration of its upper deck in the infrastructure of the Porto Metro Network.

One of the major operations fulfilled during this process was the replacement of the upper deck bridge concrete pavement, resting on steel beams, by a suitable metallic profile grid, able to transmit properly the new railway traffic loads to the truss girders. The structural elements strengthened with addition of steel profiles were the upper deck girders, the suspension ties legs, arch diagonals and the existing bracing elements scattered all over the bridge. All the pieces and structural components with severe corrosion problems were replaced. The metallic surface protection, for both new and original steel, was accomplished by a 3-layer epoxy painting cover, demanding previously the removal of the old coat in the remaining structural elements by using hydro-scouring.

The gentleness of some of these operations, the appraisal of the strengthening solutions performance and the significance of this historical bridge, made the structural monitoring of its behavior mandatory. An advanced monitoring system based on the fiber optic Bragg grating sensors was applied to the Luiz I bridge to permanently assess the service performance and the structural safety. This system was designed to be installed during the construction process, taking advantage of the general scaffolding mounted for the bridge rehabilitation, as shown in Figure 1.

In order to assess the relevant strains, 118 optic sensor holders were applied to 59 sections of the arch, upper deck, piers and ties. Ten temperature sensors appraise the environmental and steel temperature. The main relative displacements between the bridge upper deck and the abutments/piers are collected from 8 optic displacement transducers.

Figure 1. Overall view of Luiz I bridge during the ongoing strengthening and rehabilitation works.

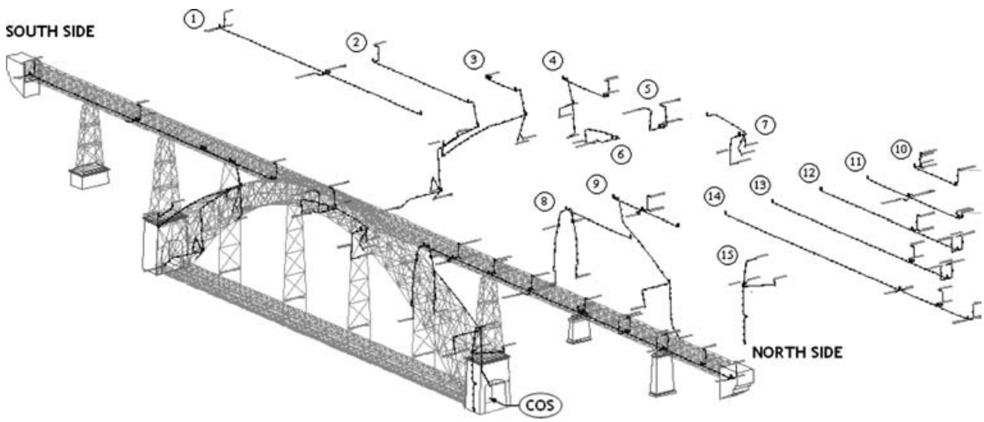

Figure 2. Location and development of branches belonging to the fiber optic network.

The fiber optic network has a tree configuration with a main optical cable with derivation to each of the branches containing up-to 10 sensors connected in series. In total, more than 210 m of main cable was deployed along the bridge, 150 m in the upper deck and the remaining between this and the COS, located nearby the arch north support. The sensors are multiplexed into the corresponding channels by using patch cords, mechanically connected in the boxes/enclosures placed at specific points of the bridge.

To achieve suitable longevity for the monitoring system, proper methodologies and materials were used to accomplish the installation and protection of its several components, namely the accommodation of the optic cables and fibers, the sensors installation and protection and the right selection of fiber optic network components.

The swept laser interrogator and the optical switch allows to collect raw data from all sensors and transducers, with a scanning frequency of 2 Hz, suitable to assess the structural response due the metro slow crossing along the upper deck. Dynamic readings can be accomplished by changing the acquisition program with a default reading period of 5 minutes. After a proper data treatment, the strain, temperature and displacement measures are provided to the client through a website portal, specially developed for this monitoring system.

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Design and implementation of the new structural monitoring system of the Tagus river suspension bridge

J. Rodrigues, J. Almeida Garrett, C. Oliveira Costa & P. Silveira
LNEC – National Laboratory for Civil Engineering, Lisbon, Portugal

1 INTRODUCTION

The first phase of the construction of the Tagus river suspension bridge in Lisbon was completed in 1966. At that time the bridge had only 4 lanes of roadway traffic, although a second phase of construction was already planned in which a railway deck would be added to the bridge. The second phase of the construction could only be putted into practice in the end of the nineties with the planned addition of two railroad tracks at the lower level of the stiffening truss and the widening of the roadway deck to 6 lanes which wasn't considered in the initial plans.

In both phases of construction, LNEC performed reception load tests on the bridge after the end of the construction works, involving static and dynamic measurements. After the first phase of the construction in the sixties, the structural behavior of the bridge was also monitored with a system developed at LNEC that included several unique and innovative equipments for that time. This first structural monitoring system was in service until 1979.

More recently, LNEC was requested to develop a new structural monitoring system in order to provide adequate information about the structural condition of this important infrastructure. This request created an interesting challenge to the team involved in the development of the system, since, from the beginning of the work, there was the purpose of developing a state-of-the-art structural monitoring system. Although interesting, the challenge was also a difficult one, mainly due to the specific characteristics of the Tagus river suspension bridge, namely its total length, the fact that it carries both roadway and railway traffic and its general importance as an infrastructure to the Lisbon's metropolitan area. The experience coming from the fact that the first system from the sixties was also developed at LNEC was an advantage to the team. However, considering the technological progress that took place since the development of the first system, especially in what concerns the data acquisition and communication equipment and, of course, the data processing capacity of computers, there was the justified intention of improving what has been achieved with the first system.

This paper discusses the design and implementation of the new structural monitoring system of the Tagus river suspension bridge, considering the transducers, data acquisition and communication equipment as well as the data analysis procedures that are being adopted in its development. This monitoring system involves the suspended steel structure and also the approach viaduct with a concrete box girder for the roadway deck and a composite double plate girder for the railway deck.

2 DESCRIPTION OF THE TAGUS RIVER SUSPENSION BRIDGE

The Tagus River suspension bridge in Lisbon has a main span of 1013 m, two suspended side spans of 483 m, and three backstay spans of 100 m, 100 m and 99 m for a total length of 2278 m between anchorages. An elevation of the Tagus River suspension bridge is presented in Figure 1. On the North side, the bridge has a prestressed cantilever box girder approach viaduct that brings the total length of elevated structure to 3223 m.

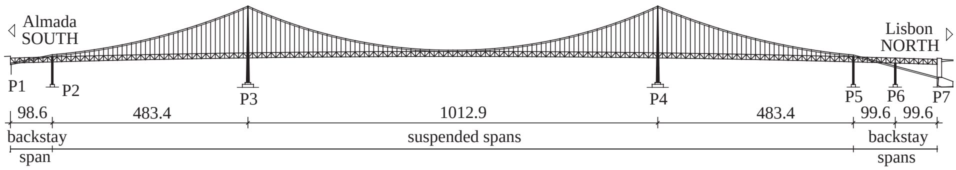

Figure 1. Elevation of the Tagus River suspension bridge.

3 STRUCTURAL MONITORING SYSTEM

The new structural monitoring system of the Tagus river suspension bridge has several modules. The first one consists in the transducers, or sensors, which are used to measure several physical quantities related to the structural response, the environmental conditions or the loads to which the structure is subjected. The power supply and signal conditioning equipment is also included in this first module.

The second module consists in the data acquisition and communication equipment. At this module, the signals coming from the transducers are transformed to digital format, which is manageable to be treated and stored by modern day digital computers. For the equipment that is going to be installed in the Tagus River suspension bridge system, the data acquisition is closely related with its communication, thus the data communication is also considered as a part of the second module.

The third module consists in the data processing methodologies which are applied to check, transform and reduce the data collected with the transducers and the data acquisition and communication systems.

The forth module is basically the methodologies which are used to store the processed data so that it can be easily accessed for further analysis and interpretation.

Finally, a fifth module is considered, where, based on observed data and also on results from analytical and numerical models, there is an interpretation of what actually is the condition of the structure (diagnostics) so that adequate decisions are taken by LNEC and by the end-users of the system, EP-EPE and REFER. This process of interpretation, diagnostics and decision making, may involve some interaction with the database, where the observed data is saved, and also with the developed analytical and numerical models.

Within the architecture of the system, visits to the Tagus Bridge are also considered for installation and on-site configuration of equipment. This is, of course, necessary in the initial installation of the system, but, most probably, will also be needed during its operation, to check and, eventually, replace transducers and/or modules of the data acquisition equipment.

The installation of additional transducers, that aren't considered at this stage of development of the system, may be adopted as a result of the interpretation of the observed data (for instance to instrument with more refinement a particular area of the structure where some type of problem is suspected to be occurring). Expandability of the system is therefore one important characteristic that is being taken into account.

The remote access to the system is also an available feature, to allow the remote configuration of the data acquisition equipment and of the data processing methodologies.

One key aspect, which is kept in mind in the design of the new structural monitoring system of the Tagus River suspension bridge, is the adequate integration and balanced development of all its modules. It might be tempting to install a large number of transducers in a given structural monitoring system, as far as the available resources permit, however it is only worthwhile to do so, if the system includes also adequate processing and analysis methodologies that are able to transform the observed raw data in meaningful and useful information for the end-users of the system. That information should help the end-users in taking decisions concerning the management of the infrastructures under their responsibility. To address this problem of system modules integration and balanced development, it is quite important to have a multidisciplinary team working in their development.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Health monitoring of large Adriatic bridges

J. Radić & J. Bleiziffer
Structural Department, Faculty of Civil Engineering, University of Zagreb, Croatia

G. Puž
Department for Development and Planning, Croatian Motorways Ltd., Zagreb, Croatia

1 INTRODUCTION

Condition monitoring, assessment and maintenance are considerably neglected in Croatia with respect to investments in new construction, but recently the attention of bridge owners and professional community is starting to turn to methods and techniques for preserving the bridges' load-carrying capacity and serviceability. These include attempts to create and implement a comprehensive bridge management system that would provide insight into the present condition of the country's bridge stock. Systematic approach to maintenance planning becomes essential as costs of preserving safety and serviceability of structures start to prevail over new investments and bridge management systems are developed for this purpose. These are well suited to data collection and analysis on network level. However, the project level management, particularly of large and significant bridges, requires bridge-specific maintenance programme. Health monitoring techniques are essential tools for efficient bridge management and maintenance.

2 LARGE ADRIATIC BRIDGES

Six large reinforced concrete arch bridges were built at Croatian Adriatic coast over the last four decades – Šibenik bridge, Pag bridge, Krk bridges (Krk I and Krk II bridges), Maslenica bridge and Skradin bridge over Krka river. While Šibenik, Pag and Krk bridges have been in use for 25–40 years, Maslenica and Skradin bridges were constructed much more recently.

2.1 *Inspection & monitoring*

The performance in service of four older arch bridges – Šibenik, Pag, Krk I and Krk II bridges, cannot be deemed satisfactory. Many problems and deficiencies were encountered during their lifetime, resulting in huge, complex and expensive repair works. Unfortunately, monitoring the bridges' performance was not continuous activity, but was carried out on the "as-needed" basis with inspection and testing undertaken only when structural damage was evident and urgent repairs were needed.

Many problems encountered in performance of four older Adriatic arch bridges and huge fi-nancial resources spent on solving these problems, urged for different approach in planning monitoring the performance of bridges built from 1990 onwards. For the first time in Croatia, a monitoring system for long-term control of stresses, deformations and corrosion progress was installed on Maslenica bridge, which was completed in 1997. The system was used to record the stresses and strains at different construction stages and under load-testing prior to opening bridge to the service. Unfortunately, the funding of the monitoring project was stopped soon aftewards. More recently, another attempt was made to ensure continuous data on structural performance of an Adriatic arch bridge, with the instrumentation of Skradin bridge, that was opened to traffic

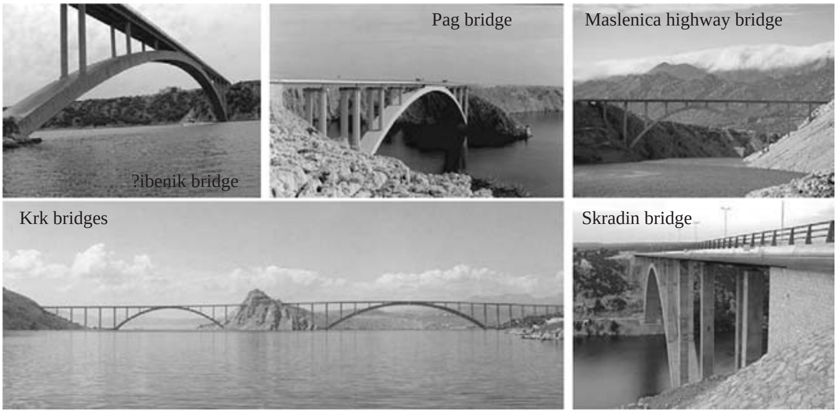

Figure 1. Six large reinforced concrete Adriatic arch bridges.

in 2005. Continuous monitoring shall provide relevant data for modeling concrete deterioration mechanisms, thus enabling more precise service-life design of future struc-tures that will be built in the Adriatic region.

2.2 *Degradation and service life modelling*

Among a set of nominally similar bridges the rate of deterioration and the need for maintenance vary significantly from one bridge to another, thus stochastic modelling is required. Among the first stochastic models was the one for forecast of the future rating based on Markov Chain theory. An attempt to apply Markov chain model to the data on Croatian bridges failed, because of difficulties related to the calculation of transition probabilities as inspections were not performed in regular time intervals. Since Markov chain operates with discrete time periods, the data set, which is considerably largely spaced for Adriatic bridges discussed in the previous section, could not be used. New model, using homogenous Markov process was therefore developed.

This new deterioration prediction model using homogenous Markov process was applied to large Adriatic reinforced concrete arches. Two examples are discussed in the paper: service life prediction of the repaired Krk bridge spandrel columns and estimating the remaining service life of the arch of the Šibenik bridge.

3 CONCLUSIONS

Large or significant bridges require structure specific monitoring and maintenance programme. Various monitoring techniques were employed over decades to determine structural performance and durability related performance of large Adriatic arch bridges. So far, it is impossible to completely substitute visual inspection with other measurement methods. Unfortunately, these depend largely on professional experience and engineering judgment of inspection personnel. More sophisticated monitoring systems enable gathering data on a real-time basis. Currently in Croatia best results, from both economic and engineering standpoint, can be obtained by combining two approaches, but much should be done on improving timely planning of monitoring activities and their careful implementation.

In order to predict future deterioration from available scattered data on Adriatic bridges' structural condition, a stochastic model that uses homogenous Markov process with continuous parameter (time) was developed. Forecasts are expressed as a distribution of bridge elements (or bridges) among condition states. Such procedure enables us to investigate the optimization of lifetime maintenance costs.

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Cable stayed bridges. Failure of a stay: Dynamic and pseudo-dynamic analysis of structural behaviour

C.M.M. del Olmo
University of Castilla La Mancha, Ciudad Real, Spain

A.C.A. Bengoechea
Technical University of Catalonya, Barcelona, Spain

ABSTRACT: This work is a study of the structural answer of cable stayed bridges when the sudden loss of one of their stays takes place. The analysis has been accomplished through two different methods: by means of a dynamic analysis and carrying out a procedure, included in the P.T.I. (2000) and S.E.T.R.A. (2001) recommendations, which consists of developing a static analysis in which the dynamic action is replaced by a static force increased through a dynamic amplification factor. The different results of bending moments in deck and pylons and tension force in stays obtained by these two methods are compared and analysed.

1 INTRODUCTION AND ANALYSIS DESCRIPTION

The design of a cable stayed bridge requires to consider the possibility of a sudden failure of a stay. Up to now, the codes and recommendations on the topic tackle this accidental situation by means of an equivalent static analysis and using a dynamic amplification factor. Thus, for example, the P.T.I. (2000) recommendations indicate that, in the equivalent static analysis, the dynamic effect of the loss of a stay will equal the tension force that the stay bears multiplied by an amplification factor equal to 2.0. In 2001, the french SETRA published a set of recommendations, S.E.T.R.A (2001), for the design of cable stayed bridges in which the same method was applied, but using a factor between 1.5 and 2.0. Since the intention of all this is to replace a dynamic analysis with a static one, in the future we will call this type of analysis Pseudodynamic. In the present work, four different models of cable stayed bridges will be studied considering the failure of one stay, and the results come out applying the criteria just mentioned are compared to those obtained using a dynamic analysis. In each of the four bridges, the case of the sudden loss of one of the stays of the lateral span and one of the principal span is studied.

2 DESCRIPTION OF THE STUDIED BRIDGES

This study has been carried out on four three-span cable stayed bridges, with concrete deck and pylons, for whose definition the bridges used by Rene Walther in his parametric study included in Walther (1999) have been taken into account. Each one of these four bridges is perfectly defined in relation to the rest through the layout of its stays (either in a fan or in a harp pattern) and through the section of its deck: deck Type A (slab deck) or deck Type B (cellular deck). In the four models studied, the total length of the bridge is 403 m, spaced out in one principal span and two lateral span, as Figure 1 shows. The space between stays in the deck is 6.2 m.

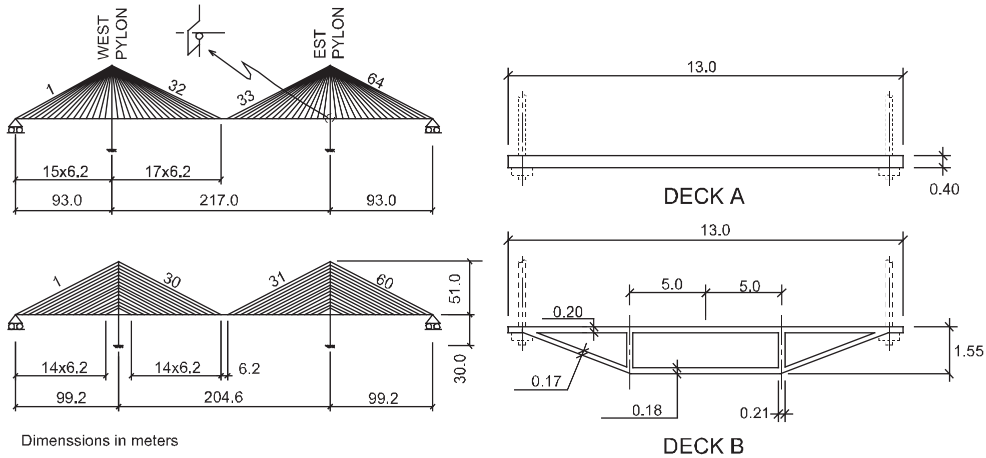

Figure 1. Longitudinal layout of the bridges, numbering of stays and sections of the decks.

3 RESULTS AND CONCLUSIONS

In view of the results obtained, which are detailed in the full paper, we can conclude that in the cases studied:

- The pseudodynamic analysis included in the P.T.I. (2000) and S.E.T.R.A (2001) recommendations shows values of the bending moments in the deck higher than those obtained through a dynamic analysis only in the sections placed near the anchorage of the lost stay. In those sections placed away from this area, the dynamic analysis shows, in some cases, values 2.0 and 4.2 times higher than the pseudodynamic does. In addition, the pseudodynamic analysis does not consider the moments of an opposite sign that the deck faces as a consequence of the oscillations caused by the stay failure.
- The bending moments in all the sections of the pylons in the pseudodynamic analysis are lower than the results obtained through the dynamic analysis. In the fixed sections, the dynamic analysis also shows, in some cases, moments of a sign opposite to that established by the pseudodynamic analysis and of an important magnitude.
- The tension force in the stays, according to the pseudodynamic analysis, is higher to the dynamic one only in those stays placed near the lost one.

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Monitoring of a bridge-deck using long-gage optical fiber sensors with a pulsed TOF measurement technique

V. Lyöri, A. Kilpelä, G. Duan & J. Kostamovaara
University of Oulu, Department of Electrical and Information Engineering and Infotech Oulu, Oulu, Finland

T. Aho
University of Oulu, Research Unit of Construction Technology, Oulu, Finland

ABSTRACT: Building stock forms a remarkable part of a nation's capital. In Finland, for example, the figure lies at about 70% (www.rakli.fi). Still, budgeted costs for structural health monitoring and repair are often inadequate with respect to the need, although continuous health monitoring would enable property owners to optimize the repair time schedule, thereby decreasing costs and improving public safety. Health monitoring includes, for example, measurements of deflection, displacement, crack width and location, acceleration, humidity and temperature.

Strain is usually measured by a point sensor with a gage length ranging from a couple of millimeters to a few centimeters. Typical point sensors, such as Bragg gratings and Fabry-Pérot sensors, have a wide application range due to their high multiplexing capability and extremely good measurement accuracy (Measures. 2001). As a consequence of their short gage length, however, point sensors can only produce micro-scale information and cannot be applied to measuring such structural deformations as deflections or displacements between two relatively distant points. The behavior of a structure with an inhomogeneous strain distribution can be more reliably measured by using a long-gage sensor with a gage length from a few tens of centimeters up to several meters. Typical constructions of this kind include large civil engineering structures like bridges and dams and different composites such as containers, pipes and windmill or airplane wings.

Long-gage sensors are usually based on the detection of Brillouin scattering (Bao et al. 2001), on low-coherence interferometry (Inaudi et al. 1999) or on the micro-bending effect (Braunstein et al. 2002). The approach discussed here utilizes a pulsed time of flight measurement technique (TOF) that is used in conventional laser rangefinders for profiling large building blocks, for instance (Määttä et al. 1993). New electronics design and optical fiber sensor structures enable an averaged precision of a few micrometers (rms-value) and sampling frequency of a few herz simultaneously, which represents to our knowledge the state-of-the-art in pulsed TOF laser radars. Moreover, the developed TOF measurement system allows static and dynamic measurement for a series of integral sensors, independently of the sensor length, which is a unique feature among measurement systems of this kind. In addition, the compact structure of the device, realized mostly with standard telecommunications components with no moving parts, makes it an attractive tool for many applications requiring continuous structural health monitoring at a reasonable cost.

In this paper the operation and basic blocks of the developed TOF measurement system are presented together with measurement results of a field demonstration at a bridge over the River Siikajoki in Finland. Figure 1 shows the principle of a deflection measurement whereas Figure 2 represents the corresponding measurement results under traffic and thermal loading. Both in dynamic and static measurements, the system's response is nearly identical with that of a commercial Bragg grating measurement system. This kind of performance makes it a potential competitor for the measurement systems already on the market.

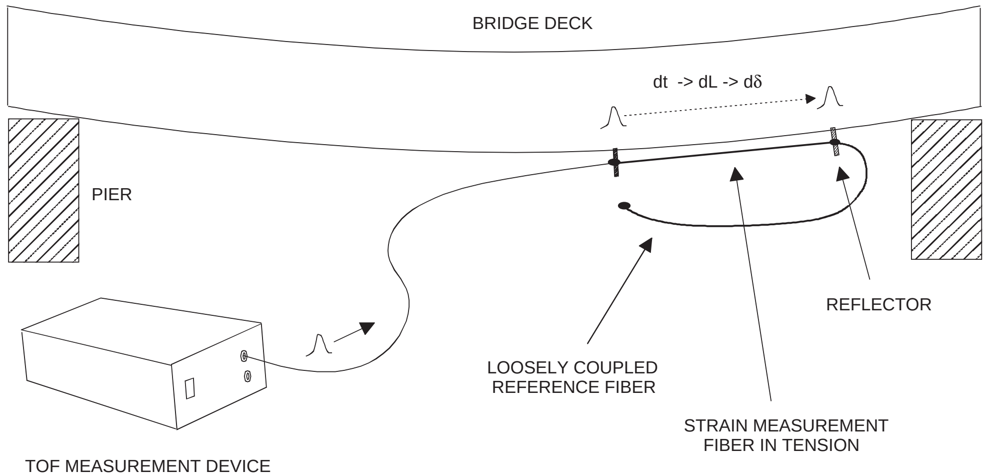

Figure 1. Principle of the deflection measurement using long-gage fiber sensors and the TOF technique.

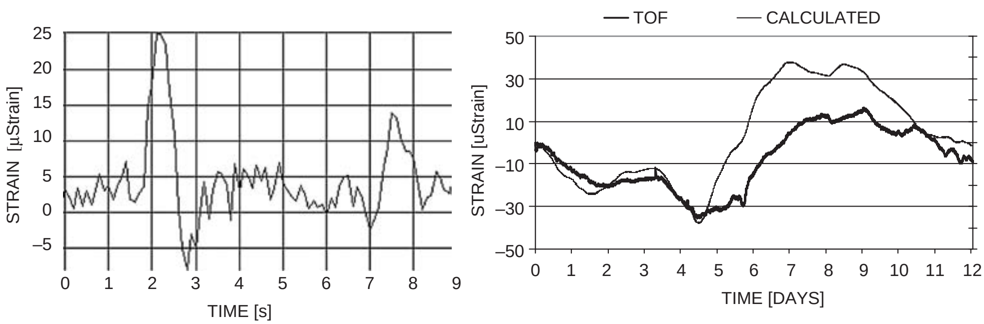

Figure 2. Strain in the bridge deck under traffic loading (left). Thermal elongation of bridge deck. TOF measurement curve and calculated model (right).

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*Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Live-bed bridge scour monitoring system development using fiber Bragg grating sensors

Yung-Bin Lin
Center for Research on Earthquake Engineering, Taipei, Taiwan

K.-C. Chang
Department of Civil Engineering, National Taiwan University, Taipei, Taiwan

Jihn-Sung Lai
Hydrotech Research Institute, National Taiwan University, Taipei, Taiwan

ABSTRACT: Bridge scour is one of the leading causes of bridge failure. The exposure and subsequent undermining of the pier/abutment foundations through the erosive action of a flood can result in the structural failure of a bridge. Bridges that are subject to periods of flood/high flow require monitoring during those times in order to protect the traveling public. In this paper, an innovative live-bed scour monitoring device using a fiber Bragg grating (FBG) sensor was developed and demonstrated in-situ during the Aere typhoon flood. In the present study, the in-situ FBG scour monitoring system is useful for real-time local scour depth measurements, and it has proven to be valid for indicating the sedimentation depth during the storm flood period at the Dadu Bridge. Based on the analysis herein, the evidence proves that the proposed FBG system can function well and survive through the flood event. The system is durable and reliable. It has been demonstrated to be successful for real-time scour-depth measurements in the processes of scouring and deposition. The field results indicate that this in-situ scour monitoring system using fiber Bragg grating sensors has the great potential for practical applications to the erosion problems around hydraulic works. The scour measurements obtained at the Dadu Bridge are also valuable for further usage to study the maximum scour depth and scour rate with various sediment properties under different flow situations. The in-situ information collected herein is very helpful in assisting engineers to design more accurate, safer and more cost-effective bridges.

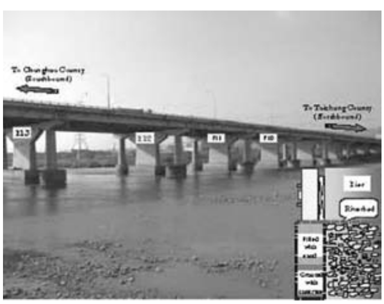

Figure 1. The Dadu Bridge spans two major counties over the Wu River.

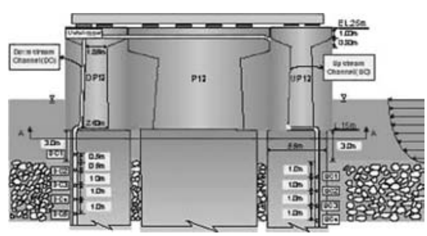

Figure 2. The layout of the FBG scour monitoring system on P12.

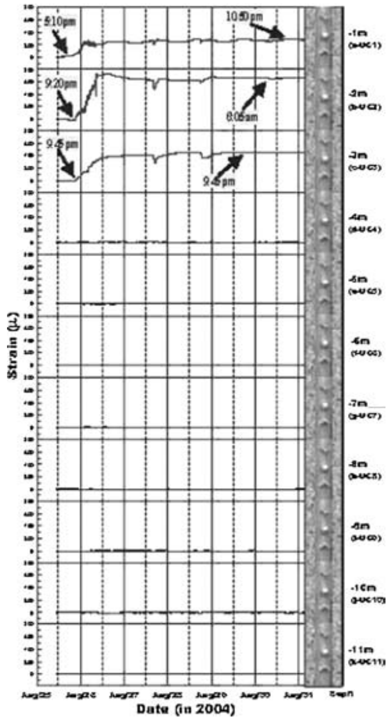

Figure 3. Strain readings against time along UC during the Aere typhoon flood.

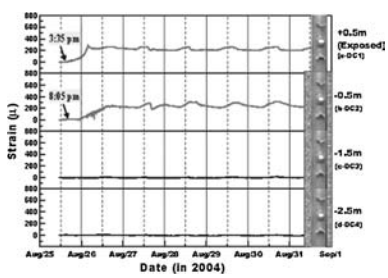

Figure 4. Strain readings against time along DC during the Aere typhoon flood.

Figure 5. Floating debris accumulation at upstream of UP12 in the flood.

Figure 6. The local scour holes around Pier 9 and Pier 8 after flood cease.

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Assessment and monitoring of cable stayed bridges

E. Laurent
Advitam, Paris, France

1 INTRODUCTION

New technologies are permanently under development in the Civil Engineering world to push forward the limits of construction. Several decades before, the construction of a bridge like Rion-Antirion was only a dream due to the high environmental difficulties to build a bridge at this location over a fault. Nobody would have thought being able to build a 3.3 km span suspension bridge like the Messina Straight Bridge. The progress of the science allows every day to understand better the behaviour of the elements thanks to numeric simulation and calculation tools that are made available through the increase of the power of the computers. Very efficient calculation tools are available to any designer that has the right skills/experience to perform the work. Pushing forward the limits means to come closer and closer to the origin of the limits (earthquake consequences simulations, wind behaviour simulations, …) and for this goal the laboratory tests are very valuable. The counterpart of it is that the designs are very specialised and focussed on specific managements of risks, and this has to be done based on specific hypothesis. If one of these hypothesis appear to be wrong after having built the structure, or should any important characteristic change through the lifespan of the structure, then the behaviour of the structure will not be according to what has been expected, and therefore the life of the structure might be reduced, with high evident financial consequences due to the increased maintenance costs, or earlier renewal of the facility. Building a structure like Messina Bridge has its own intrinsic risks, like for example flutter. Although these aspect are well understood, and designers perform all the state of the art calculations to prevent it, it remains that the bridge has to behave like it has been designed for during strong winds, and that a careful follow up of the essential elements that are necessary for the correct behaviour of the bridge is a critical issue to manage the flutter risk.

Designers are getting more and more involved for the consequences of a bad design, or a risk that has not been handled. The analysis of the redundancy, or the risks of collapse of a structure is also critical when talking about structures that could involve public injuries. This type of approach, although well known in the industry, is now becoming more and more needed in the Civil Engineering.

This paper intends to detail the methodology that Advitam has developed on high risks structure to analyse their risks, and define the management of their maintenance.

2 RISK ASSESSMENT OF A STRUCTURE

Handling the risks of a structure is a work that is done naturally by any Design Engineer that has the design criteria given by the project documentation. For most of the common structures, design criteria are conservative, and leave no risk apart. Although in terms of maintenance strategy, most common structures are inside typical inspection policies, that generally allows detecting the risks before any bad event occurs, for special structure, the common regulations for inspection and maintenance are not applicable. Typically they don’t consider the specific risks of the structure, and they require extensive investigations on areas where no risk could have an impact on the function of the structure, or where risks could be detected by much cheaper ways. For those reasons common regulations are not applicable for large structures.

Talking about long term cost management, each risks to which the structure is submitted calls for specific dedicated actions, that allow reducing the cost impact of the risks. Simulations have been performed on structures that shows that significant saving of money are generated by a proper risk reduction, and increased knowledge of the condition of the structure.

Although almost every risk has a name, and is well known in design phases, their management rarely appeals to rational techniques. The risk assessment methodology is therefore a good reply to anticipate, according to the knowledge at a given time, the risks to which a structure will be submit.

3 MONITORING SYSTEMS AND INSPECTION AND MAINTENANCE MANUALS

3.1 *Definition of a monitoring system*

The question of the definition of an efficient monitoring system is very constant. Managers complain with the fact that monitoring systems provides non comprehensive readings. Monitoring systems are providing values, but in some cases, those values are not of any interest to give to the managers good orientation for the actions to be undertaken. In too many cases monitoring systems that are installed on the structures are not used as there isn't any clear procedure, or process to be done with the readings. The transfer of knowledge between the designer and the manager is not properly done.

Regarding this last aspect, the risk analysis is of great help. It allows defining the monitoring systems according to the pertinent indicator, and information that are required to qualify a risk.

3.2 *Definition of the Inspection and Maintenance Manual*

The Inspection and Maintenance Manual is the only operational document that will remain after the construction of the structure and that is liable to contain all the important data allowing a long term management of the structure and its risks. The information contained into this document has to be organised to be easy to read, and synthetic.

The methodological risk based approach that has been implemented by Advitam on various major bridges is realised in the continuity of the risk analysis performed to define the monitoring system. The risks not handled by the monitoring system will be incorporated into the Maintenance Manual, and therefore no risk will be left apart.

3.3 *Long term management*

Considering the quantity of information to be reliably managed on a large structure, the definition of rules is a priority to avoid spending too many resources in useless data. Traditionally, standards and regulations applicable for small size structure give baselines for the interpretation of the data, and also for the ranking of the elements of bridges. Every code integrates its own transcription of a risk based management system, but that has to fit any type of structure, regardless of its complexity.

The realisation of a dedicated structural risk assessment allows defining the critical functions of the elements of the structure, and also defining specific risk ranking according to the observations.

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In-situ materials analysis for health monitoring of bridges

M. Ghandehari
Brooklyn Polytechnic University, New York, USA

ABSTRACT: The potential of integration of functional polymers and optical wave-guides as tools for in-situ analysis of the internal environment of civil infrastructure materials is discussed. In such hybrid sensing systems, analyte recognition is carried out through the selection process inherent within the waveguide materials, and the transduction is carried out by the guided electromagnetic radiation, which is spectrally modulated according to the level of the target species present. Summary of work on detection of moisture, pH, oxygen, oxidation states, as well as identification of products of materials dissolution are discussed.

1 A DISTRIBUTED CHEMICAL ANTENNA

Optical fibers can be made to sample the environment in which they are embedded. They can be functionalized and tuned either at the fiber tip or along the fiber surface to detect a wide range of chemical species. Work presented in this article employs the latter method, through optical surface waves known as evanescent waves. Such transduction mechanism, which resembles a distributed optical antenna, allows for sampling the environment in a distributed fashion along the fiber. Unlike the optical fibers used in telecommunications, whose evanescent fields do not interact with the external environment, the modified optical fiber cladding described in this report serves as a zone of interaction between the light and the analyte. In other words, the modified optical fibers are designed so the spectral features of the light traveling within the fiber is modulated by the media it encounters within a permeable cladding

In the technology presented in this paper, the environment surrounding the fiber is interrogated by light in the visible and near-infrared electromagnetic (EM) energy. This choice of wave lengths is based on the availability of compact, robust and relatively low-cost optical components and fiber materials as well as the long term durability of the sensing probe.

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Structural system identification in time domain using a time windowing technique from measured acceleration

S.-K. Park & H.S. Lee
Department of Civil Engineering, Seoul National University, Seoul, Korea

ABSTRACT: Immediate safety assessment of structures after an earthquake is extremely important in evaluating serviceability and functionality of social infrastructures. Nowadays, not only ground acceleration but also acceleration of important social infrastructures is monitored during earthquakes. It would be very helpful for quick restoration of social activities if structural damage caused by an earthquake is assessed with the measured acceleration during an earthquake in real time or near real time.

The discretized equation of motion of a structure subjected to ground acceleration a_g caused by an earthquake is expressed as follows.

$$\mathbf{Ma} + \mathbf{C}(\mathbf{x}_c)\mathbf{v} + \mathbf{K}(\mathbf{x}_s)\mathbf{u} = -\mathbf{Ma}_g \tag{1}$$

where M, C and K represent the mass, damping and stiffness matrix of the structure, respectively, and a, v and u are the relative acceleration, velocity and displacement of the structure to ground motion, respectively.

In case, system parameters vary with time, the time window technique is proposed. Figure 1 illustrates the time window concept. In this technique the minimization problem for the estimation of the system parameters is defined in a finite time interval, which is referred to as a time window.

$$\underset{\mathbf{x}}{\text{Min}}\ \Pi_E(t) = \frac{1}{2}\int_t^{t+d_w} \left\|\tilde{\mathbf{a}}(\mathbf{x}(t)) - \bar{\mathbf{a}}\right\|_2^2 dt \quad \text{subject to} \quad \mathbf{R}(\mathbf{x}(t)) \le 0 \tag{2}$$

Here, t and d_w is the initial time and the window size of a given time window. It is assumed that system parameters are constant in a time window, and that system parameters estimated by (2) represent the system parameters at time t. As the time window advances forward sequentially in time, the variations of system parameters in time are identified.

A regularization technique is used to alleviate the ill-posed characteristics of inverse problems. In case, system parameters may vary abruptly (Fig. 1) with time during earthquakes due to damage, a regularization function that can accommodate piecewise continuous functions in time is required to access damage that occurs during an earthquake. To represent discontinuity of system parameters in

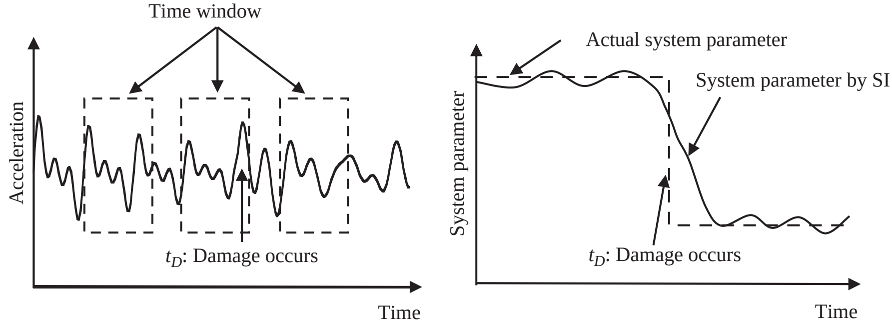

Figure 1. Time window concept.

time, this paper proposes an L_1-regularization function of the first derivative of system parameters with respect to time.

$$\Pi_R(t) = \frac{1}{2}\int_t^{t+d_w} \left\| \frac{d\mathbf{x}}{dt} \right\|_1 dt \tag{3}$$

where $\|\cdot\|_1$ representing the 1-norm of a vector.

It is crucial to determine a proper truncation number so that the TSVD produces a numerically stable and physically meaningful solution. In case a truncation number is too small, most of the useful information on a structure is lost while too large a truncation number yields noise-polluted, meaningless solutions. Therefore, the truncation number should be determined so that useful information of a structure is retained as much as possible while noise-polluted solution components are truncated.

The optimal truncation number may be defined as the truncation number associated with the smallest singular value that does not amplify noise in measurement. The bilinear fitting method (BFM) is proposed to determine the optimal truncation number for the TSVD in this paper. Figure 2 illustrates schematically the variation of the residual of the error function with truncation numbers. As the truncation number approaches an optimal truncation number, the residual of the converged error function decreases very quickly since the number of useful solution components without amplified noise increases. Once the truncation number becomes larger than the optimal truncation number, the decreasing rate of the error function suddenly reduces because noise-polluted solution components associated with smaller singular values are included in solution components. Based on this observation, the plot of the residuals of the error function versus the truncation number is represented by two straight lines, i.e., a bilinear function as shown in Figure 2.

Numerical simulation studies are presented to illustrate validity of proposed method. Numerical simulation study is performed through a two-span continuous truss in Fig. 3.

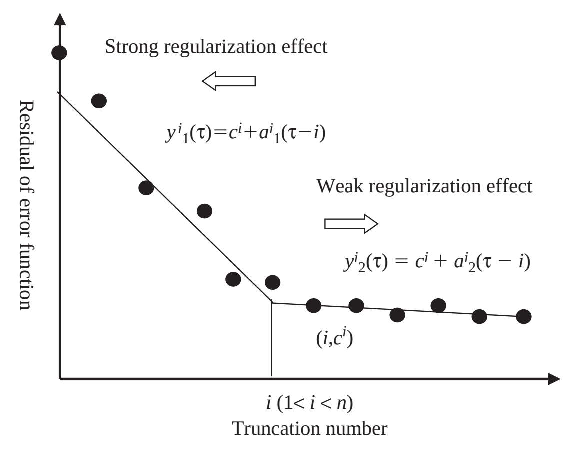

Figure 2. A schematic illustration of the bilinear fitting method

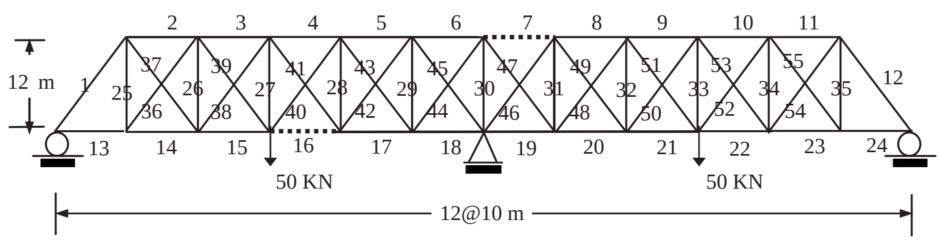

Figure 3. Two span continuous truss.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Toward more practical BMS: Its application on actual budget and maintenance planning of a large urban expressway network in Japan

M. Nishibayashi, N. Kanjo & D. Katayama
Hanshin Expressway Company Limited, Osaka, Japan

ABSTRACT: Hanshin expressway is an urban expressway network accommodating more than 900,000 vehicles per day. About 90% of its structures are viaducts and, as other highways in developed countries, those aging and deteriorating bridge structures have to be maintained with limited budget. This is a sufficient catalyst of making of its own BMS.

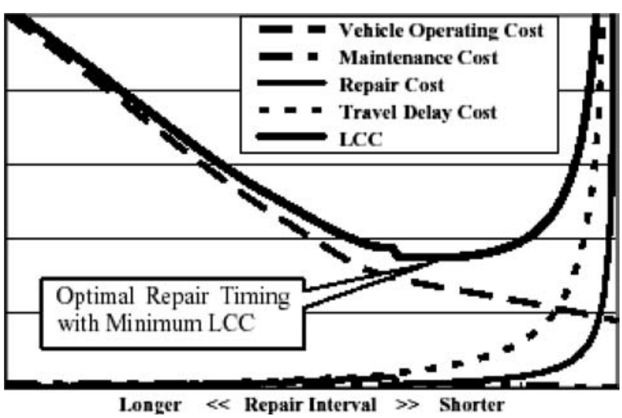
H-BMS (Hanshin expressway Bridge Management System) was developed with the help of the active database of inspection and repair. Its main objective is providing a systematical tool of building rational and efficient maintenance schemes of bridge structures. H-BMS estimates maintenance, repair costs optimized on long-term basis with the concept of life cycle cost (LCC) minimization. With the same concept, repair work prioritization by each structural element is suggested for constructing a yearly repair planning. The both outputs accompany with explicit information on calculation process, which satisfies the raising demand of accountability. It is now recognized as a main supporting tool for rational decision-making process of structural management.

LCC defined in H-BMS is the sum of direct costs and the user costs for the next 100 years, which is converted in the present value by social discount rate. Direct costs include actual regular maintenance and repair expenses estimated by the past payment. User costs are externalities like congestion cost expressed in monetary value and the increase of vehicle operation cost.

In case of pavement, the health condition is measured by Maintenance Control Index (MCI), which is a common non-dimensional indicator in Japan considering the measurement of crack and rutting. Its deterioration hysteresis is estimated by linear regression of MCI from the past inspection data. The hysteresis is prepared by route and by deck type.

LCC by span and lane is calculated with different repair interval as shown in the following figure. The optimal timing for repairing is where LCC is in minimum value.

When there is a yearly budget limit and not all sections can be repaired on their own optimal timing, the priority is judged by the extent of relative cost increase if the repair is deferred. Higher priority is given to the section where additional cost could be saved if repair were made on the ideal time.



The optimality of budget is determined by analyzing the transition of direct cost or service level with different budget limit. In case of pavement, it can be clarified that the decrease of budget by a few percent cannot sustain the acceptable condition for a long term. The suggestion of repair priority by lane is given not only in a list format but also in a mapping display which shows a visual image of damage distribution.

H-BMS is a part of comprehensive custom-made asset management system to match with the uniqueness of Hanshin expressway. The availability of complete database beforehand makes its development easy and fast. One of its unique features is that it owns two distinctive functions; budget estimation and repair plan making. They are different in time-span concerned and target users. Also, optimal total budget is sometimes contradictory with actual repair plan based on repair ordering and considering other offsystem aspects of optimization such as traffic control restraints and work scheduling.

After the test run and tuning of the first version of H-BMS covering pavement and painting works, the system are started to be used in the construction of 5-year budget framework. Also, the local maintenance offices start using the system for the making of repair work programs based on the system's suggestions of work priority and the allotted budget. From system users, many feedbacks are made. The points they are concerned most is the system's accuracy. If the system results can express the tendency empirically known, the system is regarded trustworthy instantly. Also, simulation conditions should be universal and sustainable well controlled by a designated manager. Official recognition that the system is a core part of decision-making process guarantees its credibility.

Findings obtained in the course of development and practice of H-BMS are very significant and become clues to develop a practical asset management system in other organizations.

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Life cycle costing

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Lifecycle design module for project level bridge management

E. Vesikari
VTT Building and Transport, Espoo, Finland

1 INTRODUCTION

The design of structures is more and more directed towards the entire lifetime design with multiple concurrent objectives. Today, a designer is required to justify his decisions with service life calculations, lifecycle cost analyses and the prospects of future maintenance needs. In many cases, the environmental effects must also be considered when making a decision between optional materials and structures. To cope with these new requirements, a designer must be equipped with new calculation methods and tools, which enable effective design work.

Based mainly on the financing of Finnish Road Administration, extensive research was carried out on the development of lifecycle design module for concrete bridges. The main outlines of the computer program were developed in 2000–2003 under the auspices of the EU project LIFECON (GIRD-CT-2000-00378) (Söderqvist, M-K. and Vesikari, E. 2003). However, the implementation of the design module was performed in national projects during and after LIFECON. The design module "Bridgelife" is meant for bridge owners, maintainers, bridge engineers and consultants.

The lifecycle design module was planned to form a link between the inspection of bridges and the execution of maintenance, repair and rehabilitation (MR&R) projects, which can be preliminarily designed using the module. With the help of the design module, the condition of bridge components can be predicted, the required MR&R actions can be optimally specified, the MR&R projects can be preliminarily planned and the MR&R costs can be evaluated for each project and for the whole design period. The costs evaluation includes actual MR&R costs, user costs and environmental impact.

The basic idea of the lifecycle design module "Bridgelife" is to combine a full condition analysis with a lifecycle cost analysis and a lifecycle ecology analysis. Starting from the initial data of the condition of a bridge, a statistical condition analysis is performed covering the whole design period. The optimal maintenance and repair actions are specified with the help of a decision tree and the timing of actions is automatically triggered whenever the predefined maximum allowable degradation is exceeded. The program allows also manual changes to the automatically prepared plans.

2 IMPLEMENTATION OF THE LIFECYCLE DESIGN MODULE

"Stairs of development" schematically shows the process of development for both the service life design and lifecycle planning as seen in Figure 1. The process proceeds by adding, step-by-step, new information, methods and techniques to the body of the system. At the lowest steps, the degradation and service life models for concrete structures are developed. The degradation models are a prerequisite for both service life and lifecycle design as together, with action effect models, they give the foundation for the whole life condition analysis of a structure. The Markov Chain method was chosen for the mathematical framework of the condition analysis as it fulfils all of the requirements set for a probabilistic condition analysis (Vesikari, E. 2003). The Markov Chain-based condition analysis, with the automatic triggering of actions, is attached to the framework of a lifecycle cost analysis and a lifecycle ecology analysis to produce a combined LCP, LCC and LCA

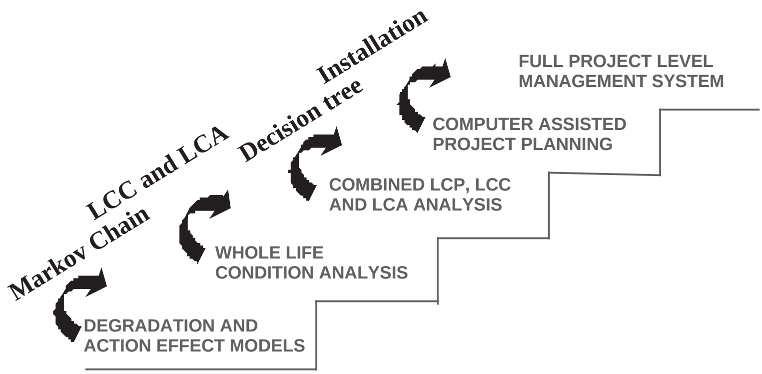

Figure 1. "Stairs of development" for a Bridge Management System.

analysis. This system, attached with decision trees for optimal MR&R action profiles, forms the foundation for computer-assisted project planning. This is what the lifecycle design module can do by itself. At the final step, the lifecycle design module is attached to the outer framework of the Bridge Management System thus providing the basis for the systematic maintenance of bridge structures as well as the annual project and resources planning and scenarios of MR&R costs.

3 CONCLUSION

A Microsoft Excel-based lifecycle design module was created for the Finnish Bridge Management System. The principles of the module follow the outlines depicted in the EU project LIFECON. The characteristics of the module are: predictive, integrated, optimising, lifecycle-based and probabilistic. The module combines traditional LCC and LCA analyses with a Markov Chain-based performance (condition) analysis. The design module is meant for bridge owners, maintainers, bridge engineers and consultants. At the moment, the module is under testing.

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Risk based approach of Life Cycle Management Systems

A. Chaperon
Advitam, Velizy, France

1 INTRODUCTION

The recent worldwide development of Bridge Management Systems and the extension of Life Cycle Cost considerations as performance studies make Civil Engineering enter the wide field of the domains where process optimization is the basis of productivity. Indeed, these domains are non-exhaustively military, industrial, aero naval and nuclear fields. No one can ignore the risk management as a foundation in those activities. The FMECA method appeared as a standard in military field in 1953, then adaptation of the FMECA to machines and processes came in the industry. Finally, aero naval and nuclear fields are well known for their failure rate normalized limitation to 10^{-7}. Therefore, it is normal to consider that the optimization science of the civil processes and costs cannot extend itself without going though risk assessments, and that techniques used for surveillance and maintenance design would avoid it.

However, where domains used to risk management are feed with large amount of feedback data, exceptional civil engineering structures are unique and the methods need to be adapted.

The fact is that any system that intends to reduce profitably maintenance costs shall be filled with outputs from practical and realistic analyses.

We propose in our article to expose the results we reached after several years of research and projects in structural risk analysis applied to computerized management system, through two examples of true projects: The Messina Project Life Cycle Cost analysis and the Rion-Antirion Bridge risk-based surveillance and maintenance.

2 FIELDS OF APPLICATION

2.1 *LCC management system*

We include as LCC management systems any computerized system that intends to provide useful information on the asset and that can help to reduce LCC:

Computerized Management Systems for Asset (Inventory), Inspection and Maintenance Management.

Other deterioration modal based systems, estimating the remaining life time as a time and an evaluated error factors.

"Smart" Monitoring Systems meant to understand the structure proper rules.

2.2 *Life Cycle Cost*

Life Cycle Costs is mainly including actualized design and construction, operation and maintenance, obsolescence and disposal costs.

Among these costs, maintenance costs can be seen as the cost of a much advanced process, including civil engineering and financial considerations. Also it can be seen as the one on which we have the more room for action, compared to frozen design and construction costs, to uncertain obsolescence and disposal costs and to regular operation costs (excluding risk provisions). Furthermore, taking long time to define maintenance costs allow generally increasing the safety on the structures.

The first example we will expose is the Messina Bridge, that will link in fine Italy and Sicilia, and for which we compared two scenarios, one including a risk based management strategy, and the other one with regular maintenance.

The second example will be the Rion-Antirion Bridge, with a practical application of the risk-based management strategy.

This abstracts provides a global presentation of both great structures. The full paper presents the works that have been achieved on them.

3 MESSINA BRIDGE

3.1 *Bridge overview*

"The Strait of Messina (divides the island of Sicily from Calabria in southern Italy) is 2 miles (3 km) wide. While the overall length is not a big problem the economics, water depth, wind, and earthquakes all have to be accounted for.

To avoid the problem of the deep water, the solution was to design the longest suspension bridge ever. It will have a 3300 m (2 mi) main span and 180 m (590 ft) side spans (overall length 3.7 km(2.5 mi)). The main piers will be founded in 120 m (400 ft) of water. There will be a new patented lighter deck design which deals with aerodynamic and seismic problems. The wind will be no problem as the aerodynamic features of the bridge will allow it to withstand 216 km/hr (134 mi/hr). Earthquakes will have to be huge as the bridge will be able to face without damage a seismic action corresponding to 7.1 magnitude in Richter scale (severer than the earthquake that destroyed in Messina on 1908). The only obstacle left is the funding. The bridge is expected to cost five billion dollars and take eleven years to build.

The bridge will be 60 m (196 feet wide) and will have 12 lanes for traffic and two lanes in the middle for trains. This will allow 140,000 vehicles and 200 trains per day. This will cut down transit times of up to 12 hours down to minutes."

http://bridgepros.com/projects/Strait_of_Messina_Bridge/

4 RION-ANTIRION, GREECE

4.1 *General presentation*

Located in the western end of the Gulf of Corinth in Greece, the Rion-Antirion Bridge links the Peloponnese to the Greek mainland. This cable-stayed bridge goes from Rion, next to Patras, capital of Achaia, to Antirion in Etoloakarnania prefecture. The Bridge unites two major roads: the intersection of the Patras-Athens-Thessaloniki motorway (which is part of the European motorway) and the western axis of the Kalamata-Patras-Igoumenitsa road. It allows for improved access between Greece and Italy through the ports of Patras and Igoumenitsa, in northwestern Greece.

The Patras area is characterized by severe wind conditions and a high seismic activity. Tectonic movements result in the South Rion coast drifting away from the North Antirion coast by several millimetres a year.

Extract from the Rion-Antirion Inspection and Maintenance Manual

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Maintenance management from an economical perspective

J. Bakker, Joris Volwerk & Jules Verlaan
Ministry of Transport, Public Works and Water Management (Rijkswaterstaat), Utrecht, The Netherlands

ABSTRACT: Acknowledging the fact that there many other perspectives, this paper discusses 'maintenance management' from an economical point of view. It will show that, using general principals of economics, an economical perspective leads to maintenance decisions differently from a technical, governance or 'user' perspective. It will also become apparent that optimisation in maintenance investments on the basis of economics often conflicts significantly with budgetary terms, frequently forcing a less economical 'short-term' maintenance strategy to be practice.

To realize the full potential of economically optimized maintenance management, it should be adapted as an investment strategy on corporate level. Setting up such a strategy requires insight in 'lifecycle costs' and the effect of 'lifespan uncertainty' for infrastructural components (balancing out risk and revenue).

To keep the current infrastructure functioning optimally in the near future (given current functional demands), the larger part of costs will be generated by the replacement economically obsolete structures (due to changing circumstances and functional demands these structures no longer meet functional requirements). From an economical perspective it is undesirable to utilize maintenance funds for this end, particularly if this leads to additional replacement costs.

Already allocated budgets can obstruct or even contradict a sound investment strategy, yet can be completely rational from a governance perspective. It is therefore critical to acknowledge these perspectives, begin to understand their relations and to incorporate them structurally in maintenance management deliberations.

1 INTRODUCTION

Historically, maintenance strategies for civil structures have been developed and put into practice by technical engineers. More often technical conditions are the main argument or case building input to substantiate maintenance decisions. In recent year however 'Asset management' (AM) has found a foothold in civil engineering. Several commonwealth states (Australia, New-Zealand and United Kingdom) have gained considerable experience with AM (some 10 years) [Ref. 1]. The essence of AM being to create a complete overview of the required functional demands for all stakeholders (e.g. functional, financial, judicial or technical), determining and monitoring all elements that impact performance in these fields on lifecycle basis and using this information to facilitate management decisions in realising or maintaining the required infrastructural functions.

Given this development in maintenance management, the economic perspective is becoming significantly more important in relation to the technical perspective which still provides the fundamental input for maintaining civil assets. The effectiveness of (existing) assets can be substantiated by researching and economically analyzing the expansion or refurbishment of existing civil structures and infrastructure.

This article addresses several issues regarding bridge maintenance management that will show that maintenance management should stem from an economically determined investment strategy related to a life cycle cost approach. In this a distinction will be made between the 'technical lifespan' (the span of time a structure is kept operational) an the 'economical lifespan' (the span of time before a structure becomes functionally obsolete).

2 MAINTENANCE TO REALIZE OPTIMAL INFRASTRUCTURAL (SYSTEM) PERFORMANCE

Roads are the economic arteries of a country. Maintenance on roads and civil structures should be focused on guarantying the short term and long term availability and as such mobility. The economic significance of mobility is a great one. The economic costs for society generated by congestion as a result of maintenance can easily exceed the costs directly related to this maintenance. This is why maintenance strategies should derive from the optimisation of availability and mobility. Integrating maintenance involving road pavement, civil structures and road accessories in to a single maintenance contract and simultaneously introducing a scale enlargement by addressing complete road/route sections, opposed to traditionally maintaining individual structures, will significantly benefit mobility. It will also increase possibilities for innovative solutions to realize these goals and reduce costs for traffic management measures due to efficiency in maintenance. Practically all maintenance on civil structures is combinable with pavement works as long as preventative maintenance strategies are applied and deterioration is kept limited. A strategy tackling sections integrally will incline to address maintenance ahead of technical necessity rather then later, if this limits congestion. With this approach critical structural components (in the Netherlands primarily concrete) can be maintained on periodically based strategy linked to pavement works instead of a condition based strategy.

This strategy, a direct result deriving from an economical perspective, strongly deviates from the international movement towards ‘reliability centred maintenance’. This movement, initiated by the technical engineers, accentuates the condition of critical structural components as the initiator for maintenance.

3 THE ROAD TO PROFESSIONAL ASSET MANAGEMENT FOR INFRASTRUCTURE

The cost analysis described in this paper, illustrate that professional asset management for infrastructure involves much more than technical perspectives. Traditional management models for maintenance no longer suffice and don’t meet asset management requirements. The Ministry of Transport, Public Works and Water Management (Rijkswaterstaat) is taking serious steps to prepare it self for future demands. The remolding into a more self sufficient agency has also replaced a budget driven structure by accrual accounting, in which organizational performance is measured by revenue indicators in a technical and social sense. The agency has liberties to re-allocate budget in time as well as purpose, in order to optimize spending and the effect thereof.

4 CONCLUSION

Bridge maintenance is often solely addressed from a technical perspective. Addressing maintenance challenges from an economic perspective will initiate fundamental en socially desirable improvements. To reach these improvements insight is needed expected life cycle costs. The economic lifespan and a realistic approach to the recognition of uncertainties should be incorporated. Parallel to this a deliberated investment strategy is paramount. On this basis a suitable management and budget structure should be implemented.

This paper shows that (given Dutch circumstances) maintenance on concrete components is by no means a leading variable in maintenance management. Postponement or omitting concrete maintenance has great financial consequences in the long term relative to the actual costs of preventative measures. Considering this, a periodical maintenance strategy (opposed to a condition based strategy) linked to the more or less fixed pavement maintenance cycle may be preferable.

The main cost drivers for maintaining a adequate and well functioning infrastructure are generated by replacement of existing civil structures on the basis of economical lifespan, due to changing functional demands. Maintenance is often subservient in a budgetary sense. Postponement or omitting maintenance on civil structures is socially undesirable when availability of infrastructure is taken in account.

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New trends in bridge management systems: Life cycle assessment analysis

Helena Gervásio
Civil Engineer, GIPAMB, Ltd., Lisboa, Portugal

Luís Simões da Silva
Civil Engineering Department, University of Coimbra, Coimbra, Portugal

ABSTRACT: Traditionally a bridge management system uses an optimization model to determine the least-cost maintenance, repair and rehabilitation strategies for a certain bridge or bridge elements using a life cycle cost analysis or an equivalent methodology. During this process, the environmental impacts of these strategies are neglected and decisions are taken regardless the negative impacts of such decisions in the environment.

In the context of a Sustainable Development framework, the environmental aspect is a fundamental issue in any decision–making process, and a final decision is usually taken considering a balance between the economic and the environmental analysis.

Bridges are massive structures and have generally a long target life, therefore environmental efficiency depends on the selection of environmentally-friendly raw materials, high durability, easy maintainability, recycling of construction wastes and finally recycling of the components and materials after demolition.

To measure the environmental performance of a system an Environmental Life-cycle Assessment (LCA) analysis can be performed. This approach is based on the belief that all stages in the life of a system generate environmental impacts and must therefore be analysed.

In this paper the requirements for an LCA approach to be included in a bridge management system are presented. A Life Cycle Analysis is not a straightforward process and there are currently some problems and limitations in its application, therefore the aim of this paper is to assess the implications of these issues in bridge management, namely, (i) to present the constraints in current databases of construction materials and (ii) to set out the boundaries in bridge life cycle for a reliable data collection in the inventory analysis.

Keywords: Structural bridge engineering, bridge management system, Life-cycle assessment, Sustainability.

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Probabilistic approach for predicting life cycle costs and performance of bridges

A.P. Silva & A.A. Fernandes
Faculty of Engineering, University of Porto, Portugal

EXTENDED ABSTRACT: The construction industry is a major consumer of non-renewal resources. Life Cycle Costing (LCC) enables the whole life cost and performance of bridges to be optimised and so can make a major contribution to sustainable development. The increasing use of procurements arrangements such as the Private Finance Initiative (PFI) and the Public Private Partnership (PPP) has focused stakeholders on the whole life cost and not simply on the lowest capital cost. This has led to a demand of LCC models that can provide uncertainty quantification in addition to accurate forecasts and estimates. The quantification of the overall risk exposure, throughout risk analysis, will help the design team to concentrate on the most critical elements, avoiding in this way waste, though over-design, or costly premature deterioration.

Life cycle costing may be defined as the estimation of the costs of acquiring and operating an asset over its intended service life. Within the construction and property sectors, life cycle costing seeks to consider *all* the costs and revenues (incomes) associated with building, operating and disposing of buildings and built infrastructure.

In particular, life cycle costing may be used to facilitate choices where there are alternative means of achieving the client's objectives. These alternatives usually differ not only in their initial costs but also in their future costs (i.e. during the operation and disposal phases). Life cycle costing may also be used to estimate the total costs of a single asset, so that appropriate provision can be made to finance it.

LCC may be used at all the important decision stages in the asset life cycle including procurement, construction, operation and finally the disposal of the asset. For example, LCC may be used to

- inform the initial investment appraisal or decision whether or not to build or acquire by purchase or lease
- assess of feasibility of alternative construction solutions, including replacement and/or maintenance over the life of the asset
- support the outline design
- support the detailed design (including choice of components and services)
- support tender appraisal
- assess the impact of variations during the course of construction
- support hand-over and final account
- assess of the effectiveness of the construction (post-occupancy evaluation).

At each decision stage of a project additional and more reliable information becomes available and the calculation of the life cycle costs may be refined to provide increased accuracy of estimates of the life cycle costs of the project. The aim must be to achieve recognition of the optimum life cycle cost of the asset, balancing the optimum value, the minimum overall life cycle cost and the maximum functionality.

The process of life cycle costing is, in itself, very simple and many programmes exist to enable life cycle costs to be calculated. The skill in estimating life cycle costs is in modelling performance in order to be able to predict when interventions are required and hence to estimate when the associated costs will be incurred. It is recognised that predictions are exactly that and are likely

to differ in quantitative terms from reality, particularly when predicting future events. The skill in LCC is therefore to understand the extent to which real costs may differ from the estimated cost and to use this understanding to manage the financial risk. To enable this to be achieved, a probabilistic approach has been adopted which takes account of uncertainties and quantifies the likelihood of deviations from the estimated LCC. This approach helps both to overcome concerns about uncertainties in forecasting, which have proven to be a major obstacle in to the application of LCC in construction, and to provide a more robust and transparent method for management of the financial risks.

The present paper reports research work in progress aiming to develop a life cycle cost and performance (LCCP) model to optimize the life cycle cost and performance of bridges and provide a more robust and transparent method for management of the financial risks.

The approach adopted consists of three distinct elements, a probabilistic life cycle cost model, a probabilistic deterioration model and the decision support application. The integrated model enables the analyst to calculate the LCC results probabilistically, for different scenarios of analysis, using Monte Carlo simulation. As follows, the model is innovative in that it provides to the decision maker the ability to understand and manage financial risk relating to the life cycle of an asset with a greater degree of transparency and rigour than is currently available.

A software tool has been developed that allows a probabilistic sensitivity analysis to be performed both at the planning stage to support go/no-go decisions on major investment or to understand project trade-offs at concept or detailed design phases.

Bridge condition assessment using combined non-destructive testing methods

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Current use of NDT in bridge condition assessments

Birit Buhr Jensen, Thomas Frølund & Torben Pedersen
COWI A/S, Kongens Lyngby, Denmark

ABSTRACT: Currently, in principle operation and maintenance of bridges in Denmark follow two steps: a general inspection and a special investigation. The general inspection comprises a visual inspection, where visual signs of deterioration are registered. Where deterioration is detected a special investigation is called for. The typical damage which calls for special inspections are often visual signs of corrosion on bridge piers and decks, water leakage as a result of a defective bitumen membrane, etc.

One of the challenges in connection with the special inspections lies in determining the cause of the deterioration, the extent of deterioration together with the deterioration rate. Where chloride-induced corrosion is a visual investigation, monitoring can be subject to a high degree of uncertainty. In this connection potential mapping together with calibration of break-ups can be a strong non-destructive testing (NDT) tool in finding the actual extent of corrosion. When combined with corrosion rate measurements, the deterioration rate can be estimated and used where repairs are postponed.

Where deck structures and other structures are concerned, it is of vital importance to evaluate the integrity of the structure together with the extent of possible damage caused by frost actions and alkali silica reactions.

The present paper describes the principal, general and special inspections in more detail together with examples on the use of NDT testing. Examples covering potential mapping will be presented together with corrosion rate measurements to evaluate the extent of corrosion and the corrosion rate. Also, the use of impulse response testing as a strong tool in evaluating the extent of flaws will be presented. The potential mapping and impulse response testing are strong NDT tools which help the engineer to get a good overview of corrosion and deteriorations besides what is visually registered.

Also the use of other testing methods are briefly presented, and two cases given where the potential mapping/corrosion rate measurements and impulse response testing has been applied with benefit.

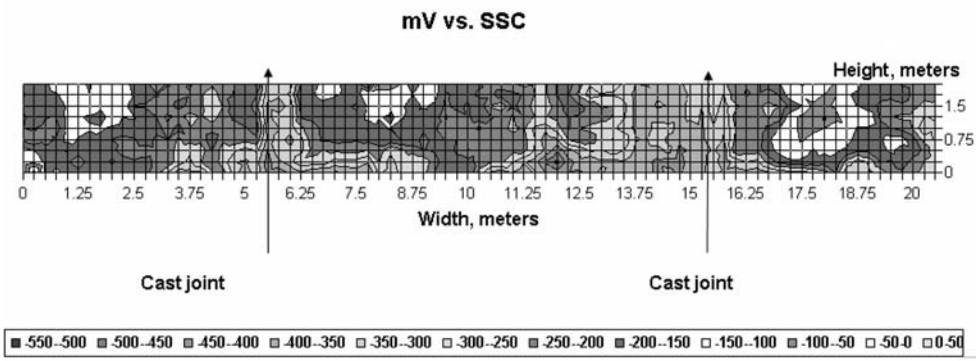

Figure 1. Half-cell potential map. Areas with possible corrosion are shown in yellow and red colours.

The “Danish way” of conducting bridge condition assessments includes all of the above-mentioned tools. They are used on a regular basis as supportive and elucidating tools for the overall evaluation of the bridge condition, the deterioration extent and rate.

1.1 *Half cell potential mapping*

Core samples, break-ups, chloride profiles and corrosion rate measurements were performed in corroding and passive areas in accordance with the half-cell potential map.

1.2 *Impulse Response testing*

In connection with special investigations of two twin bridges, the Borrevejle Vig bridges, cracks have been detected at the soffit of the deck slab. In relation to an ongoing load carrying capacity evaluation of these bridges, it has been important to know whether the concrete of the deck slab can be considered intact or not. To verify this “Impulse Response”testing was carried out.

The results indicate that considerate problems concerning the structural integrity/condition can be expected.

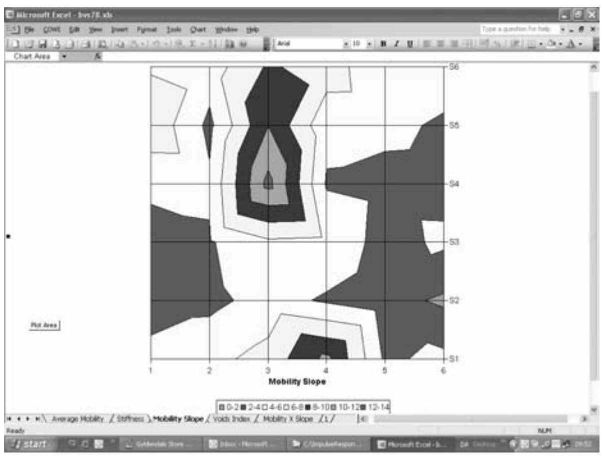

Figure 2. Plot showing mobility slope for Borrevejle Vig bridge deck. The results cover an area of approx. 6 × 6 m.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Trends in bridge condition assessment using non-destructive testing methods

E. Niederleithinger, R. Helmerich & H. Wiggenhauser
Federal Institute for Materials Research and Testing, Berlin, Germany

ABSTRACT: In a few countries, bridge condition assessment procedures include non-destructive testing techniques, but regular application and integration is still rare. Main reasons are a lack of knowledge at the bridge authorities, cost and time consumption and difficulties in data interpretation. The EC funded integrated project "Sustainable Bridges" looks at optimization of NDT for railway bridges and new ways for combining conventional and modern methods. Among other topics we are developing automated measurement systems and a method database.

1 INTRODUCTION

To fulfill future traffic demands the European railway network companies have to increase the capacity and durability of their bridge stock in terms of speed, axle load and life span. Another requirement are low maintenance costs. These goals can be reached by improved condition assessment, reassessment of load capacity, repair end strengthening. As the currently used tools are insufficient in may cases, the European Commission funds the integrated project "Sustainable Bridges"(Olofsson & Elfgren, 2004). The consortium consists of 32 partners of 10 countries. One of the workpackages is aimed to improve condition assessment procedure with focus on NDT methods. Improvement of techniques, automatisation and data processing are important tasks. But main aim is to make the research results available to and useful for railway companies and engineers in the condition assessment process.

2 TRENDS IN THE USE OF NDT

Method Improvement: The state of the art in NDT in civil engineering has reached a high level in the last decade. So the target was not to invent new methods but to fit existing ones to the need of the railways. Other papers in this volume document some of the developments done in the project.

Automatisation: To ease operation and to decrease the time needed per measurement point we have developed several automatic scanning systems. They can be equipped with radar, ultrasonic echo and impact echo sensors. A camera or an airbrush marking system can also be mounted. The system has been tested at a railway bridge under traffic (figure 1) and has shown its excellent applicability and reliability. The results showed only minor influence of traffic vibrations to measured data and no interference to signaling and communication.

Interpretation: Most data measured by NDT (e.g. travel times of acoustic waves) have no direct use for the bridge engineers. They have to be transformed and interpreted to give a meaning. The basic steps are: a) Reconstruction: Calculation or estimation of boundary, object or defect positions from measured data, b) Combination: Use of more than one technique to answer multifacetted, complex problems, c) Data fusion: to merge datasets of various techniques into one, and d) Visualisation: Make results visible in 2D or 3D pictures or CAD plans.

Figure 1. Scanning system at a German railway bridge.

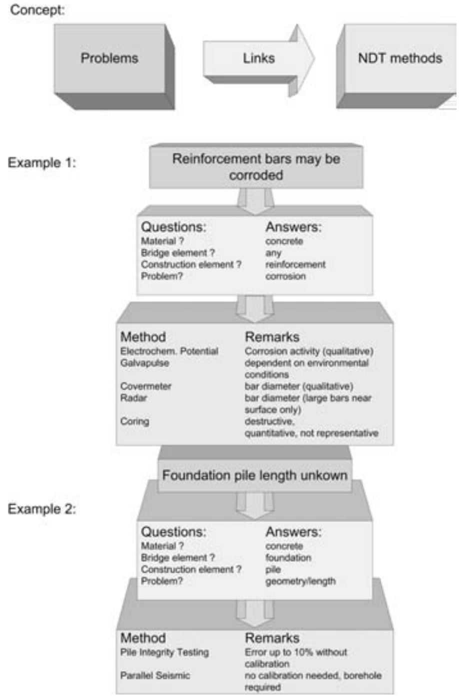

Figure 2. Use of NDT Toolbox. The engineer is guided by problems and a few questions.

3 NDT TOOLBOX – INTEGRATION INTO CONDITION ASSESSMENT SYSTEMS

Since many bridge inspectors may be not well informed about the capabilities and limitation of these techniques some kind of information database had to be developed.

We decided to collect one-page information of at least the most important methods including fields of application, requirements, limitations and examples. At the moment the toolbox contains about 40 sheets and can easily be extended.

To enable the bridge inspectors to select methods suitable for his problems and to assess the results he gets, the NDT toolbox is currently linked to a new damage catalogue and classification system under development in the project (Maksymowicz et al, 2006). For each problem the inspector may encounter he will be guided by answering a few questions to a list of the most applicable methods (figure 2).

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Verifying design plans and detecting deficiencies in concrete bridge using GPR

L. Topczewski, F.M. Fernandes, P.J.S. Cruz & P.B. Lourenço
University of Minho, Department of Civil Engineering, Guimarães, Portugal

1 DESCRIPTION OF THE SPECIMEN

The concrete specimen that has been specially designed and built for the testing campaign in April 2004 aims to simulate a small part of a typical concrete bridge deck. It is a square with dimensions of 250×250 cm and a thickness of 32 cm. The elements that were placed inside were chosen among typical structural elements used in the construction of reinforced and pre-stressed concrete bridge decks as well as a number of typical defects and anomalies that may occur deriving from errors during the construction.

Inside the square slab were placed three tendon ducts, in PVC, with 110 and 35 mm of diameter. The larger tendon duct is generally used for the placement of post-tensioning cables, while the two smaller tendon ducts are simulating simple mono-cables, one in straight line and the other with curved path. The objective is to have a clear detection by radar and to verify if all tubes are correctly detected, with special attention to the curved tube. The measurements will be performed without cables inside the smaller tubes. However, in the largest one, half the tube will be fully grouted with cement grout and the other half only half grouted, in order to check the difference between fully filled and partially filled sections in tendon ducts. Several other deficiencies were simulated as the main objective of these measurements was addressed to the detection of anomalies. The event of a poorly vibrated concrete was simulated by: blocks of concrete with insufficient binder (poor concrete), different density concrete (light-weight concrete), large voids, and blocks with inclined surfaces. One void bottle, one clay brick, wood and mortar prisms and two metallic bras were also put in this specimen.

Additionaly, a precasted concrete element was placed at the bottom of the specimen, and it occupies half of the total area of the slab.

Two different measurement areas were used for acquisition of parallel and closely spaced 2D profiles in order to obtain a more realistic 3D image.

2 RESULTS SUMMARY

2.1 *Results from Area 1*

The objective in this area was to determine the position and shape of all the elements (poorly vibrated concrete and voided prisms in particular) position of the opposite surface and the precasted slab. First, 2D images were processed and analyzed. Subsequently, the 3D Image was build by performing linear distance interpolations between successive and closely spaced 2D radargrams. In this way, it was possible to obtain a more realistic view of the data. Eventually, the shapes and relative position of the objects previously detected with 2D profiles were better defined. The whole data is migrated in order to transform the diffraction hyperbolas in patterns closer to the shape of the objects they represent, and slices at different depths can be extracted to compare with the original design drawings. Generally, a good correlation between the dimensions and relative position of single objects with the original design plans was obtained.

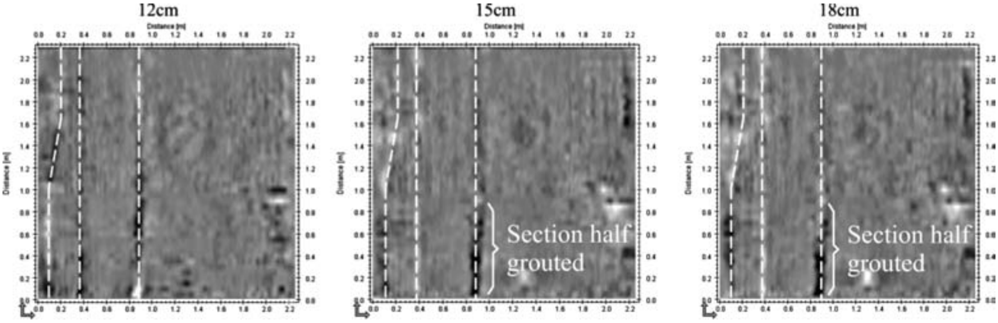

Figure 1. Radargrams showing the three tendon ducts.

2.2 *Results from Area 2*

The second area considered for the investigation corresponds to the totality of the slab's surface, with the objective of mapping the features located in the interior of the slab. In this case, particular interest lied on the detection of the three tendon ducts, especially in the acquisition of the curved shaped tendon duct and of the larger one, which has half and fully grouted sections.

The analysis of the 2D profiles showed that the three tendon ducts were detected in all cases. The analysis of the time slices gave good results. It was possible to resolve all intended targets, namely the three tendon ducts, and the position and shape of some of the other targets (not all targets were indeed resolved). The mapping of the three tendon ducts is shown in Figure 1, where it can be observed that all tendon ducts are correctly detected and the curvature of the curved tendon duct is well defined. The largest tendon duct has been correctly detected, although the intensity of the signal's amplitude was not uniform along its entire length, as expected due to the irregularity of the filling. As it can be seen in the 2nd and 3rd radargrams of Figure 1, the radiowave's energy seems to be higher in the bottom part of the section where the tendon duct's section is half grouted or half air filled. The air in that area could explain the higher energy due to the high contrast between dielectric constant of concrete ($\varepsilon_{r,concrete} = 6$) and air ($\varepsilon_{r,air} = 1$), which does not exist between concrete and grout. Thus, it seems possible to detect differently filled tendon ducts sections by analyzing the amplitudes of the reflected signals.

3 CONCLUSIONS

The analysis of design conformity and mapping of deficiencies in concrete bridges is a matter of extreme importance for future maintenance. The use of GPR with 3D reconstruction showed good potential in the detection of the main structural elements as well as with the mapping of typical deficiencies. The antenna with a central frequency of 1.6 GHz exhibited very good resolution and high accuracy. The tendon ducts were all detected and the curvature of the curved one was correctly assessed.

However, not all targets were resolved, but the analysis pointed out several reasons to explain these results. Effectively, some of the targets were not detected due to its unfavorable orientation relatively to the direction of the profiling, which was the case of all objects parallel to the direction of the profiles. Because the profiles started and ended 10–15 cm after and before the specimen's border, some objects located very close to the edges of the specimen were not detected. Particularly the steel bar perpendicular to the tendon ducts. Moreover, the poor contrast between materials resulted in poor reflectivity and thus weak detection, particularly in the case of the lightweight concrete specimen. Additionally, the triangular shapes did not reflect favorably for detection by the radar signals, which resulted in the non detection of those objects.

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Crack depth determination at large concrete structures using scanning impact-echo-techniques

M. Krüger
TTI GmbH – TGU Smartmote, Stuttgart, Germany

C.U. Grosse
Department of Civil and Environmental Engineering, University of California, Berkeley, USA

H.W. Reinhardt
Institute of Construction Materials, University of Stuttgart, Germany

ABSTRACT: In reinforced concrete structures concrete cracking is an aspect to be further analyzed, because concrete cracking often results in stiffness loss and also in corrosion of the steel reinforcement. The corrosion rate strongly depends on the crack width and the crack depth so there is the need to measure and to characterize its dimensions. The impact-echo technique is a useful non-destructive test method for the detection of diverse faults in concrete structures. Unfortunately, the existing instruments and analysis tools designed for measurements were lacking of several features in the past detaining the extensive use. A new concept for impact-echo testing systems is presented in this paper. A new device was developed, which is small, robust and easy to handle. The system utilizes advanced impact generation for fast scanning techniques and reproducible impacts. The data acquisition, filtering and visualization of data are optimized for the inspection of large structures with a large number of measurement points. A new option is the estimation of crack parameters like crack depth. In this paper fundamental principles and the combination of the different measurement techniques as well as details of the measurement system are described and some test results are shown.

1 INTRODUCTION

A growing market for non-destructive testing has evolved due to the demands for quality control, defect diagnosis and sustainability of structures in civil engineering. Several methods are well-introduced concerning defect characterisation of concrete structures. Ultrasound, radar, thermography, x-rays, electro-potential-field methods and others are currently being used to detect voids, cracks, corrosion, etc. Several years ago, the Impact-Echo (IE) method, that considerably enhanced the detection of voids and honeycombing in massive structures, was introduced by Carino et al. One of the advantages of this method is its ability to detect voids in structures very fast and to measure the thickness of concrete parts with good accuracy. However, the potential of this technique is currently not being used to its full extent regarding handling as well as analyzing techniques, therefore reducing the economic value of this method.

2 STANDARD MEASUREMENT TECHNIQUE

The Impact-Echo method uses transient stress waves generated by an elastic impact on the surface of concrete or masonry structures. This impact causes stress waves that propagate through the material being tested. At internal interfaces (discontinuities in the material) and external boundaries of the structure these stress waves are reflected. Examples of such interfaces are delaminations, voids, honeycombing and cracks, as well as rising mains or large steel bars. In order to detect such interfaces, the reflected waves are recorded by a displacement or acceleration transducer which is placed near the impact point on the surface of the structure.

The depth of any internal flaws or external interfaces can be determined by analyzing the recorded signal, its characteristic frequency spectrum and the wave speed.

3 PRINCIPLES OF CRACK DETECTION USING IMPACT-ECHO

One principle of crack detection using the test proposed test setup is similar to time of flight techniques that was firstly introduced by Sansalone et al. A signal emitted by the impactor will be detected after a certain travel time t and with a certain amplitude or energy, respectively. If a surface crack with a tip depth d_{cr} is present between emitter and sensor, a time delay Δt occurs in the signal. With knowledge of the wave speed v of the compressional wave and the distance d_s between the sensor and the impactor the travel time t_{cr} will correlate with the crack tip depth.

Unfortunately, the time delay depends very strong on the material filling the gap between the crack edges since there is usually not only air in between. Additional effects are caused by the steel reinforcement able to bridge the crack flanks.

Therefore, it is appropriate to use a second parameter that is the energy of the emitted signal as recorded by the sensor ensuring that the emitter produces a highly reproducible constant signal. This is especially true for the used electronic impactor. Tests have shown that the cumulative energy (samplewise addition of the squared amplitude of the received signal) is a good discriminator between concrete surfaces with and without cracks. The peak amplitude of a time signal travelling across a crack is delayed and the overall energy is significantly lower compared to a wave travelling along an undisturbed surface, because a part of the impact energy is reflected at the crack surface. It has to be noted that this technique might also be used to detect vertical cracks that were opposite to the inspection side.

An interesting aspect is that the new developed Impact-Echo measurement technique as shown above provides different values simultaneously, which could be used for the characterization of the observed structure. The value which is normally used for the detection of flaws is the resonance frequency in the spectrum. However, with the new developed measurement system it is possible to use the Impact-Echo technique to receive additional values e.g. onset time for the time of flight technique and in addition the cumulated energy for a more precise characterization of the inspected structure. Horizontal and vertical crack and flaw detection as well as the concrete quality characterization could now be combined in just one single measurement.

4 OUTLOOK

Regarding the civil engineering industry an increasing demand for quality control of structures can be observed. Advanced Impact-Echo testing techniques that are easy to use for fast, repeatable and reproducible measurements can be used to improve the existing techniques or to replace visible inspections detecting voids or cracks. The benefits are already obvious to be the one-sided access and the easiness to conduct measurements saving time and money and bring the inspection on a more reliable and objective level.

It was shown that the IE technique has the potential to detect precisely large voids, honeycombs and inhomogeneities as well as the thickness of concrete structures. The new developed device reduces the time necessary for each measurement by a factor of ten. Some new methods are described concerning the detection of cracks. First promising results were shown using the cumulative energy of the transmitted signal as a crack discriminator. Essential is that the new system provides a more reliable impact generation.

A combination of the described methods can be used for crack detections and in future for an automatic determination of other crack parameters like crack depth and width. However, the signal amplitude and the signal energy could be used as criteria only if an internal horizontal interface or external boundary exists at the inspected structural part. As a conclusion it could be shown that the test method permits the detection of vertical cracks in general as well as crack width and crack depth. Due to the fact that the described methods for vertical crack determination require just a slightly modified Impact-Echo system, it could be combined with the standard Impact-Echo measurement methods, which are nowadays widely used to detect voids, delaminations and honeycombs.

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Development and combined application of NDT echo-methods for the investigation of post tensioned concrete bridges

D. Streicher, D. Algernon, Ch. Kohl, M. Krause, C. Maierhofer & H. Wiggenhauser
Federal Institute for Materials Research and Testing, Berlin, Germany

ABSTRACT: In the last time impulse-echo methods were increasingly applied for investigating concrete structures. These methods are developed for existing structures as well as for new structures in the frame of quality assurance. The three echo methods impulse radar, ultrasonic echo and impact echo, based on the penetration of electromagnetic and acoustic waves, are complementary related to the detectability of interfaces, voids and honeycombing and reinforcement. Each of the methods is further developed and leads – together with reconstruction calculations – to a precise image of the inner structure of the concrete element. This is achieved by collecting data along a selected area of the surface and by three-dimensional data evaluation.

In this contribution, new results of bridge assessments are presented with the respect of the combination and data fusion of the different non-destructive echo methods.

1 INTRODUCTION

In this contribution, new results of bridge assessments are presented with the respect of the combination and data fusion of the different echo methods. This mainly concerns the

- distinction and complete imaging of reinforcing bars and tendon ducts in the near surface area by combination of radar data recorded with different polarisation
- inspection of the internal condition of tendon ducts with the acoustic methods
- localisation and investigation of deeper layers of tendon ducts with the ultrasonic echo method

The three echo methods are steadily advanced in order to enhance their reliability, also when the site conditions are more complicated in the case of very dense non-prestressed reinforcement or deeper layers of tendon ducts.

2 EXAMPLE FOR ULTRASONIC APPLICATION

The interpretation of the ultrasonic echo measurements is based on 3D-reconstructed data, calculated with a SAFT-program of the Fraunhofer-Institute Non-Destructive Testing (IZFP) in Saarbrücken. By the use of this calculation the signal-to-noise-ratio is increasing considerably.

Tendons in investigated building structures could be localised with an automated ultrasonic echo equipment up to a measurement depth of 40 cm. The ultrasonic waves are also reflected on fully grouted tendon ducts, but with a lower intensity. Therefore the lateral position of the tendon ducts could be reliably identified by means of reflections from tendon ducts and the shadowing of the back wall behind them. If the reflections from the tendon ducts are more intense than the noise of the concrete texture it is possible to specify the concrete cover of the tendon ducts. Figure 1 shows that also tendons arranged behind others were reconstructed at the investigated webs at the bridges in Austria. The application of ultrasonic echo on larger areas allows the visualisation of perpendicular arranged reinforcement bars. In comparison with radar the display of the

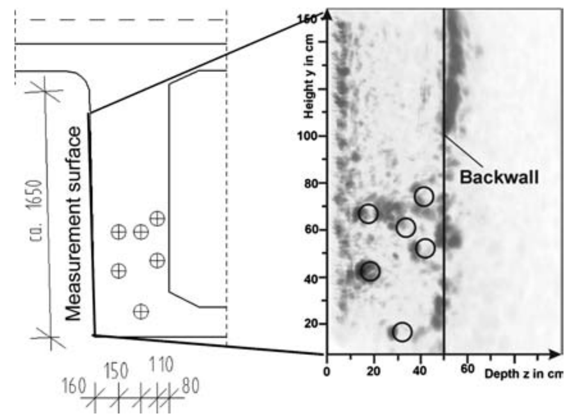

Figure 1. Arrangement of tendon ducts in the cross section (1/10-point 0.7) of a box girder web in Austria, left: according to construction plan, right: located at a SAFT-B-projection by ultrasonic echo.

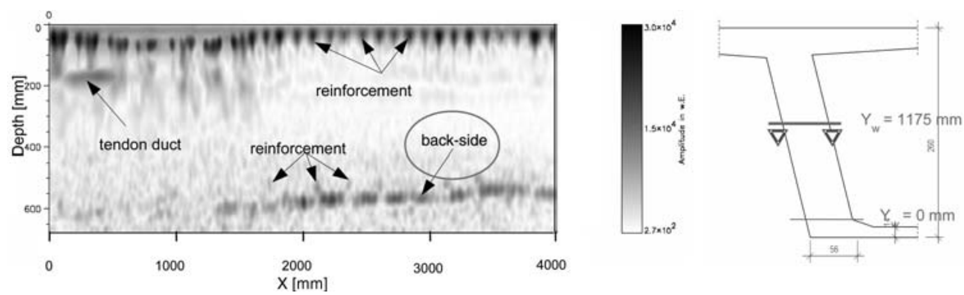

Figure 2. B-scan through the box girder web of the bridge in Germany at the level of $y_W = 1175$ mm from the fused dataset of the reconstructed data of radar and ultrasonic echo.

non-tensioned reinforcement detected with ultrasonic echo is more diffuse and incomplete. On the other hand the thickness of the structures up to 80 cm could be clearly determined.

3 EXAMPLE OF DATA-FUSION

It is possible to combine data sets measured with different NDT methods, like radar and ultrasonic echo. Both methods complement one another as it is shown in Figure 2 (slice perpendicular to the surface).

The B-scan is part of the fused data set. Radar and ultrasonic echo data sets, measured on the box girder web in Germany, were combined. The multitude of reflections of rebar near the surface and the reflection of the tendon duct on the left side of the B-scan were measured mainly with radar. The radar measurement in this depth range is more suitable and useful than ultrasonic because of the enhanced resolution, especially for the detection of metallic reflectors on the other hand. The reflection of the back wall and signals of the rebar above the back wall, both at depths of 45 cm–60 cm, were exclusively measured with ultrasonic echo. For the radar these reflectors are too deep to give a significant reflection. By the combination of the radar and ultrasonic echo data sets it is possible to compress information in one data set. This allows the complex visualisation and interpretation of one data set measured on the box girder web with ultrasonic echo and radar.

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Cruz, Frangopol & Neves (eds)

Concrete railway bridges – taxonomy of degradation mechanisms and damages identified by NDT methods

M. Maksymowicz & P.J.S. Cruz
University of Minho, Portugal

J. Bień
Wroclaw University of Technology, Poland

R. Helmerich
Federal Institute for Materials Research and Testing, Germany

1 DAMAGES AND DEGRADATION MECHANISMS

This paper presents an approach of defining and classifying degradation processes in concrete railway bridges and also their relations with damages classified in regard to their effects. This work focuses also on the references to the NDT methods, which can be used for identification of damages and degradation processes. The aim of this work is creating basis for unification of bridge condition assessment and for application of expert tools supporting evaluation process. In order to use a common procedure of structure condition assessment an unified taxonomy – accepted on international level – is needed.

The proposed taxonomy of basic degradation mechanisms identified in concrete railway bridges is presented in Figure 1.

Relationships between the degradation mechanisms and the basic types of damages, based on analysis of many practical cases, are presented in the Table 1.

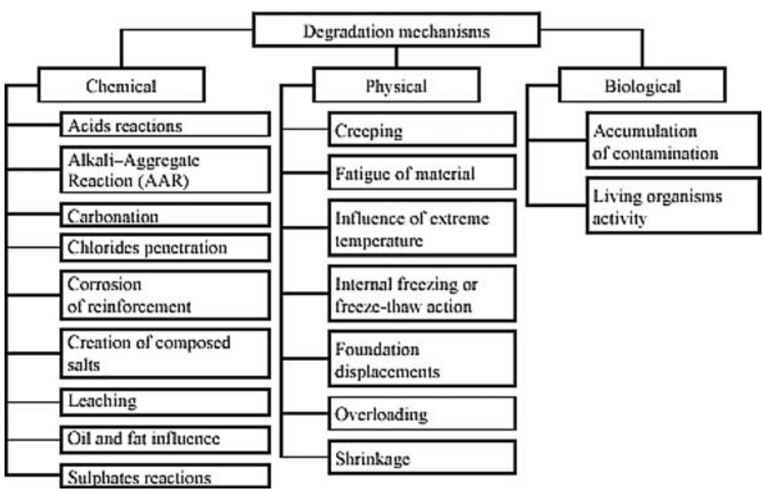

Figure 1. Degradation mechanisms of concrete railway bridges.

Table 1. Degradation mechanisms and basic types of damages.

Degradation mechanisms / Damages	Chemical									Physical									Biological	
	Acids	Alkali – Aggregate Reaction (AAR)	Carbonation	Chlorides penetration	Corrosion of reinforcement	Creation of composed salts	Leaching	Oil and fat influence	Sulphates reactions	Creeping	Fatigue of material	Influence of high temperature	Internal freezing	Foundation displacements	Overloading	Salt-frost scaling	Scour of foundation	Shrinkage	Accumulation of contamination	Living organisms activity
Destruction	✓	✓	✓	✓	✓	✓	✓	✓	✓		✓	✓	✓			✓			✓	✓
Discontinuity		✓	✓	✓	✓	✓			✓	✓	✓	✓	✓	✓	✓	✓	✓	✓		✓
Losses	✓	✓	✓	✓	✓				✓		✓	✓	✓	✓	✓	✓	✓		✓	✓
Deformations										✓		✓		✓	✓		✓	✓		
Displacements														✓	✓		✓		✓	
Damages of protection	✓	✓	✓	✓		✓	✓	✓	✓	✓	✓	✓	✓		✓	✓	✓	✓	✓	✓
Contaminations		✓			✓		✓					✓	✓				✓		✓	✓

2 CONCRETE BRIDGE DEGRADATION AND NDT METHODS

Evaluation of concrete bridge condition needs identification of structure damages and ongoing as well as potential degradation mechanisms. The most popular and efficient tools offer non-destructive testing (NDT) technology. The NDT methods enable testing without additional damaging the structure.

The authors present the application range of the selected NDT methods applied for identification of the basic damage types. Shown comparison of various NDT techniques can be helpful in selection of the testing methods in the process of condition evaluation of concrete railway bridges.

3 CONCLUSIONS AND ACKNOWLEDGEMENTS

Presented solutions can be considered as a part of European discussion concerning common methodology of advanced bridge condition assessment and forecasting. The proposed uniform taxonomy of degradation processes occurring in the concrete railway bridges as well as presented analysis of the available NDT methods can be applied in evaluation of bridge condition and in modelling of structure lifetime. The aim of this work is creation of theoretical background for expert tools supporting decisions in bridge management. The described relations between degradation mechanisms, damages and NDT methods are the basis for creation of the knowledgebased computer tools for evaluation of the current bridge condition or for forecasting the future behaviour of construction.

The proposed methodology was partly developed in the Integrated Project "*Sustainable Bridges – Assessment for Future Traffic Demands and Longer Lives*", contract No TIP3-CT-2003-001653, a part of 6. Framework Programme of European Union, for which the authors thank very much.

Durability performance of bridges in severe environments

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Durability of bridges in severe environments: The high quality cover plus monitoring-approach

M. Raupach
ibac – Institute of Building Materials Research, Technical University of Aachen, Germany

Gan Weizhong
Institute for Concrete Construction, Ningbo University of Technology, China

ABSTRACT: During recent years several large concrete bridges and other structures have been built in marine and coastal environment with designed service life times of 100 years or even more. One problem of such structures is the achievement of a durable corrosion protection of the reinforcement in the areas exposed to chlorides from seawater or contaminated soils, especially in the splash water and spray water zones. Nowadays the following concept is often used to ensure durability: The quality of the concrete cover (thickness and diffusion resistance against chlorides) is designed and executed to provide sufficient protection and monitoring systems are used to verify the design assumptions.

Different sensor systems have been developed to monitor the corrosion risk for the reinforcement, designed to be installed directly into the concrete during construction or afterwards into drilled holes of existing structures. Most of these sensors consist of different single sensors in defined distances from the concrete cover, e.g. of 6 steps, each of about 1 cm width, depending on the thickness of the concrete cover.

These sensors indicate the depth of the critical chloride content and the depth of the carbonation front initiating corrosion: The time-to-corrosion can be determined, enabling the owners of buildings to initiate preventive protection measures before cracks and spalling occur, i.e. before the costs for repair measures increase over-proportionately. This paper shows the principles and practical examples of the measurement of the corrosion behaviour of the reinforcement under on-site-conditions for selected bridges.

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Durability design of concrete structures in marine environment

O.E. Gjørv
Norwegian University of Science and Technology – NTNU, Trondheim, Norway

ABSTRACT: In the present paper, current experience with probability-based durability design of concrete structures in Norwegian harbors is briefly outlined and discussed. In principle, the design procedures are based on the following elements:

– Probability-based durability analyses
– Additional strategies and protective measures
– Quality control and documentation of obtained durability
– Manual for condition surveying and protective maintenance.

In order to carry out the durability analyses, onset of steel corrosion is defined as the serviceability limit state. In a very simplified way, the calculation of time for the chlorides to reach embedded steel is based on Fick's Second Law of Diffusion in combination with a time-dependent diffusion coefficient. Since all the input parameters for such a calculation show a high scatter and variability, a combined calculation with a Monte Carlo Simulation has proved to provide a simple and proper basis for calculating the probability or risk of steel corrosion after a certain period of time. A special software for this combined calculation has been developed.

Based on a risk level of 10% for steel corrosion to occur during a certain period of service, requirements to both chloride diffusivity and concrete cover are established. Although the control of chloride diffusivity is based on an accelerated test method which only takes a couple of days for testing, this is not good enough for a regular quality control of chloride diffusivity during the construction process. For the given concrete, therefore, the relationship between chloride diffusivity and electrical resistivity is established. Then, the quality control of the chloride diffusivity based on measurements of the electrical resistivity, is carried out during the construction process. These measurements are carried out on the same concrete specimens as that used for the testing of compressive strength.

In addition to the regular quality control of chloride diffusivity in the laboratory, it is also important to provide documentation of obtained chloride diffusivity on the construction site. In principle, such a documentation should be based on a number of concrete cores taken from the real structure under construction. In order to minimize such a coring from a densely reinforced structure, however, a small but representative reference element without any reinforcing steel is separately produced on the construction site. This element is produced at an early stage of the concrete construction, where both concrete casting, removal of formwork and curing are carried out as representative as possible for the full scale concreting. From this reference element, a number of concrete cores are removed at different periods of time during the construction process. In addition, a few concrete cores from the real concrete structure are also removed. It is the testing of all these cores that provides the basis for control of obtained chloride diffusivity on the construction site. When the construction process is over, a new risk analysis provides the basis for documentation of obtained durability.

For the durability analysis of a given concrete structure, it is always desirable to reach a combination of concrete quality and concrete cover which gives the highest safety against steel corrosion possible. However, very often it is not possible to obtain a sufficiently high safety level under given conditions. Also, for many concrete structures, experience has shown that a high rate of chloride penetration may take place already during the construction process before the concrete

has gained sufficient maturity and density. Therefore, for all concrete structures in chloride containing environments where safety, durability and service life are of special importance, additional strategies and protective measures should be considered both from a technical and economical point of view. As part of such a multi-layer protection strategy, cathodic prevention or preparation for such prevention or partial replacement of the conventional steel with stainless steel or a composite reinforcement system should be considered. Protection of the concrete surface by proper coatings or surface treatments may also be a possible measure.

Based on the current experience with durability design as outlined in the present paper, the following conclusions appear to be warranted:

1. When the durability requirement is based on an upper risk level of 10% for steel corrosion to occur during a certain service period, this should only be considered as an overall quality requirement to the durability of a given structure. However, this is a type of quality requirement which is based on an engineering judgment of all those factors which are considered relevant for the durability, including the natural scatter and variability of all factors involved.
2. For a given environment, the established procedures for durability design have shown to provide an appropriate basis for comparing the risk level of steel corrosion for various combinations of concrete quality and concrete cover. For the given requirements to chloride diffusivity and concrete cover, a basis for an extended quality control during the construction process is obtained. By the end of the construction process, a documentation of obtained durability in the form of a risk level for corrosion is also obtained. If the obtained durability for a given structure is distinctly inferior to that specified, protection of the concrete surface by a proper coating or surface treatment should be considered.
3. Even if the strictest requirements to concrete quality are both specified and obtained, experience has shown that a certain rate of chloride penetration may still take place. Therefore, at the end of the construction process when the service period starts, it is very important to have a service manual for regular condition surveying and monitoring of the real chloride penetration. It is such a monitoring in combination with protective measures for control of further chloride penetration which provide the ultimate basis for obtaining a more controlled service life.

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Chloride penetration into silica fume concrete after 10 years of exposure in Aursundet Bridge

V. Årskog
Ålesund University College, Ålesund, Norway

O. Sengul
Istanbul Technical University, Istanbul, Turkey

R. Dahl
Norwegian Public Roads Administration, Oslo, Norway

O.E. Gjørv
Norwegian University of Science and Technology, NTNU, Trondheim, Norway

ABSTRACT: Aursundet Bridge which is a four-span and 486 m long cantilever bridge with a post-tensioned box girder, is located on the west coast of Norway. When the bridge was constructed in 1993–1995, special efforts with both the structural design and the selection of concrete quality were made in order to obtain a good durability. Thus, for the most exposed parts of the bridge, a type of concrete with 12.5% silica fume by weight of cement was selected. An ordinary portland cement and a silica fume with more than 90% SiO_2 were used, and the concrete mixture had a water/binder ratio of 0.36. In order to find out how such a high-volume silica fume concrete would be able to withstand chloride penetration from the local marine environment, detailed investigations of the bridge after 3 and 10 years of exposure were carried out.

As part of the investigations, a number of concrete samples both for determination of chloride penetration and material characterization were removed. From the bridge columns, the samples for control of chloride penetration were partly taken from the splash zone at a level of 3.6 m above the mean water level and partly within the tidal zone having an extent of $+/-1.43$ m. The material characterization included the testing of degree of capillary suction, suction porosity and air porosity. These tests were carried out on "undisturbed" concrete samples taken from the bridge column by sawing 40–50 mm deep grooves about 20 mm apart on the concrete surface. Thus, upon arrival in the laboratory, it was possible to determine the moisture content in the undisturbed samples. After 10 years, accelerated chloride diffusivities were also determined by use of an accelerated migration test method. In addition, a control of compressive strength was carried out.

Both after 3 and 10 years of exposure, a significantly different chloride penetration between the windward and the leeward side of the bridge columns had taken place, and this is a pattern of chloride penetration which is typically observed on concrete bridges along the Norwegian coastline. For the most protected parts of concrete coastal bridges, experience shows that the salt accumulates, while for the most exposed parts, the intermittent rain is successively washing off the salt from the concrete surface.

For the high silica fume containing concrete, a very high resistance against chloride penetration was observed. Thus, after 10 years of exposure in the splash zone and the tidal zone, apparent diffusion coefficients of 5.3 and 3.3×10^{-13} m^2/s were observed, respectively, and based on the difference in chloride penetration after 3 and 10 years, a typical coefficient of 0.26 for the time dependence of the chloride diffusivity was obtained.

The material characterization also confirmed the high quality of the bridge concrete, with suction and air porosities typically varying from 11.8 to 15% and from 1.6 to 3.2%, respectively.

The observed values for accelerated chloride diffusivity and electrical resistivity of 5.0×10^{-12} m^2/s and 234 ohm·m, respectively, also reflect the high resistance of the concrete against chloride penetration.

By visual observations, the high silica fume containing concrete did not show any cracking of the concrete surface, but the test results reflect a high scatter and variability of the concrete quality, which is often observed for the testing of in situ concrete quality of existing concrete structures.

The concrete showed a high moisture content with values typically varying from 86.2 to 88.7%, but a value of 75% was also observed.

The combination of a concrete with a high resistance against chloride penetration and a nominal concrete cover of 100 mm had given the concrete bridge a high safety against steel corrosion. Thus, on the basis of a probability-based durability analysis, a probability for steel corrosion of approximately 8% after a period of 100 years was obtained.

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Effect of blast furnace slag on chloride penetration into concrete bridges

O. Sengul
Istanbul Technical University, Istanbul, Turkey

O.E. Gjørv
Norwegian University of Science and Technology – NTNU, Trondheim, Norway

ABSTRACT: In the present paper, the results of an experimental study are presented, where the objective was to provide more information about the effect of granulated blast furnace slag on the resistance of concrete against chloride penetration. In a concrete with a water/binder ratio of 0.40, the portland cement were successively replaced by 40, 60 and 80% of slag, the effect of which was tested by observing the change of both chloride diffusivity and electrical resistivity. For the testing of chloride diffusivity, an accelerated non-steady state migration test method was applied, while the electrical resistivity was tested by use of both the two electrode and the four electrode method.

At an age of 3 days, all the slag containing concretes showed a significantly higher diffusivity than that of the pure portland cement, but already after 7 days, the slag concretes showed a distinctly lower diffusivity compared to that of the pure portland cement concrete. For all the slag concretes, the reduction in chloride diffusivity was very rapid compared to that of the portland cement concrete, and the more slag the lower the diffusivity. By 28 days, the chloride diffusivity was reduced from 11.2 to 4.9, 3.6 and $2.3 \times 10^{-12} m^2/s$, respectively, while after 365 days, a chloride diffusivity of 3.0 to $1.0 \times 10^{-12} m^2$ for the slag concretes compared to $7.0 \times 10^{-12} m^2/s$ for that of the pure portland cement concrete was observed.

Based on the two electrode method, the electrical resistivity of all the slag containing concretes increased very rapidly compared to that of the pure portland cement, and the higher the slag content the higher the resistivity. Thus at 28 days, the resistivity of the slag concretes varied from 10 to 30 kohm·cm compared to approximately 5 kohm·cm for that of the pure portland cement. After 365 days, the resistivity varied from 23 to 50 kohm·cm for the slag concretes compared to approximately 8 kohm·cm for the portland cement concrete. Based on the four electrode method, the slag concretes increased even more rapidly than that obtained by the two electrode method. Thus at 28 days, the resistivity of the slag concretes varied from 22 to 58 kohm·cm compared to approximately 8 kohm·cm for that of the portland cement, and after 365 days, the resistivity varied from 38 to 83 kohm·cm for the slag concretes compared to approximately 12 kohm·cm for the portland cement concrete. Although the observed levels of resistivity obtained by the two test methods were quite different, the results showed a good correlation.

The obtained test results also showed a good correlation between chloride diffusivity and electrical resistivity, which is in accordance with both existing experience and theory. Thus, after establishing the relationship between the chloride diffusivity and the electrical resistivity for a given concrete with given temperature and moisture conditions, a regular monitoring of the electrical resistivity can be used as a basis for performance-based concrete quality control of chloride diffusivity during concrete construction.

In order to better demonstrate how the observed effect of slag would affect the risk of steel corrosion for a given type of concrete structure in a given type of marine environment, some

probability-based durability analyses were carried out. Based on a 10% probability of corrosion as a serviceability limit state, the pure portland cement type of concrete would exceed such a limit state already within a period of 10 years, while for increasing incorporation of slag, a substantially longer period would be reached, and the more slag the longer the period. Thus, for a slag content of 60%, a 10% probability of corrosion would not be reached within a period of 100 years.

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Improving durability through probabilistic design

R.M. Ferreira
Department of Civil Engineering, University of Minho, Portugal

ABSTRACT: In order to obtain a more controlled durability and long-term performance of concrete structures in chloride containing environment special care is needed in the design phase of reinforced concrete structures. The recent development of probability-based procedures has proven to give a more realistic basis for both durability design and condition assessment of reinforced concrete structures. Although there is still a lack of relevant data, this approach has been successfully applied to several new concrete structures, where requirements to a more controlled durability and service life have been specified.

Since parameters both for concrete durability and environmental exposure typically show a high scatter, a probability-based approach has shown to give a very powerful basis for durability analysis. This approach is primarily being applied for obtaining a more controlled durability and long-term performance of new concrete structures, but it also provides a very valuable basis for condition assessment of existing concrete structures in chloride containing environment.

1 INTRODUCTION

The recent development of probability-based procedures has proven to give a more realistic basis for both durability design and condition assessment of reinforced concrete structures. Although there is still a lack of relevant data, this approach has been successfully applied to several new concrete structures, where requirements to a more controlled durability and service life have been specified.

Since parameters both for concrete durability and environmental exposure typically show a high scatter, a probability-based approach provides a very powerful basis for durability analysis. This approach is primarily applied in order to obtain more controlled durability and long-term performance of new concrete structures, but it also provides a very valuable basis for condition assessment of existing concrete structures in chloride containing environments.

In the following article, a generic model is described and applied in order to demonstrate the importance and sensitivity of the various durability parameters affecting and controlling the durability of concrete structures, namely in a chloride containing environment.

A computer software with the model applied using a Monte Carlo simulation is used to perform the durability analysis (Duracon 2004, Ferreira 2004a).

2 MODEL DESCRIPTION

When corrosion is caused by chloride ingress, the service life is usually assumed to be equal to the initiation time. The period of propagation, which may be of short duration, is traditionally not taken into account because of the uncertainly with regard to the consequences of localized corrosion (Bertolini *et al* 2004).

The modelling of chloride penetration and time to depassivation is commonly based on Fick's Second Law of Diffusion. However, as this law does not correctly model the diffusion of ion through

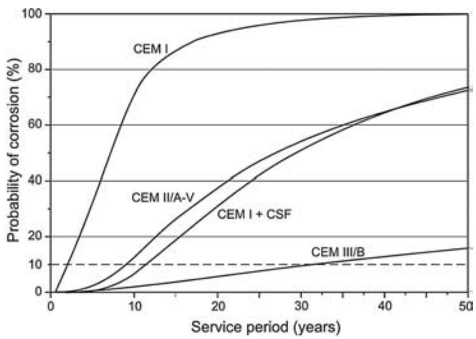

Figure 1. Effect of cement type on the probability of corrosion initiation.

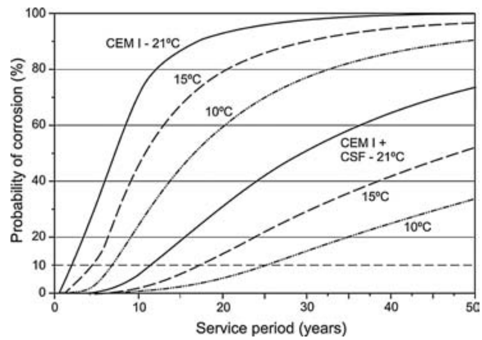

Figure 2. Effect of temperature on the probability of corrosion initiation.

concrete, modifications can be made to take into account several phenomena, such as the variation in time of the diffusion coefficient (due to cement hydration and influence of temperature) and the surface concentration of chlorides. The stochastic nature of the individual durability parameters (reflected in mean, standard deviation and type of distribution of relevant variables) require that the model be applied probabilistically.

The initiation time may be calculated as a function of the chloride transport properties of concrete (usually the apparent diffusion coefficient which may either be obtained from accelerated laboratory testing or curve fitting of chloride profiles from existing concrete structures); the time dependence of the diffusion coefficient; the surface chloride concentration influenced by the environment (obtainable from either measurements or previous experience); the thickness of the concrete cover and the critical chloride content for depassivation of embedded steel (both of which may be obtained from existing literature or other experience for the given type of cement and concrete). The model used in this case only contemplates the dependency of the diffusion coefficient with time and with temperature.

3 DURABILITY DESIGN OPTIONS

In order to demonstrate the use of this model for durability design and how it can be used to select appropriate concrete mixtures or concrete cover so as to obtain a more controlled durability; the effects of three different parameters on the durability performance of concrete are simulated below. In the first example, the cement type used in the concrete mixture was investigated. In the second example, the effect of varying temperature was studied, while in the third example, the concrete cover was varied.

3.1 *Effect of cement type and temperature*

In order to study the effect of cement type, four concrete mixtures with four different types of cement were produced. Based on four types of cement some concrete test mixtures were produced in order to test the chloride diffusivity. Three types of cement included a high-performance portland cement, a blended fly-ash cement and a blast-furnace slag cement with approximately 70% slag. The test mixtures had a cement content of 420 kg/m^3 and a w/c ratio of 0.45. The forth cement was a high-performance portland cement mixed with 10% CSF.

As can be seen from Figure 1, the type of cement has a significant effect on the probability of corrosion initiation. As can be seen from Figure 3, where three average temperatures were used: 21°C, 15°C and 10°C, the temperature is also of importance for the probability of corrosion initiation. The temperature influences greatly and can optimise the design procedure.

4 CONCLUSIONS

The importance of critical information concerning the durability design of concrete structures prior to the decision making process is invaluable. This paper demonstrates how this information can easily be generated. The influence of several parameters can be rapidly evaluated and design decisions can be made that improve and optimise durability performance. The type of cement influences significantly the probability of chloride-induced corrosion. The chloride diffusivity of the concrete is a very sensitive durability parameter. The choice of temperature is crucial for the durability design as it affects the parameters measured. A more realistic design procedure would find the optimal design parameters by combining the chosen cement type with the appropriate concrete cover for the average environmental temperature. For new concrete structures, this provides an appropriate basis for establishing overall durability criteria for the structures. For existing concrete structures, where the chloride front has still not reached the embedded steel, this procedure can be used for estimating the probability of corrosion initiation after a certain period of time.

Civil structural health monitoring

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Monitoring with fiber optic sensors of a cable-stayed bridge in the Port of Venice

A. Del Grosso
Department of Structural and Geotechnical Engineering, University of Genoa, Italy

A. Torre
D'Appolonia S.p.A., Italy

G. Brunetti
Tecniter S.r.l., Italy

D. Inaudi
Smartec S.A., Switzerland

A. Pietrogrande
The Port Authority of Venice, Italy

ABSTRACT: The Port Authority of Venice, in the framework of the development of the container and multipurpose terminals of the Marghera basin, situated at the inner edge of the Venice Lagoon, have decided the construction of a new road link between the national highway system and the port areas. The road link is crossing the "West Industrial Channel" and the railways serving the terminals, thus requiring the construction of a long-span bridge.

An international competition has been launched for the design of the bridge and related access viaducts. The preferred design was including a cable-stayed bridge formed by a composite steel and reinforced concrete beam, continuous over two spans of 105 m and 126 m in length, respectively. The bridge axis is a circular segment of 175 m radius. The deck on each of the two spans is supported by 9 cables, composed by 31 to 85 strands, attached to a reinforced concrete pylon nearly 80 m high.

The beams are composed by two I-shaped longitudinal steel girders connected by means of transverse diaphragms to a central box girder, in order to provide torsional stiffness. The concrete deck is made of precast slabs integrated by an in-situ layer.

Because of the characteristics of the bridge structure, namely its curvature, leading to a complex behavioral scheme, it has been decided to design and install a permanent monitoring system able to verify the design assumptions and to provide information on the response of the structure during its service life.

For the realization of the monitoring system, the choice has been made to use the SOFO™ fiber optics sensor family, recently extended to cover both static and dynamic responses.

The monitoring system has been installed during construction, in order to acquire control over the most significant construction phases. The system has been mainly designed for permanent static monitoring but linear deformation sensors and their locations have however been selected in order to allow the execution of periodic dynamic measurements and identification of the structural characteristics.

The system comprises 48 linear SOFO deformation sensors, 4 SOFO compatible fiber optic inclinometers and 24 temperature sensors placed on the two spans. In addition, each cable is equipped with a specially packaged SOFO sensor while 12 SOFO sensors and 6 thermocouples have been embedded in the structure of the pylon. An anemometer has also been placed on the top of the mast.

All the signals are routed to a control room placed in the basement of the mast, where the permanent static data acquisition hardware is located. When needed, the signal lines may be

manually switched to the dynamic data acquisition hardware, temporarily attached. The permanent acquisition system is linked to a standard telephone line for remote operation and control.

The paper is aimed at describing the data processing algorithms that have been adopted for the monitoring of the construction phase and of the in-service life of the bridge.

The main principle of the method that will be used for the in-service phase consists in subdividing the structure into a number of macro-elements that undergo relatively simple deformations. These sections are further segmented into cells containing only a few sensors or even a single one.

The first step requires the structure to be subdivided into sections. Each section is supposed to have a constant or continuously varying inertia, a constant load across its length and introduction of local forces and supports only at its ends. If the behavior of the materials in a section can be considered as homogeneous, the polynomial degree that best approximate its deformation is determined either analytically or using finite-element programs. If a degree $N+2$ is found to approximate satisfactory the deformation of the section, this will be subdivided into N cells.

For a beam with constant inertia the deformation is a polynomial of fifth degree and three cells are therefore necessary. For sections with variable inertia, it is sometimes useful to use cells with variable size. If local variations of the material behavior are expected, more cells are needed. In this case the number N and the size of the cells will be determined in such a way that the material properties inside each single cell can be considered reasonably constant. The deformation of the section will in this case be approximated by a polynomial of at most degree $N+2$.

After analyzing each cell separately one obtains an estimation of the curvature as a function of the curvilinear abscissa along the beam length. From the local curvatures it is possible to obtain the section's curvature function by fitting the cell's curvature values to a polynomial of appropriate degree. By combining the curvature function of each section one can now calculate the curvature of the whole structure and obtain its deformed shape by double integration.

During the construction phase, the monitoring system has stored the deformation data during the second phase of tensioning of the cables. The first tensioning of the stays has been carried out during their installation by load steps and continuous tension adjustments, until the project configuration was reached. The second phase has been carried out for braces of stays, following a procedure that involve pouring of the slab into portions of 10.50 m length, symmetrically with respect to the pylon and subsequently increasing the tension of the two stays interested by the poured slab portion.

A pre-processing of the deformation data in every section has been carried out: such analysis has shown that, especially in the sections near the stays, the sensors installed in the central beam were measuring anomalous deformations with respect to the sensors installed on the lateral beams. This observation has induced to discard, in the first data processing stage, the data of the central sensors, retaining only the data relative to the lateral sensors; in a subsequent phase, once the final configuration has been determined, the data of the sensors discarded could be used again.

The beam has been subdivided in two macro-elements, corresponding to the two main spans, everyone composed by 4 cells. Once obtained the mean curvatures in every section, these values have been interpolated by means of a polynomial of degree $n-1$ ($n=$ number of cells) in every macro-element, imposing the continuity of the curvature at the boundary.

The presence of a concentrated load inside the macro-elements, does not allow applying completely the algorithm previously described: therefore, for this phase it has been necessary to get the deformed shape of the beam by means of a finite-difference method, imposing zero displacements at the extremes and an additional continuity condition on the central support.

The use of this modified algorithm, therefore, has allowed obtaining the vertical deformed shape of the beam during the main prestressing phases: data comparison with topographic measures has shown satisfactory results. The processing has been repeated for the pylon, obtaining deformed shapes also comparable with the direct topographic measures.

A finite-element model of the bridge has been carried out, in order to compare the real behavior of the structure with that given by the model, with the aim of updating the parameters of the model.

The updated model will be used to verify the proof-load test on the structure that will be run shortly and to interpret the results of the subsequent in-service monitoring phase. During the proof-load test, measurements under dynamic excitation will also be performed.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Distributed fiber optic strain and temperature sensing for structural health monitoring

D. Inaudi & B. Glisic
SMARTEC SA, Manno, Switzerland

ABSTRACT: Distributed fiber optic sensing presents unique features that have no match in conventional sensing techniques. The ability to measure temperatures and strain at thousands of points along a single fiber is particularly interesting for the monitoring of large structures such as bridges, pipelines, flow lines, oil wells, dams and dikes. Sensing systems based on Brillouin and Raman scattering have been used for example to measure cables and pavement temperatures in bridges, detect pipeline leakages, prevent failure of pipelines installed in landslide areas, optimize oil production from wells and detect hot-spots in high-power cables.

The measurement instruments have been vastly improved in terms of spatial, temperature and strain resolution, distance range, measurement time, data processing and system cost. Analyzers for Brillouin and Raman scattering are now commercially available and offer reliable operation in field conditions.

New application opportunities have however demonstrated that the design and production of sensing cables is a critical element for the success of any distributed sensing instrumentation project. Although standard telecommunication cables can be effectively used for sensing ordinary temperatures, monitoring high and low temperatures or distributed strain present unique challenges that require specific cable designs.

This contribution presents different cable designs for high-temperature sensing, strain sensing and combined strain and temperature monitoring, as well as relevant application examples to the monitoring of civil and oil & gas structures.

1 INTRODUCTION

Structural health monitoring is certainly one of the most powerful management tools and is therefore gaining in importance in the civil engineering community. A typical health monitoring system is composed of a network of sensors that measure the parameters relevant to the state of the structure and its environment.

Conventional sensors based on mechanical and/or electrical transducers are able to measure most of these parameters. In the last few years, fiber optic sensors have made a slow but significant entrance in the sensor panorama. After an initial euphoric phase when optical fiber sensors seemed on the verge of invading the whole world of sensing, it now appears that this technology is only attractive in the cases where it offers superior performance compared to the more proven conventional sensors. The additional value can include an improved quality of the measurements, a better reliability, the possibility of replacing manual readings and operator judgment with automatic measurements, an easier installation and maintenance or a lower lifetime cost. The first successful industrial applications of fiber optic sensors to civil structural monitoring demonstrate that this technology is now sufficiently mature for a routine use and that it can compete as a peer with conventional instrumentation.

From many points of view, fiber optic sensors are indeed the ideal transducers for civil structural monitoring. Being durable, stable and insensitive to external perturbations, they are particularly interesting for the long-term health assessment of civil structures. This contribution will concentrate on distributed fiber optic sensors that offer unique characteristics that are unparalleled by the conventional sensors.

2 DISTRIBUTED FIBER OPTIC SENSORS

Unlike electrical and localized fiber optic sensors, distributed sensor offer the unique characteristic of being able to measure physical and chemical parameters along their whole length, allowing the measurements of thousands of points using a single transducer. The most developed technologies of distributed fiber optic sensors are based on Raman and Brillouin scattering. Both systems make use of a non-linear interaction between the light and the silica material of which the fiber is made. If light at a known wavelength is launched into a fiber, a very small amount of it is scattered back every point along the fiber. The scattered light contains components at wavelengths that are different form the original signal. These shifted components contain information on the local properties of the fiber, in particular their strain and temperature.

Traditional fiber optic cable design aims to the best possible protection of the fiber itself from any external influence. In particular it is necessary to shield the optical fiber form external humidity, side pressures, crushing and longitudinal strain applied to the cable. These design have proven very effective in guaranteeing the longevity of optical fibers used for communication and can be used as sensing elements for monitoring temperatures in the −20°C to +60°C range, in conjunction with Brillouin or Raman monitoring systems.

Sensing distributed temperature below 20°C or above 60°C requires a specific cable design, especially for Brillouin scattering systems, where it is important to guarantee that the optical fiber does not experience any strain that could be misinterpreted as a temperature change due to the cross-sensitivity between strain and temperature.

On the other hand, the strain sensitivity of Brillouin scattering prompts to the use of such systems for distributed strain sensing, in particular to monitor local deformations of large structures such as pipelines, landslides or dams. In these cases, the cable must faithfully transfer the structural strain to the optical fiber, a goal contradicting all experience form telecommunication cable design where the exact opposite is required.

Finally when sensing distributed strain it is necessary to simultaneously measure temperature to separate the two components. This is usually obtained by installing a strain and a temperature sensing cables in parallel. It would be therefore desirable to combine the two functions into a single packaging.

The use of distributed fiber optic sensors for the monitoring of civil structures and infrastructures opens new possibilities that have no equivalent in the conventional sensors system. Thanks to the use of a single optical fiber with a length of tens of kilometers has sensing elements; it becomes possible to obtain dense information on the structure's strain and temperature distribution. This technology is therefore particularly suitable for applications to large or elongated structures; such as dams, large bridges and pipelines.

The presented applications examples show that using an appropriate sensor design, it is possible to successfully install distributed sensors on large structures and obtain useful data for the evaluation and management of the monitored structures.

In particular the paper presents results obtained in the following monitoring projects:

- Luzzone Dam: Monitoring of 2D temperature distribution of concrete during setting and in the long term.
- Plavinu Dam: Permanent monitoring of bitumen joints.
- Gas Pipeline: Periodic monitoring of strain distribution in a gas pipeline installed in a landslide area.

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Development of structural health monitoring methodologies for cable-stayed bridges by fiber optic sensors

L.M. Giacosa
Polytechnic of Torino, Torino, Italy

F. Ansari
University of Illinois at Chicago, Chicago, USA

A. De Stefano
Polytechnic of Torino, Torino, Italy

ABSTRACT: Recent surge in design and construction of cable stayed structures, especially bridges necessitates monitoring to assess structural health, safety and behavior for further improvements in design. Many of these structures are currently being monitored by using conventional sensors such as accelerometers, and strain gauges. Moreover, most of the sensors are installed within the deck system and employ vibration based structural health monitoring methodologies for assessment of structural characteristics. This approach has proven effective for certain types and sizes of bridges. However, effective condition monitoring of larger and more complicated structures require placement of multitudes of sensors in order to extract useful information beyond the signal to noise ratio levels experienced in such structures. Moreover, since stay cables contribute immensely to the capacity of the bridge in carrying the structural loads, direct monitoring of their health will be essential in structural health monitoring programs. The work presented here will discuss the collaborative research that has been undertaken by the University of Illinois at Chicago and the Polytechnic of Torino in assessment of fiber optic sensors in structural health monitoring of cables. This program aims at development and testing of robust field deployable fiber optic sensors based on Bragg gratings and interferometric principles for monitoring of cable forces, deformations and the dynamic response. Besides all the obvious advantages of optical fibers sensors in such applications, the very high signal to noise ratio levels in optical sensing will prove to be very effective in this application. Structural health monitoring analysis techniques are developed for direct assessment of the bridge condition based on monitoring of the cable stays. This research will potentially provide a cost effective approach in structural health monitoring of cable stayed bridges.

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Determination of concrete properties by fiber optic sensor

Q. Li
Department of Hydraulic Engineering, Tsinghua University, Beijing, P. R. China

F. Ansari
Department of Civil & Materials Engineering, University of Illinois at Chicago, Chicago, Illinois, USA

1 INTRODUCTION

The objective for this study was to develop a theoretical model based on the ideal elasto-plastic constitutive law for the protective coating of the optical fiber. An optical fiber sensor was embedded in each of the concrete specimens with pre-cast side cracks in order to keep the final crack location within the gauge length of the fiber optic sensor for measurement of axial strains, as shown in figure 1. The analysis for the theoretical model in this paper is based on the following assumptions:

(1) Both the glass core and the cladding are considered as the same material and collectively referred to as the fiber core. And the fiber core is assumed to be a linear elastic material.
(2) The protective coating is supposed to be an ideal elasto-plastic material, and its constitutive relationship could be written as the following expression:

$$\begin{cases} \tau = G_c \gamma , & \gamma < \dfrac{\tau_{cr}}{G_c} \\ \tau = \tau_{cr} , & \gamma \geq \dfrac{\tau_{cr}}{G_c} \end{cases} \tag{1}$$

Where τ is the shear stress, γ is the shear strain, G_c is the shear modulus of the coating, and τ_{cr} is the critical elastic shear stress which can be evaluated through calibration test.
(3) The bonding surface between the core and the polymer coating and that between the polymer coating and the matrix concrete are firmly attached with each other very well, no debonding taking place.

2 THEORETICAL MODEL

2.1 *The elastic phase*

When the polymer coating is in elastic state, the strain-transferring coefficient from fiber core to matrix concrete was derived as

$$\alpha(k, L) = \left[1 - \frac{\cosh(kL) \ - 1}{kL \sinh(kL)} \right] \tag{2}$$

In which, L is the half gauge length of the fiber optic sensor, k is a material parameter for fiber optic sensor and is expressed as

$$K^2 = \frac{2G_c}{r_g^2 E_g \ln(r_c / r_g)} \tag{3}$$

where r_g, r_c represent the radii of the fiber core and the outer edge of the polymer coating respectively, E_g is the modulus of elasticity of the fiber core.

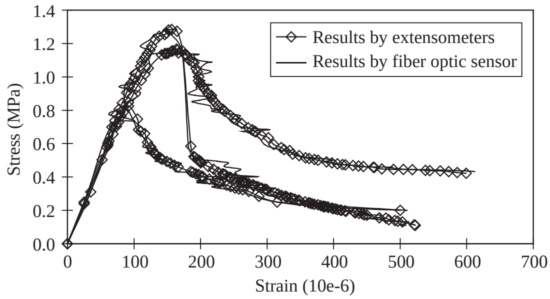

Figure 1. Stress strain curves of concrete in uniaxial tension measured by fiber optic sensor and by extensometer respectively.

2.2 *The elasto-plastic phase*

With the increase of the strain in the matrix concrete, the shear stress at the interface between the polymer coating and the matrix concrete will increase to the elastic critical one, and the plastic region will appear afterward, the strain-transferring coefficient in the elasto-plastic phase can be expressed as

$$\alpha(k,L,l_0) = 1 - \frac{1}{kL\beta}[\cosh(kl_0) - 1] - \frac{k}{L\beta}\frac{(L-l_0)^2}{2}\cosh(kl_0) - \frac{(L-l_0)}{L\beta}\sinh(kl_0) \tag{4}$$

3 EXPERIMENTAL INVESTIGATION

Three specimens were tested in this paper. The experimental results for these three concrete specimens in uniaxial tension measured by the embedded fiber optic sensor and the extensometer respectively were shown in figure 1. These results indicate that the theoretical and the experimental results are in good agreement. And this implies that the model proposed in this paper can predict the experimental results.

4 CONCLUSIONS

In this paper, the theoretical model for internal strain measurement of concrete in uniaxial tension by embedded fiber optic sensor was proposed. The fiber core and the matrix concrete were supposed to be elastic materials, the coating of the optical fiber was assumed to be ideal elasto-plastic. All the bonding surfaces were supposed to be intact, that is to say, there is no relative slip deformation at the interfaces. Some experiments for concrete specimens in uniaxial tension were done to verify the validation of the theoretical model. The previous study supports the following conclusions:

(1) Based on the ideal elasto-plasticity theory, the process for the shear transferring of the fiber optic sensor can be divided into three phases with the increase of the strain in the matrix concrete, they are the elastic phase, the elasto-plastic phase.
(2) The strain-transferring coefficient for the fiber optic sensor embedded in concrete prism specimen under uniaxial tension derived in this paper can be used to predict the strain exerted in the tensile matrix concrete.

ACKNOWLEDGMENTS

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Multiple fiber optic twin-sensor-array based on Michelson optical low-coherence reflectometer

L. Yuan, J. Yang & Z. Liu
Department of Physics, Harbin Engineering University, Harbin, China

Qingbo Wen, Chongjie Liu, Yuxin Jie & Guangxin Li
Department of Hydraulic Engineering, Tsinghua University, Beijing, P. R. China

ABSTRACT: There has been considerable interest recently in the development of fiber optic sensors based on white light interferometry. The use of such a technique for distributed strain or temperature sensing in advanced composite or other structural materials has been discussed in several recent articles. Fiber optic Michelson sensor arrays are of particular interest for such applications because a number of reflective-type sensors networked either in a serial or a parallel topology may be interrogated by use of a common path-length variable reference, thus keeping the system cost down. Several schemes have been reported for multiplexing this type of sensors. These include coherence turning, time-division, Spatial division, duplicate fiber optic switch and $1 \times N$ star coupler multiplexing technique.

In this paper, we propose and demonstrate a white light twin-sensor-array scheme that measures the absolute optical path lengths between each fiber segment and reflective surfaces. The sensing principle is shown in Figure 1. The broadband LED source is directly coupled into the fiber optic Michelson interferometer by passing thorough a 3 dB coupler and launched into the twin-sensor-array. The twin-sensor-array consist of $2 \times N$ fiber segments (N twin-sensors) connected in series with partial reflectors in-between the adjacent fiber segments, configuring a Michelson interferometer based multiplexed fiber optic sensor array. The reflected signals of each sensor pair retune and travel in the same path towards to the PIN detector end. One arm of the fiber optic Michelson interferometer as optical path adjusting part has been used to demodulate the twin-sensor-arrays. So that each individual sensor pair is corresponding to a unique interference peak due to the optical path length is different each other. For a sensor pair, if one of the twin sensors is used as a strain sensor, while the other sensor can be used as a temperature compensated sensor.

Therefore, the proposed sensing scheme will be useful for temperature compensation of distributed strain measurement. An important application of the sensing system could be deformation sensing in smart structures.

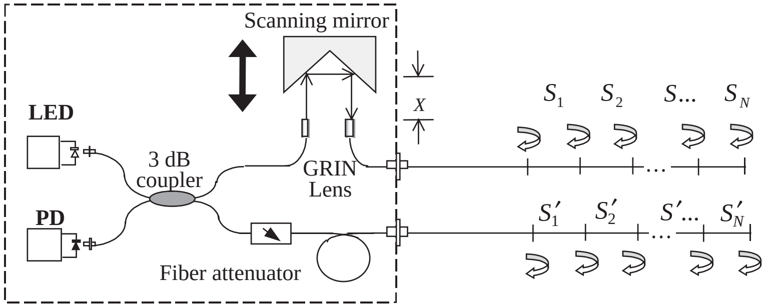

Figure 1. Working principle of the twin-array Michelson fiber optic interferometric strain sensing system.

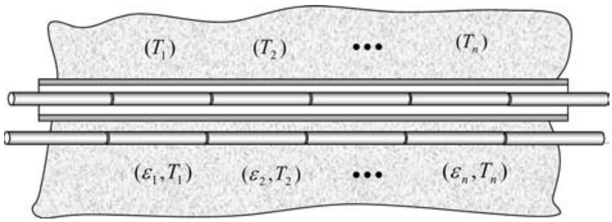

Figure 2. For the case of the twin sensor array arrangement in the structure: the sensor array has been embedded in the concrete material, while the reference array has been put on the pipe in free state near by the sensor array.

Figure 2 shows a twin-sensor-arrays has been embedded in a concrete structure. A number of fiber segments pairs l_i and l'_i are connected in serial to form the twin-sensor-arrays, which is further connected to a lead in/out fiber of length L_0 and L'_0. In the sensing system, the length of the two lead in/out fiber cables has been chosen as nearly equal, and same as for each fiber segment pairs, i.e.: the optical path length of the reference arm L'_0 can be varied through the use of a moving graded index (GRIN) lens (as shown in Fig. 1(a)) or a scanning right angle mirror (as shown in Fig. 1(b)). One adjusts the optical path of the reference arm to match and trace the change of the twin-fiber-sensor gauge length in each sensing pairs. When the optical-path difference (OPD) between the sensing branch and the reference branch falls within the coherence length of the light source, a white light fringe pattern is produced. The central fringe, which is located in the center of the fringe pattern and has the highest amplitude, corresponds to the exact path match of the two optical paths.

Fiber optic white light interferometric twin sensor array multiplexing technique has been demonstrated, which the matching multi-wave was demodulated by a moving GRIN lens or a right-angle mirror. The fiber optic sensors multiplexing capacity is strongly depend on the light source power of the sensing system.

The proposed sensing scheme will be useful for the measurement of temperature or strain. An important application could be deformation sensing in smart structures. By incorporating fiber optic twin sensor array into structures such as large-scale buildings, bridges, dams, tunnels and highways, smart structures can be realized for situations where material strains must be monitored throughout the lifetime of the structure.

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Intrinsic polymer optical fiber sensors for civil infrastructure systems

S. Kiesel, P. Van Vickle & K. Peters
Department of Mechanical and Aerospace Engineering, North Carolina State University, USA

O. Abdi, T. Hassan & M. Kowalsky
Department of Civil, Construction, and Environmental Engineering, North Carolina State University, USA

1 INTRODUCTION

In order to truly understand the failure mechanisms of structures, it will be essential to have reliable sensors that can measure large strains at high strain rates. Such a sensor would have significant immediate implications for structural testing and monitoring as researchers would gain previously unavailable insight into the localized failure mechanisms due to inelastic deformation and damage accumulation. Recent advances in the fabrication of singlemode polymer optical fibers (POF) have made it possible to extend POFs to interferometric sensing, however several challenges make the application of the interferometric sensor in the POF more difficult than its counterpart in a silica optical fiber due to the large strain values to be measured. These include the: (1) finite deformation of the POF cross-section at high strain values; (2) nonlinear strain optic effects in the polymer; and (3) attenuation with strain of the POF.

2 ANALYSIS

For the treatment of the POF sensor at large strain values, we introduce the tensor $\boldsymbol{g}$ of dimension three to represent the nonlinear strain-optic effect (using the summation convention). For an initially isotropic material, the components of the linear strain-optic tensor, $\boldsymbol{p}$, are given in terms of two constants (Nye 1985). Since the most common loading case for calibration experiments is pure axial loading of the optical fiber, we derive the phase shift, $\Delta\varphi$, in the POF interferometer for this case. Applying the nonlinear photo elastic effect and the finite deformation solution of Bertholds & Dandliker (1987), and expanding $\Delta\varphi$ as a Taylor series about $\varepsilon = 0$, we obtain the linear and nonlinear sensitivities of the interferometer due to strain,

$$\Delta\varphi \cong \left.\frac{d(\Delta\varphi)}{d\varepsilon}\right|_{\varepsilon=0} \varepsilon + \left.\frac{d^2(\Delta\varphi)}{d\varepsilon^2}\right|_{\varepsilon=0} \frac{\varepsilon^2}{2!} \tag{1}$$

where,

$$\left.\frac{d(\Delta\varphi)}{d\varepsilon}\right|_{\varepsilon=0} = \frac{2\pi}{\lambda} Ln_0\left\{1 - \frac{n_0^2}{2}\left[p_{12} - \nu(p_{11} + p_{12})\right]\right\}$$
$$\frac{1}{2!}\left.\frac{d^2(\Delta\varphi)}{d\varepsilon^2}\right|_{\varepsilon=0} = \frac{2\pi}{\lambda} Ln_0\left\{\delta - \frac{n_0^2}{2}\left[\left(\delta + \frac{1}{4}\right)p_{12} + \left(\alpha + \frac{5}{4}\nu\right)(p_{11} + p_{12}) + \nu(g_{111} - g_{123}) + (2+\nu)g_{122}\right]\right\} \tag{2}$$

where ε is the strain in the axial direction, L is the length of the optical fiber, λ is the wavelength of the lightwave, n_0 is the initial mode index of refraction and δ and α are the nonlinearity constants of the modulus and Poisson's ratio respectively. The use of the POF to measure large strain magnitudes in

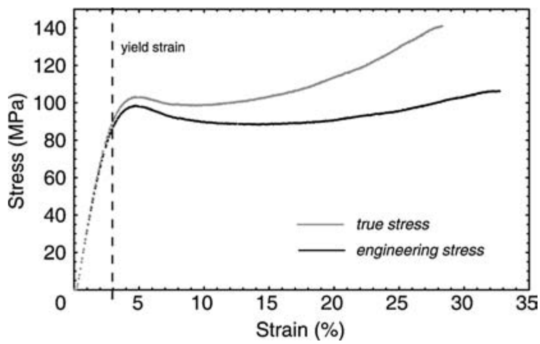

Figure 1. Measured stress-strain curve for PMMA optical fiber. Plotted data points are the mean of ten fiber samples. Loading rate was 60 mm/min.

structures under general loading conditions requires the calibration of four mechanical parameters (E, v, δ, α) and six opto-mechanical properties ($p_{11}, p_{12}, g_{111}, g_{122}, g_{112}, g_{123}$).

3 EXPERIMENTAL RESULTS

Preliminary tensile tests of the POF were performed to identify the modulus of the POF in the linear range and to identify the strain value at which the fiber behavior was no longer linear. Figure 1 plots the stress-strain curves obtained from these tests. In addition to the mechanical and optical properties of the POF, the bond strength of the POF with typical civil structural materials was also determined. POFs were embedded during casting of three separate potential materials: mortar, hydrostone, and cement paste, for pullout testing. The cement paste was the optimal material system in which to embed the POF due to the ease of mixing and casting and the strong bond between the cement and the POF. An absolute position time-of-flight telemeter technique was also been developed for the POF sensors for which one can adjust the desired strain measurement range through the choice modulation frequency (Donati 2004).

4 CONCLUSIONS

This article derives the required calibration tests for a large strain POF sensor based on phase shift measurements. Future experimental studies will perform these calibrations, motivated by the above analysis. Finally a data acquisition system was designed and implemented to measure the phase shift in the POF sensor over the strain range required.

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Implementation of a fiber Bragg grating sensor network for structural monitoring of a new stone bridge

L.A. Ferreira & F.M. Araújo
INESC Porto, Campus da FEUP, Porto, Portugal
FiberSensing, Portugal

C. Barbosa & N. Costa
FiberSensing, Portugal

A. Arêde, A. Costa & P. Costa
Fac. Eng. Universidade do Porto, Porto, Portugal

As it is the case in many European countries, there is in Portugal a large number of bridges with stone arches, which are traditionally old and were not designed for the loads and traffic that are presently crossing them. Moreover, several bridges of this type show clear signs of structural degradation and lack of maintenance that may lead into situations where safety is not assured. Therefore, the possibility of monitoring and analyzing the structural behavior of a real bridge with stone arches represents an important contribution for the knowledge of this type of structures. The work presented here refers to the development and implementation of a fiber Bragg grating (FBG) based sensing network for structural monitoring of a new stone bridge over Vizela River in Vila Fria, Felgueiras, Portugal (Figure 1).

In general, modeling structures like the Vila Fria bridge brings several issues on how to define parameters to calibrate the mathematical models of the structure and on how to be certain about the results predicted by those models. Therefore, the possibility of monitoring an arch stone bridge from the beginning of its construction is of particular interest because it allows the installation of a large number of sensors according to a pre-established plan and accessing points impossible to achieve in structures that are already built. With the large amount of information that can be generated, improved calibration and models validation can be obtained. The fact that the bridge can then be monitored for a long period also allows the evaluation of its structural behavior and its relation to the usually observed pathologies in this kind of structures.

Nevertheless, the establishment of a large sensing network for monitoring multiple parameters that are structurally relevant is not always easy when using conventional electric technology. In fact, conventional sensors suffer from high EMI/RFI sensitivity, environment induced drift and require individual electrical stimulus that makes large scale structural health monitoring highly complex. FBGs[1] constitute a particularly appropriate and competitive alternative for structural health monitoring applications[2,3]. These sensors add to the long recognized advantages of fiber optic sensors (e.g., immunity to EMI/RFI, remote monitoring, small size and weight, electrical isolation, intrinsically safe operation, high sensitivity, long-term reliability) the inherent

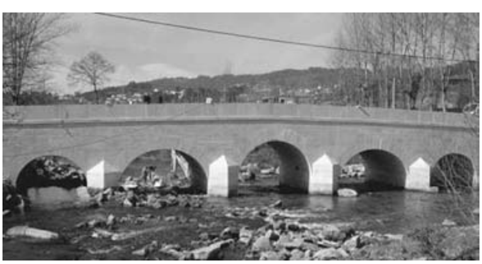

Figure 1. The Vila Fria new stone masonry bridge.

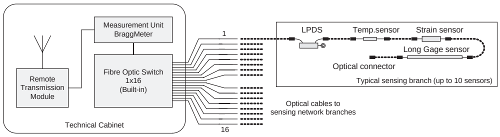

Figure 2. Fiber optic sensing network architecture.

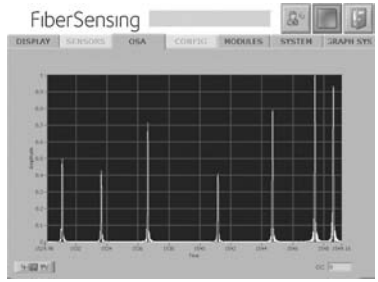

Figure 3. Branch 8 reflection.

multiplexing capability and the ability to provide absolute measurements[4]. FBG technology is therefore becoming the natural substitute for the conventional sensing technologies by easing health monitoring of large structures during construction, load tests and long-term service[5].

The monitoring of the Vila Fria stone masonry bridge pursued two main objectives: the establishment and calibration of analytical modeling techniques suitable for the numerical simulation of similar constructions; the evaluation and systematization of the main features of this type of structures in terms of their structural behavior and their relation with the most frequent structural pathologies. To achieve these goals, a vast instrumentation project was put forward to transform the bridge into a "live laboratory", which included, in addition to the installation of several conventional sensors[6], the deployment of a large fiber sensing network for the measurement of temperature (28 sensors), displacement (48 sensors) and strain (9 sensors). Some of the sensors used in the bridge were specifically developed by FiberSensing for this project. Most temperature sensors were used for referentiation of both fiber optic and conventional sensors.

A tree network configuration with 15 branches was preferred. To interrogate this sensing network, a BraggMeter measurement unit containing an optical switch with 16 channels was adopted. In total, more than 800 m of fiber optic cables were installed in the bridge. Figure 2 shows a simplified scheme of the fiber optic sensing network architecture and in Figure 3 the spectrum reflected by the sensors in branch 8 obtained with the BraggMeter is presented as an example. In general, the sensors developed for this project demonstrated a good performance in laboratory tests, and it was verified to keep their characteristics months after the installation.

A remote transmission module is presently managing the transmission of the measured data both from the BraggMeter and the conventional data takers over a standard GPRS communication protocol to a computer in FEUP. This system will soon start continuous operation in order to update a website completely dedicated to the real time observation and analysis of monitoring data relative to the Vila Fria bridge. It is also expected that the results relative to the general load test will be available at the website shortly after its completion, which is expected to happen within the next few weeks.

The authors would like to thank Teresa Restivo from the DEEC-FEUP, the LESE staff (Daniela Gloria and Valdemar Luís), Filipe Ferreira from the Contractor, and Marta Girão and Filipe Sá from FiberSensing for their valuable contribution to this work.

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Implementation of a fiber Bragg grating sensor network for structural monitoring of a rehabilitated metallic bridge

F.M. Araújo & L.A. Ferreira
INESC Porto, Campus da FEUP, Porto, Portugal
FiberSensing, Portugal

C. Barbosa & N. Costa
FiberSensing, Portugal

Luiz I Bridge crosses the Douro river between Porto and Vila Nova de Gaia since 1886. It is a steel arch bridge 172 m long and 45 m maximum height –, that ensures river crossing at two different levels (Figure 1). Since its construction several rehabilitation services were preformed on the structure, mainly stimulated by traffic category exchange along the years. The implementation of the Metro network on the metropolitan area of Porto imposed the most recent of these rehabilitations. It was decided that the upper deck of Luiz I Bridge would accommodate two parallel Metro lines to extend the infrastructure to Vila Nova de Gaia. This connection is considered to be critical to the accomplishment of the entire project given its strategic importance.

Although a small scale electric based monitoring system was implemented to evaluate the progress of the rehabilitation process, it was decided to replace this system by a high performance monitoring system based on fiber Bragg grating (FBG) technology to continuously assess the structural integrity of Luiz I Bridge[1]. FBGs[2] constitute a particularly appropriate and competitive alternative for structural health monitoring applications[3,4]. These sensors add to the long recognized advantages of fiber optic sensors the inherent multiplexing capability and the ability to provide absolute measurements. FBG technology is therefore becoming the natural substitute for the conventional sensing technologies by easing health monitoring of large structures during construction, load tests and long-term service[5].

The fiber optic monitoring network installed in the bridge consisted in a tree configuration composed by a main cable and 15 branches with up-to 10 sensors connected in series. The complete network comprises 118 composite strain and 10 temperature sensors for effective environmental

Figure 1. The Luiz I Bridge during the rehabilitation process.

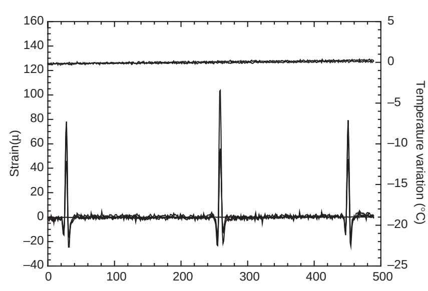

Figure 2. Data collected for one branch with four strain and two temperature sensors during traffic.

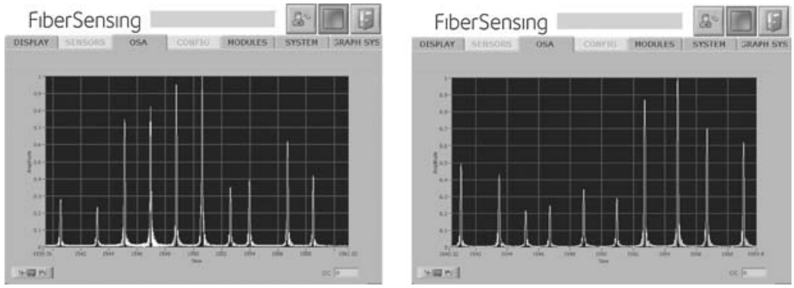

Figure 3. Spectral responses of the FBG sensors included in two different branches.

drift compensation. The implementation of such a system imposed most demanding challenges in terms of product development from the sensor design to the sensing network deployment. The instrumentation service was contracted by Metro do Porto to LABEST, being the FBG sensor manufacturing and network design and implementation sub-contracted to FiberSensing.

As an example, Figure 2 shows data collected for one branch containing four strain and two temperature sensors during typical metro traffic across the bridge during preliminary tests.

Even though a large number of experimental demonstrations of the use of FBG sensors in Civil Engineering applications can be found in literature, it is still scarce the number of qualified sensors available in the market as standard solutions. Since the beginning of the company activity that the policy of product development at FiberSensing was targeted towards the development of complete monitoring solutions. Such a practice implies the investment of a great initial effort on the definition of manufacturing processes that can be exploited after in an industrial basis. In the present case a set of requirements was defined by LABEST for the strain sensor design. It should be made of composite material with a rectangular cross section, being its longitudinal dimensions at least 100 mm. In order to manufacture such a sensor in a reproducible manner, a dedicated computer controlled manufacturing workstation was designed and implemented.

It was also specified that some of the strain sensors should integrate a temperature sensor for effective temperature compensation. The temperature sensor was implemented in-line on the composite strain sensor fiber pigtail by proper mechanical isolation of the cross-strain effects. A stainless steel external package was then applied for mechanical protection.

The sensor field deployment was performed by LABEST technical team in close collaboration with FiberSensing. The critical points of the structure were defined in a number of 59 by the technical staff of LABEST, being each of these points instrumented with a pair of composite strain sensors. The complete sensing network comprised 118 strain and 10 temperature sensors for effective compensation of environmental temperature fluctuations. The decision of limiting the maximum number of branches to 16 imposed the major constrain to the selection of the sensor network configuration. It was selected a tree configuration composed by a main optical cable with derivation to each of the 15 branches containing up-to 10 sensors connected in series. The main optical cable has a total length of 200 m and 3 access points. The complete fiber optic network comprises more than 3000 m of optical cables and hundreds of FC/PC connections.

The initial performance evaluation of the sensing network installed in Luiz I Bridge was done with a BraggMeter measurement unit at the end of the main cable located at the central cabinet. Typical spectral responses obtained for two of the fifteen branches are shown in Figure 3. Complete successful installation of all the 128 sensors was achieved, demonstrating full compliance with the initial instrumentation project. This was only possible due to the careful supervision of all the field deployment procedures.

The authors would like to thank Filipe Sá and Marta Girão from FiberSensing for their valuable contribution to this work. The technical support from the contractor LABEST is also greatly acknowledged.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

The deterioration of concrete deck slabs in bridges – A Canadian experience

L.G. Jaeger
Dalhousie University, Halifax, Nova Scotia, Canada

W. Saltzberg
ISIS Canada Research Network, Agricultural & Civil Engineering Bldg, University of Manitoba, Winnipeg, MB, Canada

G. Tadros
SPECO Engineering, Calgary, Alberta, Canada

N. Banthia
University of British Columbia, Vancouver, British Columbia, Canada

1 INTRODUCTION AND METHODOLOGY

In the severe Canadian winters, large quantities of salt are put on roads and bridges. The result is that the concrete deck slabs of highway bridges, when reinforced with steel, experience rapid deterioration because of corrosion of the steel. Highway bridges are usually designed for a life of 75 years, but deck slabs with steel reinforcement need to be replaced after about 25 years. Recently ISIS Canada has been studying the deterioration of concrete deck slabs with a view to comparing the life expectancy of slabs reinforced with GFRP with those reinforced with steel. In this study five conditions of slab are used, as follows:

C1 Excellent Condition; needs only routine maintenance.
C2 Very Good Condition; some minor repair needed.
C3 Fully Functional; needs moderate repair.
C4 Barely Functional; needs large repair.
C5 No Longer Functional; needs replacement.

A probabilistic Markhov method is used, with the probability q of a deck slab deteriorating from one condition to the next one below in a given interval of time being specified; q increases both with the age of the slab and the amount of deterioration that has already taken place.

2 EXPERT ENGINEERING JUDGEMENT

As part of the ISIS Canada study, a number of Canada's most experienced bridge designers were asked to estimate the pattern of deterioration in their provinces of deck slabs reinforced with black steel without any epoxy coating or protective membrane. The results showed remarkable unanimity across Canada. Table I shows the estimated condition vectors of 25-year old deck slabs reinforced with black steel; in this table, for convenience, the conditions C2 and C3 are combined into a single figure representing "Light to Moderate Damage" and conditions C4 and C5 are similarly combined to represent "Very Heavy to Total Damage". We see that there are two patterns of deterioration which are, however, relatively close to one another, that for British Columbia and Alberta being somewhat more severe than for Manitoba, Ontario and Quebec.

The study also looked at the pattern of deterioration up to the 24 or 25 year age point. Table 2 shows a deterioration matrix that is closely representative of Manitoba, Ontario and Quebec; the

Table 1. Condition Vectors by Engineering Judgement. 25 Year Old Slabs.

	BC (Brett)	Alberta (Tadros)	Manitoba (Saltzberg)	Ontario (Bakht)	Québec (Richard)	Canada Average
C1	5	5	5	3	5	5
C2+C3	25	25	43	42	40	35
C4+C5	70	70	52	55	55	60

Table 2. Deterioration matrix for Manitoba (Saltzberg).

Year	0	6	12	18	24
C1	100	70	42	21	4
C2	0	30	43	30	17
C3	0	0	15	34	30
C4	0	0	0	15	34
C5	0	0	0	0	15

time interval between columns is six years. The pattern for British Columbia and Alberta is very similar if the time interval between columns is takes as five or five and a half years.

3 A POSSIBLE GENERALIZED DETERIORATION MATRIX

In Table 2 the time interval is T = 6 years. A possible generalization of the matrix is to keep the numbers the same but to treat the time interval "T" between the columns as a variable. Thus in a given case, with a given water/cement ratio, given material of reinforcement, etc., the designer/analyst would select the most appropriate value of "T" and the existing matrix would stand.

An alternative route, is also being actively considered at this time. The approach is based upon taking the concrete alone, with no inclusions of any kind, and determining its service life. Thereafter, the addition of other elements in the deck slab design are accompanied by changes in the assessed service life; for example if black steel rebar is introduced, the service life is diminished; and if a membrane is provided, the service life is increased.

For a given design, the resulting estimate of service life is foreseen to be linked to the time interval "T" referred to above.

4 ONGOING WORK

Intensive study is currently being carried out to establish whether the generalised form of the matrix can be applied to deck slabs reinforced with GFRP and if so what the appropriate value of "T" years between the columns of the matrix will be. It is foreseen that these findings will be reported within one year.

ACKNOWLEDGEMENTS

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Weighing-in-motion of truck axle weights through a bridge

B. Bakht
JMBT Structures Research Inc., Toronto, Canada

A. Mufti
ISIS Canada, Winnipeg, Canada

G. Tadros
ISIS Canada, Calgary, Canada

R. Eden
Manitoba Floodway Authority, Winnipeg, Canada

G. Mourant
Stantec Consulting Ltd., Winnipeg, Canada

EXTENDED ABSTRACT: Truck weight data have been gathered in several countries during the last two decades by using medium span bridges as weighing-in-motion (WIM) devices. Several methods are used to obtain the axle weight and gross vehicle weight (GVW) data from observed bridge responses; all these methods lead to unavoidable errors. While errors in the estimation of GVW, being as large as 10%, might be considered acceptable, errors in estimating axle weights by most methods are unacceptably large. Recently, Japanese researchers have tested a WIM method, called the reaction force method, in which vertical stiffeners on support diaphragms of steel girder bridges are used to measure axle weights. This method, which also requires knowledge of vehicle speed, is yet to be improved to give axle weight data of acceptable accuracy. A test on a Canadian steel girder bridge provided an opportunity to verify the Japanese reaction force method as well as to develop another method to measure truck axle weights. The Shell River Bridge in Manitoba has four steel girders, with T-section stiffeners on both sides of each girder above its bearings. All these stiffeners were installed with sensors to measure vertical strains. In addition, all four girders had strain gauges at the mid-span and at sections near the simple supports to measure longitudinal strains. An attempt has been made in the paper to examine the Japanese reaction force method and the other proposed method so as to suggest possible improvements in the methods, which will lead to economy in the amount of collected data and improvement in the accuracy of predictions.

Consider a simply supported beam of span L under a set of three point loads, P_1, P_2 and P_3, moving from left to right. The left hand support of the beam is installed with a gauge to monitor vertical strains, which are directly proportional to the left hand reaction in the beam. It is assumed that the reaction is obtained by multiplying the strains in the gauge with a constant C. The reaction due to P_1 is CP_1 when the load is directly above the support. As the load moves towards the right support, the reaction at the left hand support reduces linearly. The load P_2 straddles the left hand support when load P_1 has moved a distance between P_1 and P_2. The sum of influence lines due to the three loads can be seen to have three distinct peaks. Utilizing the influence line components, it can be concluded that the three peaks of the cumulative influence line, designated as R_1, R_2 and R_3, lead to three equations, the sequential solving of which requires knowledge of L, the distances between the various point loads and the determination of the constant C. With a known value of C, the 1st equation directly gives the value of P_1, utilizing which and the known value of the distance between P_1 and P_2, the 2nd equation gives the value of P_2, and so on.

The advantage in using the Japanese reaction force method is that the influence line for the reaction has distinct peaks, which are associated with either single wheel loads or with groups of closely spaced wheels. Since the influence line for moments near mid-span is devoid of distinct peaks, these moments cannot provide reliable information about individual axle loads. However, if moments in a girder are investigated near one of its support, the influence line for the moment will be found to have distinct peaks. For example, consider the influence lines for bending moment in a simply supported beam near one of its supports due to each of three moving point loads. It can be seen that similar to the cumulative influence line for reactions, the cumulative influence line for moments near a support also has three distinctive peaks. Using the same logic that led to the previous set of three equations, such cumulative influence lines can be used to compute axle weights. If a girder is installed with strain gauges near both its supports then the time lag between peaks due to the same loads at the two instrumented sections can be used to calculate the speed of the vehicle. Thereafter, the determination of the distances between successive peaks, i.e. the distance between axles or axle groups, is a straightforward matter.

The Shell River Bridge has three simply supported spans, each nearly 26.4 m long. Each of its spans comprises four steel plate girders at a spacing of 2.784 m; this bridge was tested under a 5-axle tractor-trailer and a 3-axle dump truck. it was found that the strains in the end diaphragms were sensitive to the transverse position of vehicles, which position can be determined from longitudinal girder strains at the mid-span. A method is proposed in the paper to determine the transverse position of a vehicle from these strains. The proposed technique of measuring axle weights is summarized in the following with particular reference to the sensors on the shell river bridge. The technique is, however, general enough to be applied to any girder bridge. The following steps are proposed for continuous monitoring of the 4-girder Shell River Bridge with the objective of measuring axle weights of moving vehicles. The two marked travel lanes of the bridge lie directly over the two middle girders.

- Ensure that all strain gauges at the mid-span near the bottom flange are working.
- Similarly, ensure that strain gauges in bottom flanges of the middle two girders near their supports are working.
- Through a series of control tests, determine the representative value of constant C in the gauge in the second internal girder near the north support for vehicles traveling from north to south.
- Similarly to above, determine C for the second internal girder near the south support for vehicles traveling from south to north.
- During control tests determine the values of distribution factors (DFs) for longitudinal strains in all four girders due to at least five transverse vehicle positions.
- Store five sets of DFs obtained above.
- Record readings from all of the above 8 guages at a sampling rate of 250 readings per sec.
- Filter the readings from gauges on the middle two girders near their supports to get rid of noise and dynamic magnification, and divide data into ‘vehicle zones’ and ‘non-vehicle zones.’
- Reject data in ‘non-vehicle zones.’
- Corresponding to maximum strains in the middle two girders near their supports, calculate the distribution factors for longitudinal strains in all girders at mid-span.
- Utilizing the stored sets of DFs and the scheme presented in the paper, determine the nearest transverse position of each vehicle.
- Reject data if the vehicle is not in the normal traveling position.
- The filtered strains near girder supports plotted over time should have distinct peaks. Using corresponding peaks for strains near the two supports, determine the speed of the vehicle.
- Use the speed of the vehicle and the time difference between successive peaks of the same strain-time plot, to determine the distance between axles or between centres of closely spaced axles.
- Using simple engineering mechanics for influence lines for moments and recorded strain-time plots, determine weights of axles or closely spaced axles.

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Comparing conventional and innovative bridge deck options: A life cycle engineering and costing approach

K.J. Kostuk
Federated Co-operatives Limited, Saskatoon, Saskatchewan, Canada

G.A. Sparks
University of Saskatchewan, Saskatoon, Saskatchewan, Canada

G. Tadros
SPECO Engineering, Calgary, Alberta, Canada

ABSTRACT: North America is facing an infrastructure crisis. To illustrate, the American Society of Civil Engineers (ASCE) estimates over the next five years there will be a $1.7 trillion investment need for U.S. infrastructure. In Canada there are 60,000 bridges, 30,000 with concrete decks. The majority of these bridges were built between 1950 and 1975 resulting in a significant cohort between 30 and 55 years of age. Typically Canadian agencies focus their resources on new construction and minimal attention is paid to maintenance and rehabilitation. As a result, most of these bridges require some form of rehabilitation and roughly 14% require major rehabilitation or replacement now. Over the next 10 years, 46% are projected to need rehabilitation. Estimates for these repair and rehabilitation costs range from $10 to $25 billion. The question that arises from these statistics is how can this problem be addressed and can it be prevented from occurring again?

Engineers solve problems. Innovation arises when engineers are pressed to overcome extreme conditions or lack of resources. Although innovative materials such as GFRPs exist to help address the rate of deterioration, higher initial costs and uncertainty about their long term performance have to date precluded their consideration during the design process. Infrastructure agencies must begin to consider life cycle costs. By considering the total cost of ownership (initial construction, operating and maintenance, and end of life disposal) agencies can start making value based, instead of price based, decisions.

A difficulty with state-of-the-practice life cycle costing is its inability to address uncertainty. Life cycle costing by definition addresses long term costs, but future costs are a function of performance and future performance is uncertain. Decision analysis, which evolved in part from the field of statistical decision theory, is capable of accommodating this uncertainty. By integrating life cycle costing, the principles of decision analysis, and the traditional engineering design process we have developed an approach promoting the consideration of new materials and designs which will assist in the development of sustainable infrastructure. This methodology has been labeled life cycle engineering and costing.

To demonstrate the methodology, four preliminary bridge deck designs were compared on a life cycle costing basis.

The basic bridge specifications were as follows: the setting was an urban area; the traffic load was 20,000 vehicles per day; the bridge had 2 lanes, 7 spans of 43.5 m each for a total length of 304.5 m; continuous over 6 piers, with a deck width of 11.64 m, and roadway width of 10.4 m.

The costing assumptions were as follows: the planning horizon is 100 years; the discount rate is 5% (real); and, agency costs include the initial construction, maintenance, and decommissioning. User costs are typically a major issue, but were not included as part of this analysis.

The common design consideration was the use of high performance concrete for the slab. The first design alternative was a traditional design with epoxy coated steel for top and bottom reinforcement.

The slab was 225 mm thick, covered with a water proofing membrane to reduce chloride penetration, which in turn was covered with a 90 mm lift of asphalt. The second alternative was similar to the first but the epoxy coated steel was replaced with MMFX-2. The third alternative utilized a steel free deck design. In this design, the load carrying steel is placed external to the deck. In the bottom of the deck, GFRP reinforcement is used for crack control, and over the piers, epoxy coated steel is used for moment resistance. The fourth alternative replaces the asphalt wearing surface with high density concrete. The epoxy coated steel over the pier was replaced by GFRPs. It should be noted that because of the lack of materials that corrode in alternative 4, we are not concerned with penetration of chloride ions in the deck slab for this alternative, unlike that for the other three alternatives.

Each alternative had common characteristics and parameters but costs varied with design. Typical life cycle costing compares the costs of each alternative based on a nominal costs. To accommodate uncertainty, three values for each parameter, across all alternatives, was collected (low, nominal, and high) with the low-high range representing 95% confidence intervals for each parameter estimate. The life cycle costs for the nominal values acted as a base case, and then deterministic sensitivity was performed. One-by-one, each parameter was varied from the low to high parameter values and the corresponding life cycle costs were recorded. The variation in parameters impacting life cycle costs the most were identified and probability distributions were assigned. The other parameters were fixed at their nominal values. Based on these probability distributions, risk profiles were generated. The risk profiles are the result of calculating the life cycle costs associated with all combinations of parameters and calculating the associated joint probabilities (the product of the associated probabilities). Plotting the probabilities allows one to compare probability distribution for life cycle costs associated with each alternative, providing the opportunity to make an informed decision based both on expected value and risk.

The analysis determined that the steel free deck with GFRP reinforcement stochastically dominated the other alternatives. In other words, this alternative had the lowest life cycle cost at all levels of probability.

Life cycle engineering and costing is a method founded on the principles of engineering design, statistical decision theory, and economics. By integrating the methods of each field into a holistic approach, engineering design can be taken to a new level where both performance and value can be addressed concurrently. The concepts and methods introduced in this paper were illustrated by comparing four preliminary design alternatives for an urban bridge deck. The four designs were relatively similar; the key differences being the reinforcing materials. The analysis found that a GFRP reinforced deck slab with an external steel strap was the design with the lowest life cycle cost (approximately 15% less expensive than the next best alternative based on life cycle costs). In turn this indicates it would provide the best long term value. In summary, by integrating a probabilistically based life cycle costing method with the traditional iterative design process, engineers are better able to develop sustainable infrastructure.

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ISIS Canada educational modules on fibre reinforced polymers and structural health monitoring

Luke A. Bisby
Department of Civil Engineering, Queen's University, Kingston, Ontario, Canada

ABSTRACT: The next generation of engineers, faced with rapidly deteriorating infrastructure, is destined to face many difficult decisions in maintaining and preserving our civil infrastructure systems. The Intelligent Sensing for Innovative Structures Research Network of the Canadian Network of Centres of Excellence (ISIS Canada) has, for more than ten years, been a recognized world leader in the development, advancement, and application of fibre reinforced polymers (FRPs) and structural health monitoring (SHM) in infrastructure applications including bridges. These technologies are now recognized as important tools for bridge construction and repair. However, ISIS Canada recognizes that these important technologies will not become common place until the broader civil engineering community is made aware of their benefits and applications. Clearly, a key group that will play a role in shaping the future of bridge maintenance, safety, and management, is the next generation of civil engineers and engineering technologists. As such, ISIS Canada has developed a series of Educational Modules on FRP and SHM technologies for use in undergraduate engineering and technical college curricula. ISIS' goal in producing these modules is to enable and encourage the teaching of ISIS technologies in educational curricula where they are not currently covered. The modules have been developed to allow seamless integration into various existing courses with minimal effort on the part of the instructor. Teaching resources consisting of lecture notes, presentations, worked examples, case studies, and sample assignments and laboratories have been developed and are now available. This non-research paper presents a brief overview of the educational modules that have been developed by ISIS Canada and which are now freely available for use by all interested parties. The philosophy, objectives, and content of the toolkits are presented, along with examples and illustrations, in the hope that these materials will be taken up and used by the civil engineering education community.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Performance of concrete bridge deck slabs reinforced with glass FRP composite reinforcing bars

A. El-Ragaby, S. El-Gamal, E. El-Salakawy & B. Benmokrane
Department of Civil Eng., University of Sherbrooke, Sherbrooke, Quebec, Canada

ABSTRACT: This paper presents the construction details and field testing results of six innovative FRP reinforced concrete bridges recently constructed in North America. Five bridges, Wotton, Magog, Cookshire, Val-Alain, and Melbourne Bridges are located in Quebec, Canada, while the sixth one, Morristown Bridge, is located in Vermont, USA. All six bridges are girder-type with main girders made of either steel or prestressed concrete. The main girders are supported over spans ranging from 26.2 to 50.0 m. The deck is a 200 to 230 mm thickness concrete slab continuous over spans of 2.30 to 3.15 m. Different methods to design the deck slabs of the six bridges were used. As a result, different types of glass and carbon FRP reinforcing bars and conventional steel were used as reinforcement for the concrete deck slab. The six bridges are located on different highway categories, which mean different traffic volume and environmental conditions. The bridges are well instrumented at critical locations for internal temperature and strain data collection using fibre optic sensors. These gauges are used to monitor the deck behaviour from the time of construction to several years after the completion of construction. Except Melbourne Bridge, all bridges were tested for service performance using calibrated truckloads running in different paths that produce maximum strains in both of the concrete and reinforcement. Both static and dynamic loading, while the trucks travels at different speeds, were carried out. The construction procedure, field tests and monitoring results, under real service conditions, showed very competitive performance to concrete bridges reinforced with steel. The FRP-reinforced bridge decks are very well performing under very harsh environment (de-icing salts, freeze/thaw cycles, elevated temperature, and heavy traffic). No additional or propagation of cracks, if any, were observed under these severe service conditions. The serviceability performance of the concrete deck slab reinforced with FRP bars in terms of strain, cracking, and deflection was very similar to that reinforced with steel bars. During the entire tests, the maximum tensile strain in FRP bars was less than 0.5% of the ultimate strain of the material. For bridges with either concrete or steel girders, the concrete deck slab reinforced with FRP bars can distribute the truck loads over girders approximately the same as concrete deck slab reinforced with steel bars. Finite element modeling and analysis were carried out to investigate the effect of using a reduced amount of reinforcement on the service and ultimate behaviour of the bridges. The FEM shows that the proposed reinforcement ratios adopted by the updated version of Section 16 of CHBDC are adequate for satisfying the serviceability and the strength criteria.

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Fatigue and static investigation of innovative steel free bridge decks

C. Klowak
ISIS Canada Research Network, University of Manitoba, Winnipeg, Canada

A. Memon
Wardrop Engineering, Saskatoon, Canada

A. Mufti
ISIS Canada Research Network, University of Manitoba, Winnipeg, Canada

ABSTRACT: Highway systems and their bridges have been adversely affected by age and weathering over the past two decades. The majority of highway bridges have reinforced concrete decks supported on steel or concrete girders. Rainwater and the application of de-icing chemicals to roadway surfaces during the winter months have penetrated through many concrete decks and caused corrosion of the reinforcing steel.

This paper describes the static and fatigue behavior of two different cast-in-place second generation steel-free bridge decks. Although cast monolithically, the first bridge deck was divided into three segments. One segment was reinforced according to conventional design with steel reinforcement. The other two segments were reinforced internally with a carbon fibre reinforced polymer (CFRP) crack control grid and a glass fibre reinforced polymer (GFRP) crack control grid, respectively, and externally with steel straps. The hybrid CFRP/GFRP and steel strap design is called a second generation steel-free concrete bridge deck. A performance comparison of all three segments for the first bridge deck under 25-ton and 60-ton cyclic loads is reported in this paper.

Test results show that for a 25-ton or 222 kN cyclic loading, all three segments of the bridge deck completed 1,000,000 cycles without significant damage. All three segments of the bridge deck were then subjected to a load of 60 tons or 588 kN. All three segments of the bridge deck failed in punching shear. It was found that the segment reinforced internally with GFRP and externally with steel straps had the best fatigue resistance. The segment conventionally reinforced with steel failed approximately twenty times as fast as the GFRP/steel-free hybrid section. The CFRP/steel-free hybrid section of the deck failed approximately twice as fast as the section internally reinforced with GFRP.

The paper also outlines the details of a larger second generation steel-free bridge deck and briefly outlines some of the preliminary results from the internal panel and cantilever static tests.

The internal panel failed in punching shear, a failure very typical to steel-free bridge decks. The results show that cantilevers may not behave as expected. If we assume that the wheel load is distributed at a 45° angle from the loading plate to the center line of the girder, and compute the moment resistance based on strains in the top transverse reinforcing bars, we can calculate an ultimate load approximately only two thirds of the ultimate load determined from static testing. Preliminary results indicate that there must be some arching action taking place in the cantilevers. However, researchers at the University of Manitoba are currently working with a finite element program developed for steel-free bridge decks as well as commercially available finite element software to gain a better understanding into the mode of failure of the three cantilever sections.

All three cantilevers exhibited a punching type failure similar to that of an internal panel steel-free bridge deck. The shape of the punch cone was approximately a half of a circle compared to a full circle punch cone typical to an internal panel.

A bridge deck containing an internal CFRP or GFRP crack control grid and external steel straps prevents the growth of longitudinal crack widths and eliminates corrosion completely. Experimental results suggest that the area of CFRP or GFRP can be reduced. The second generation steel-free bridge deck with external steel straps and internal GFRP reinforcement for crack control provides the best fatigue performance and is an efficient, economical and corrosion free system for bridge deck superstructures. Flexural theory predicts an ultimate load of approximately two thirds of the ultimate load observed, indicating that there is some arching action taking place in the cantilevers. Researchers at the University of Manitoba are currently looking at finite element methods to gain a better understanding into the mode of failure of the different cantilever sections. The static behavior of a bridge deck plays an important role in understanding the fatigue theory developed from the series of fatigue tests outlined in this paper. The deck will also be subjected to a series of fatigue tests that will help confirm the fatigue theory derived from the first bridge deck.

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Salmon River steel-free bridge deck – 10 year review of field performance

J. Newhook & J. Gaudet
Civil Engineering, Dalhousie University, ISIS Canada, Halifax, Canada

ABSTRACT: An innovative concrete bridge deck design for slab on girder bridges was developed in Canada based on research dating back to the 1980s. The design philosophy enabled the concrete deck slab to be constructed entirely devoid of internal reinforcement. The rationale was to eliminate the principal source of deterioration in concrete bridge decks in North America, corrosion of steel reinforcement. This corrosion and the subsequent concrete deterioration is responsible for dramatically increased maintenance costs and reduced service life. The 'steel-free' deck construction promises to be more durable than traditional steel-reinforced concrete decks and appears to be able to meet the 100 year design life requirement recently adopted by the Canadian Highway Bridge Design Code.

The first application of this system was in the Salmon River Bridge in Canada constructed in 1995. This structure is a highway bridge which experiences regular heavy truck loads and to date has been subjected to 10 winter seasons of freezing and thawing cycles and heavy deicing salt usage. The structure was instrumented and monitored during its first few years of service but has only seen periodic visual inspection in recent years. In 2005, the instrumentation and monitoring program was re-instated. Being the first field application of this new innovation anywhere in the world, its long term service performance is of key interest to many researchers and engineers.

The 10 year evaluation of the performance of the Salmon River Bridge was conducted and included both visual inspection as well as an evaluation of in-service response to vehicle loads. These results were compared with similar field data collected from the initial field monitoring of this bridge.

The steel-free deck slab evolved out of the philosophy of using arching behaviour as the primary resistance mechanism in the design of bridge decks. This approach necessitated the enhancement of internal arching forces through a clear and quantifiable in-plane restraint system. This restraint is provided in the longitudinal direction by making the deck composite with the supporting girders and in the transverse by the addition of external steel straps. The Salmon River Bridge in particular has two simply-supported spans of approximately 31.2 m each. Each span is identical in geometry except that one span was constructed as a steel-free concrete deck supported by six steel girders with a system of external 12×100 mm steel straps spaced at regular intervals of 1200 mm. The six steel plate girders are approximately 1500 mm deep and spaced 2700 mm apart. The concrete deck is 200 mm thick with a haunch thickness over the top flange of the girders of 100 to 125 mm. The remaining span was constructed with similar geometry proportions but utilized a conventional steel-reinforced concrete deck with top and bottom layers of orthogonal reinforcement grids.

Six months after the bridge was opened to traffic, longitudinal cracks were first observed in the underside of the steel-free span. This led to a monitoring program in which the crack pattern and widths were visually inspected and manually monitored on an on-going basis. In addition, a system of strain gauges was installed on several steel straps and on the web of each girder at the midspan of the bridge. From 1996 through 1998, strains readings were collected for heavy vehicle ambient loading. The structural health monitoring program focused on monitoring crack growth in terms of both crack length and crack width. In addition, the strain readings were used to assess the level of strain in the straps and girders, the vehicle load distribution among girders and the composite behaviour in each girder. During the fall on 2005, the crack and strain monitoring program was repeated.

This crack pattern in the steel-free span has remained largely unchanged in terms of crack length and number of cracks since it was first noted in 1996. Comparison of average and maximum cracks from 1998 to 2005 indicate that only a slight change in crack width has occurred over that period. The 2005 average crack width was 1.0 mm compared with 0.82 mm recorded in 1998. The cracking observed in the Salmon River Bridge is a result of fatigue loading on the deck. Comparison of the cracking behaviour observed in the field with that of extensive laboratory testing revealed that the deck is still in the stable range of its fatigue life. In 2005, it was further observed that underside cracking was now evident on the steel-reinforced span as well.

A comparison of strap and girder strain response for ambient highway truck loadings was also compared and the live load response was found to be a very small fraction of the yield value of the steel. The girder strain readings were then analysis to determine the amount of load sharing and the amount of composite interaction between the deck concrete and the steel plate girders. A comparison of these parameters as measured in 2005 with those of 1998 indicted that the superstructure is responding in a stable and satisfactory condition after 10 years of service life.

The Salmon River Bridge was constructed devoid of any internal reinforcement. Cracking behaviour observed in this first field application led to extensive research on fatigue response of this system and slab on girder bridge deck in general. Although the structural response of the Salmon River Bridge has been found to be acceptable, much debate has taken place over acceptable crack widths for these types of systems. The 2005 observed maximum crack width was 1.55 mm. Design consensus at this time, is that it would be desirable to reduce the magnitude of the crack width in future field structures. The new design provisions of CHDBC will therefore require use of nominal crack control reinforcement in the deck slab. While the Salmon River Bridge now belongs to the first generation of steel-free deck structures, continued monitoring of its long-term service performance is a value tool to increasing the acceptance of this technology.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Experimental modal analysis of a cable-stayed bridge

P. Clemente
ENEA, Casaccia Research Centre, Roma, Italy

A. Manuli & F. Saitta
Dept. of Civil Engineering, University of Messina, Messina, Italy

ABSTRACT: The identification of the dynamic characteristics of civil structures, such as bridges are usually performed by means of forced vibration testing or ambient vibration testing. In the first method the structure is excited by shakers and it is also possible to record free vibrations subsequently an imposed displacement. With ambient vibration testing, the response of structures to actions such as wind, traffic, or seismic micro-tremors, is considered. Better results can be obtained when the ambient unknown action is a broad band signal, which guaranties the excitation of a wide range of frequency components.

In traditional modal analysis the dynamic parameters of the structure are obtained in the frequency domain by Frequency Response Functions (FRF), that are the ratio between output and input Fourier transforms. Obviously, the input must be known. However for very-long and high-rise structures, such as bridges and tall buildings, it is more convenient to measure the natural ambient response to unknown excitations, because testing is cheap, fast, and not interfere with the operation of the structure. Nevertheless, responses are small, and often covered in noise. So the analysis becomes more difficult then a traditional modal analysis.

A large number of methods for the extraction of modal parameters, working some in the time domain, some in the frequency domain, has been proposed. The simplest way to estimate the resonant frequencies of structures, using only output measurements, is the peak-picking method, based on the calculation of the Power Spectral Densities of the recorded signals, through use of Discrete Fourier Transform (DFT). The coherence function computed for two simultaneously recorded output signals has values close to one at the resonant frequencies and phase angle closes to zero. Mode shapes can be obtained from FRFs, evaluated, in the context of ambient testing, as the ratio between the response measured in a point of the structure and the response measured in the reference point. In this method the dynamic response at resonance is assume to be dependent only on one mode; this assumption is almost true when modes are well separated and damping is low. The identification in the frequency domain from ambient vibrations can be also pursued by means of the Frequency Domain Decomposition, which is a technique closely related to the classical approach. In the hypotheses of white noise exciting force and of lightly damped structures, through a singular value decomposition of power spectral density function matrix, we obtain a set of auto power spectral density functions, each of them corresponding to a single degree of freedom system. This technique is very suitable to identify close modes. Furthermore the singular vectors are estimates of the modal shapes.

Another way to estimate modal parameters from ambient vibration testing, always by making the assumption of structure excited by a stationary random white noise (or by a filtered white noise), is to operate in the time domain. In the above mentioned case it has been shown that correlation functions between recorded signals can be expressed as a sum of decaying exponential of complex quantities. It is therefore possible to apply time domain methods using impulse response function as input, such as Polyreference Last Square Complex Exponential (LSCE), Eigensystem Realization Algorithm (ERA), Ibrahim Time Domain (ITD).

In this work the results of the experimental investigations on the Indiano cable stayed bridge are explained together with the numerical elaborations of the picked data.

The Indiano cable-stayed Bridge over the Arno River in Florence was completed and opened to the traffic in 1977. The 189.1 m span girder of the bridge is simply supported by two piers, which are structurally independent of the other parts of the structure. In the central portion of the girder, whose length is 128.1 m, two boxes spaced of 6.0 m compose the cross-section. They are linked one to another, at the upper and lower levels, by means of truss structures. As a result, the beam is characterized by large torsional stiffness. The boxes have a width of 4.0 m and a height variable from 2.6 m to 1.6 m. Cantilever beams start from the boxes to support the external portion of the road. In the zones near the ends the cross-section becomes a three boxes cross-section.

The girder is suspended by six couples of fan-shaped stays, starting at the tops of two steel towers. Cables are regularly spread along the deck and are 3.0 m spaced from the centre line, therefore their contribution in supporting the beam torsion is very low. Six cables constrained to an external gravity anchoring compose each anchor cable.

The pylons have steel box cross-section and a height of about 55.0 m from the ground. They are fully constrained at their foundations, which are founded on large piles, and linked to the cable anchoring by means of a pre-stressed concrete truss, which is supposed to support horizontal component of the tower stress. A footbridge is suspended to the girder.

The experimental set up was composed by eight Kinemetrics SS1 seismometers, an HP3566A signal conditioner and a laptop. The synchronized signals from several seismometers were recorded by the acquisition system and analysed in real time in order to have a first glance at the experimental data.

Based on measured data a preliminary analysis in frequency domain has been performed, in order to determine auto and cross power spectral density functions. With MATLAB routines, Welch periodogram method has been used for this analysis. In order to minimize leakage a Hanning time window function has been used. Several peaks have been observed in the spectra.

The final vertical and torsional mode shapes have been obtained by assembling the results obtained from three tests with different sensors locations; a reference location has been used. In the same way another test has been used to derive lateral mode shapes of the bridge deck. This choice was related to the very few number of instruments used on the bridge deck.

The experimental frequencies and mode shapes have been derived by using a simple program on purpose developed and tested for MATLAB, based on Frequency Domain Decomposition method.

The results have been compared with those obtained by means of a 3D finite element model, developed using SAP2000. Experimentally and analytical mode shapes have been object of comparison for by MAC modal indicator. Thus, six modes were correct identified. In spite of the few measurement locations used, a good agreement between numerical and experimental modes was obtained. This can be viewed as a further confirmation of the validity of FDD method in the identification of modal parameters of civil structures subjected to ambient vibrations and for output-only measures.

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Assessment and NDE of FRP rehabilitation of bridge deck slabs at systems level

K. Ghosh, H. Guan & V.M. Karbhari
Department of Structural Engineering, University of California, San Diego, California, USA

ABSTRACT: Externally bonded Fiber Reinforced Polymer (FRP) composites have been shown through laboratory testing and some field applications to be an efficient alternative to conventional repair/strengthening techniques for deficient deck slabs and girders. Most of the current research has focused on the evaluation of the effectiveness of FRP composite strengthening at component level, namely slabs and girders. However research to date has not focused on the effects of strengthening of individual components on overall structural performance at the systems level and changes in load distribution and load paths caused by such structural modifications. There is also need to evaluate NDE techniques that can be used not only as a tool for periodic inspection and quality assessment but also to quantitatively monitor over time the progression of damage to better predict the remaining capacity and life of the damaged structure.

After review of reinforcement details in some typical RC slab and girder bridge deck systems it was observed that the longitudinal girders will have sufficient reserve capacity at the load where the deck slabs will reach limit state, usually in punching shear. Hence most of the damage at the initial load levels will be concentrated in the slabs and the general tendency will be to strengthen only the deck slabs with FRP composites. However after the slabs are strengthened the bridge deck will be able to withstand higher loads and this will increase the load demands on the longitudinal girders pushing them towards flexural or shear criticality before a significant percentage of the additional capacity of the strengthened deck slabs can be used. The girders will thus require strengthening for utilization of the enhanced capacity of the slabs. The research program involved testing of a slab-girder segment with reinforcement details representative of typical existing bridges, in which the structural components were sequentially strengthened with FRP composites and the progression of damage was monitored through experimental instrumentation and periodic non-destructive inspections using forced vibration based modal testing.

The test specimen was a three-girder two bay bridge deck assembly with c/c distance between girders of 1.68 m (5.5 feet) and longitudinal span of 3.6 m (12′4″). The reinforcement details were representative of typical existing slab-girder bridges. The specimen was tested under monotonically increasing load applied by two actuators, one on each deck slab, and the load was cycled at predetermined levels to check for structural stability and to perform NDE. The test was carried out in three phases. Phase 1 involved loading to initiate cracking in the slabs and causing them to reach 75% of punching shear capacity. This load level was deemed to be representative of deterioration in the deck slab that would warrant strengthening of the slabs with externally bonded FRP composites to prevent further degradation. Phase 2 involved loading to initiate shear criticality in the middle longitudinal girder defined as 75% of yield in the internal steel stirrups. The girder was then designed to be strengthened with externally bonded FRP composite stirrups. Phase 3 involved further loading of the test specimen until the strengthened slabs reached flexural capacity governed by debonding of the composite strips and ultimate failure of the specimen due to local punching shear failure of the deck slabs. The progression of damage in the deck slabs and the girder was assessed using both visual and NDE techniques at beginning and end of each phase as well as at intermediate load levels.

At 400 kN (90 kips), which corresponded to 75% of the predicted punching shear capacity, the slabs had extensive cracking characteristic of punching shear deficiency. The slabs were

strengthened with two composite systems, in the form of site impregnated carbon fabric laminates and prefabricated carbon pultruded strips, to test the comparative effectiveness of the systems. Different spacing for the two systems was used to have equivalent transverse flexural capacity.

After strengthening, there was a reduction in the mid-span slab deflection by 15% and reduction in the strain in the transverse steel below the load area in the slabs by ~25% from the test data obtained at 400 kN (90 kips), indicating effectiveness of the composite strengthening schemes. In Phase 2 the specimen was loaded to 667 kN (150 kips) at which the middle girder was predicted to reach shear criticality. Shear cracks were observed in middle girder near support areas and the strain in the steel stirrups reached ~75% of yield strain. The middle girder was strengthened in shear with 3 layers of CFRP composite U-stirrups along with resin impregnated glass fiber bundles installed through holes drilled at the web-chamfer intersection and splayed between the second and third CFRP layers to anchor the composite stirrups.

In Phase 3 the specimen was loaded to failure of the strengthened slabs predicted to occur at 934 kN (210 kips). In both the slabs the failure was initiated by debonding of the composite strips at the locations of the primary punching shear cracks and was followed by punching shear failure around 934 kN (210 kips), as predicted. With the strengthening of the slabs and the girder, the damage was localized at the slab-girder intersection region resulting in the formation of cracks running on top of the slab at the negative moment area near the slab-girder intersection area. Through thickness cracks running through the slab were also visible in this region at the outer edge of the specimen.

Experimental monitoring of the damage progression in the deck slabs was carried out through instrumentation of the specimen with linear potentiometers and strain gages. Based on the material properties of concrete, steel reinforcement and FRP composite, as obtained through material testing, and the geometry details, the moment curvature response of the deck slabs in the primary direction of load transfer between the longitudinal girders was computed following the sectional analysis methodology. From the strain gage data in the internal steel reinforcement and external FRP composite, the moment capacities of the slab section were then computed over the load stages and the corresponding curvatures were obtained from the previously computed moment-curvature response. The trend of "effective" flexural slab stiffness at a particular location over the load stages were then computed from the ratio of the moment and curvatures.

Non-destructive inspections using forced excitation based modal testing were carried out both at the beginning and end of each phase as well as at intermediate load levels using three excitation sources. The natural frequencies of the test specimen were obtained from the Frequency Response Function (FRF) plots and the frequency ratio for the primary identified modes, defined as the ratio of the frequency obtained at a particular load stage to the baseline frequency of the structure before loading, over the load stages were obtained.

Thus the test results demonstrated that the deck slabs with reinforcement representative of typical existing bridge decks were susceptible to punching shear failure under field-representative wheel loads. Also it was evident that strengthening of individual components can cause other components to reach their limit state under higher load demands and prevent the strengthened component to reach ultimate capacity. Thus it can be concluded that to utilize the efficacy of the FRP strengthening, the design should consider the overall structural response at system level rather than treating it as a patch repair technique. Both the pultruded strips and fabric laminates were effective in strength enhancement and the predictions matched experimental results closely. The degradation of slab stiffness was computed from the moment curvature response and the strain gage data. The strengthening of the slabs with composite was found to result in a stiffness enhancement by about 15%. A NDE methodology using forced vibration based modal testing was incorporated for monitoring the progression of damage in the strengthened and un-strengthened components. The trend of the changes in the natural frequency of the structure matched well with the degradation/enhancement of structural stiffness caused by the progression of damage/composite strengthening. Research work is ongoing to use the NDE results for prediction of in-service capacity of the structure for the current state of damage.

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Innovative seismic design of bridges of the South Carolina Department of Transportation (SCDOT)

L.E. Mesa
South Carolina Department of Transportation, South Carolina, USA

ABSTRACT: Based on the magnitude 7.3 M Charleston earthquake of 1886, the discovery of several sand boils along the coastal plains, and the development of the State Seismic Hazard Maps, South Carolina is preparing for the next major earthquake. South Carolina is the second state in the United States to develop and adopt its own seismic design specifications for highway bridges. The South Carolina Department of Transportation (SCDOT) has very strong and visionary leadership, and is always looking for state of the art technology to support its mission of providing a safe transportation infrastructure for the people of South Carolina.

In the early 1990's SCDOT identified the need for new seismic design specifications. The AASHTO Division–IA Seismic Design Specifications did not specifically address the geologic conditions in South Carolina. AASHTO Division–IA used the 1988 seismic hazard maps, developed by the United States Geological Survey (USGS).

By that time, after the Loma Prieta (1989) and Northridge (1994) earthquakes in California, California Transportation (CALTRANS) made a contract with researchers to produce the ATC-32 guidelines, recommending state of the art practice.

It was in this document where performance based design guidelines were first introduced. Damage and serviceability levels of performance were defined. Ductility factors, new site class, plastic hinge location and length, no-splicing zone in columns, longitudinal reinforcing, and transverse reinforcing were also recommended.

The SCDOT decided to improve its seismic design practice, developed and adopted a "SCDOT Draft Seismic Specification for Highway Bridges" which used the 1996 USGS seismic hazard maps. The SCDOT incorporated some of the ATC-32 recommendations and adopted the 1996 USGS seismic hazard maps hoping this would be a much better approach than using the AASHTO Division-IA specifications. Soon, the SCDOT identified the need to have a new fully developed specifications not only capturing the performance based design, but we also needed the State Seismic Hazard maps to complement the design in a more consistent manner.

The SCDOT "Seismic Design Specifications for Bridges and Highways" were developed and adopted in 2001. New bridges in the state are seismically designed using these specifications. No-collapse is the minimum requirement for the 2% in 50 years seismic event that was adopted for design. Performance based design procedures, and requirements for a more reliable reinforcing steel (A-706), butt-welded hoops in columns, and ultimate mechanical couplers to splice column main reinforcing were included in the Specifications. The Specifications originally used the 1996 USGS seismic hazard values.

USGS (1996, and 2002 update) maps did not reflect the actual geological conditions in South Carolina.

It was learned then, that California and Oregon on the United States West Coast have developed their own state seismic hazard maps to be used in bridge design. The SCDOT followed, and by the end of 2001, the agency adopted its own South Carolina Seismic Hazard Maps to be used on the seismic design of bridges.

Several issues were remarkably incorporated into the development of the maps. Two conditions were identified to simplify results: Atlantic Coastal Plain, and sites outside the Coastal Plain. For

the Coastal Plain sites ground motions were mapped for a hypothetical outcrop of "firmcoastal plain sediment" (NEHRP B-C Boundary), with a shear wave velocity of 760 m/s. The motions serve as the acceleration map values that will be modified by site factors to produce the acceleration design values. With those values an Acceleration Response Spectra (ARS) curve is created, and used as input for the dynamic analysis.

The motions mapped outside the Coastal Plain region were interpreted as surface motions on "weathered southeastern U.S. Piedmont rock". This is very different from "weathered rock" in California. Shear wave velocity for this region is 2500 m/s, overlying a hard rock basement.

The map results are provided by SCDOT to designers, to develop the ARS curves for design, according to the specifications. Deaggregation graphics for liquefaction potential prediction, and peak ground accelerations are provided. These are a great tool to the geotechnical engineers.

The specifications and the map values have been adopted and used in design. Close overview of the implementation process has shown the SCDOT that there was a need for updating the specifications and to have a more user-friendly seismic hazard maps program.

Because of the performance-based design of bridges, we have been able to categorize bridges according to the importance on the transportation network. Retrofit will have a big role on upgrading newer bridges to the new required standards for South Carolina.

Seismic Design of Highway Bridges is part of the Federal Highway Administration (FHWA)/ SCDOT strategic planning, where a complete alternate routing system for critical infrastructure has to be identified by December of 2006. The development of contingency plans to restore traffic and emergency plans to repair/replace critical infrastructure following a catastrophic event has to be finished by December of this year.

The SCDOT is also supporting the FHWA Long Term Performance of Bridges Program, and is studying the possibility to instrument the Cooper River Bridge that is a Cable Stay Bridge, the longest span in North America. Structural Health Monitoring is a new field in bridge engineering that SCDOT is very interested in. All these state-of-the-art developments in civil engineering will allow a better understanding of the structural behavior, including ground motion induced loads.

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Operational modal analysis for long-term bridge performance monitoring

Hong Guan & Vistasp M. Karbhari
University of California, San Diego, California, U.S.A

Charles S. Sikorsky
California Department of Transportation, Sacramento, California, U.S.A

ABSTRACT: The measurement of structural dynamic properties such as modal parameters is an essential step in vibration-based structural health monitoring, which is based on the premise that changes of structural properties, such as mass, stiffness and damping, will affect the behavior of the structure during vibration. So by studying the changes in measured structural vibration behavior and in essence solving an inverse problem, the unknown changes of structural properties can be identified.

Modal analysis has been widely used for the task of extracting structural modal parameters from vibration measurement. Traditional Experimental Modal Analysis (EMA) makes use of measured input excitation as well as output response. For large civil structures, it is typically very difficult to excite the structure using controlled input. In the last decade and a half or so, Operational Modal Analysis (OMA) has drawn great attention in the civil engineering field as an attractive way to tackle this problem. Compared with traditional EMA, OMA does not require expensive excitation sources and can be applied to structures while they are in operation. The latter attribute is particularly attractive for vibration-based bridge health monitoring application because the target bridge does not to be closed to perform the modal parameter identification.

Recently a vibration-based bridge health monitoring system was installed on a FRP composite highway bridge in California. One aspect of the project that is relevant to the current discussion is the use of OMA to identify modal parameters under ambient traffic excitations. Due to the relatively short span of the bridge, traffic excitation to the bridge exhibited strong non-stationary feature. To the authors' knowledge, most of the OMA techniques existing to date rely explicitly or implicitly on the assumption of stationary input. Thus it appeared to the authors that there was a need to evaluate the applicability and accuracy of OMA technique under unknown non-stationary inputs.

An improved Operational Modal Analysis (OMA) technique called NExT-RFP is proposed in this paper. The theoretical development of NExT-RFP technique is first discussed. By extending the traditional displacement response based time-domain Natural Excitation Technique to frequency domain and acceleration response based, a unique approach to the OMA problem is provided. The laboratory experiment that was used to evaluate the two OMA techniques: NExT-RFP and FDD, is then described. The results showed the superior performance of the proposed method under non-stationary ambient excitations. Further research in underway to integrate the NExT-RFP technique into a vibration-based bridge health monitoring system.

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Development of a field useable interrogation system for RF cavity wireless sensors

A. Hladio, R. Jayas, D.J. Thomson & G.E. Bridges
Department of Electrical and Computer Engineering, University of Manitoba, Winnipeg, Canada

1 INTRODUCTION

Existing sensing technologies for civil structural health monitoring have a serious deficiency in that they require some type of permanent connection to the outside world. This causes significant problems in the installation and ongoing use of these sensors. Several recent cases have shown that the cost of installation and preparation of site for monitoring equipment can easily equal the cost of sensors and interrogation equipment. Therefore, not all sites will justify the cost of installation and maintenance of a permanent site. Wireless technology could make the monitoring of many more sites feasible and therefore significantly increase the utility of SHM for civil structural health monitoring.

We have developed a new type of wireless strain that requires no battery or electrical power. The strain sensor is a passive device that can be easily embedded in concrete civil structures. In this presentation we will be discussing some recent advances in the development of a field usable interrogation system for this sensor.

The strain sensor is a metal cavity that changes dimension in response to an applied force. A short length of wire from the inside to the outside couples RF signals from the cavity inside to an antenna. The sensor can then be interrogated via the antenna. Such a system has the advantage of requiring no permanent physical connection between the sensor and the data acquisition system. Changes in the structure's dimensions will be reflected in changes in the resonant frequency, which is then used to calculate the strain on the structure.

2 FIELD USABLE INTERROGATION SYSTEM

Using a sensor that has been embedded in concrete test cylinders interrogated using an external antenna we have demonstrated a strain resolution of better than a few microstrain with a bandwidth

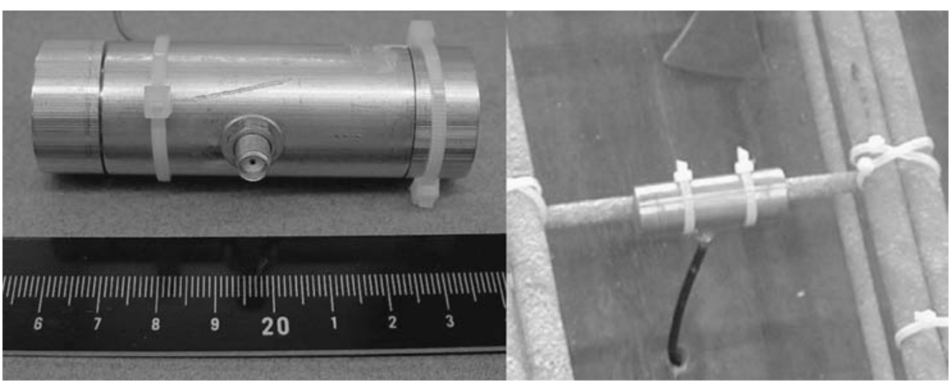

Figure 1. Photograph of sensor and of the sensor installed on a reinforcing bar.

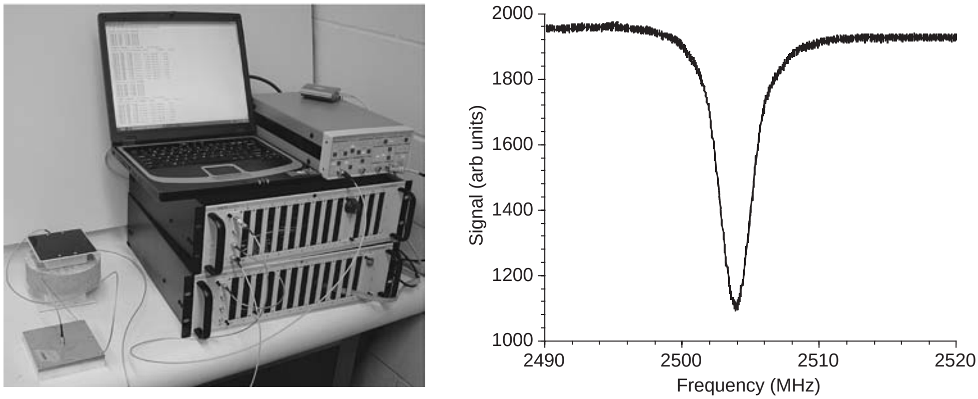

Figure 2. Photograph of interrogation system and measurement through 4.5 cm of concrete.

of 30 Hz [Chuang 2005]. However, those results were taken with a large and heavy apparatus that is unsuitable for field applications. In this work we report on results obtained with a portable system that is suitable for field applications. The system is constructed with simple "off the shelf" components and a lap top computer. In figure 2 is a photograph of the portable interrogation system and a typical result measured through 4.5 cm of concrete. Using measurements of this type the resonant frequency of the cavity can be estimated. The strain can then be estimated using the strain to frequency conversion factor, which is 2.5 kHz/$\mu\varepsilon$ at the frequencies used in this study.

The repeatable measurement of the resonant frequency is critical to this sensing technique. Several techniques for determining the resonant peak position were explored. Maximun value, peak fitting, matched filter, centroid analysis and a new servo technique were all implemented. The signals were taken with 4.5 cm of concrete between the interrogation antenna and the antenna attached to the sensor. The repeatability of these techniques ranged from 24 $\mu\varepsilon$ using the maximum value to 2 $\mu\varepsilon$ using a centroid analysis. However, there were systematic differences between the center peaks determined by the various techniques. The center peak determined by the maximum signal technique and the center peak determined by the match filter technique were separated by 140 kHz, which corresponds to a strain difference of 56 $\mu\varepsilon$.

REFERENCE

Chuang, J, Thomson D.J. and Bridges G.E., 2005, "Embeddable wireless strain sensor based on resonant rf cavities, Review of Scientific Instruments, Vol 76, Iss. 9, Art No. 094703

Repair and strengthening

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)
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Planning and working of overall recoating for long-span bridges

I. Yamada
Long-span Bridge Engineering Center, Honshu-Shikoku Bridge Expressway Co., Ltd. (the former Honshu-Shikoku Bridge Authority), Japan

K. Sumi
Naruto Operation Center, Honshu-Shikoku Bridge Expressway Co., Ltd. (the former Honshu-Shikoku Bridge Authority), Japan

ABSTRACT: The Ohnaruto Bridge (Figure 1) is a suspension bridge with the main span of 876 meters, completed in 1985. Due to the severe corrosive condition, the overall recoating for the truss-type stiffening girders and the truss-type main towers was carried out from 1998 to 2005. Especially, the automatic coating method was developed and applied to the main towers of the Ohnaruto Bridge for the first time in the existing long-span bridges. This paper describes the planning and working of overall recoating for the stiffening girders and the main towers of the Ohnaruto Bridge, considering the surrounding natural condition, the working ability/safety and the coating quality/efficiency.

The original coating system for the Ohnaruto Bridge has the following features; 1) application of the high build type zinc rich paint to the surface treatment, 2) application of the polyurethane resin paint to the finish coat. Recently, the fluoro resin paint with a higher weather resistance was developed and applied to original coating systems for new bridges, including the Akashi Kaikyo Bridge, as well as recoating systems for existing bridges, including the Ohnaruto Bridge.

The basic policy of coating system for the Honshu-Shikoku Bridges is the "preventive maintenance," which requires the recoating work to be completed before disappearance of the intermediate coat (Figure 2). Because the epoxy resin paint for the intermediate coat and the under coat has a very high aging speed, the high build type zinc rich paint for the surface treatment might be damaged if the preventive maintenance is not carried out. In the Honshu-Shikoku Bridges, the preventive maintenance of coating system enables to enhance the durability of steel bridges and to reduce the maintenance cost, considering the life cycle cost.

In the stiffening girder, coating engineers repainted diagonal members in lateral trusses and longitudinal trusses by using brushes and newly improved scaffoldings. The improved scaffolding enabled to reduce the weight and the cost, ensuring the required workability. The coating engineers repainted other members by using brushes and outside inspection vehicles or inside inspection

Figure 1. The Ohnaruto Bridge, completed in 1985 and repainted in 1998–2005.

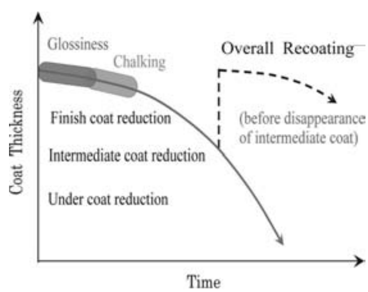

Figure 2. Aging process of coating system and overall recoating for preventive maintenance.

vehicles. On the other hand, the coating engineers repainted steel deck plates by using brushes and suspended scaffoldings.

The maintenance guideline of recoating work for the Honshu-Shikoku Bridges was revised in 2002. The following new inspection descriptions were added to the existing inspection descriptions in the supervisor inspection; 1) saline element, 2) cleaning, 3) coat thickness, 4) adhesion resistance, etc. Especially, the coat thickness should be measured in the wet coat and in the dry coat. The wet coat thickness and the dry coat thickness were measured by means of the wet coat gauge and the dry coat device, respectively.

The dry coat device has an excellent accuracy although it damages the coating a little. The dry coat thickness should be measured once per recoating areas of $500\,m^2$ by the dry coat device. The measurement results clarified that the finish coat had the specified thickness of more than 25 micrometers and the intermediate coat had the specified thickness of more than 30 micrometers, although they had small variations in measurement points.

In general, gondolas are used as scaffoldings in the maintenance of towers or architectures. The ordinary gondolas, however, cannot be used in the Ohnaruto Bridge, because strong winds usually occur around the year in the Naruto Strait. Therefore, the “magnetic wheeled gondola” was developed and applied to the main towers of the Ohnaruto Bridge.

In addition, an automatic recoating was carried out in the tower shafts by the “automatic coating machine” loaded on the magnetic wheeled gondola (Figure 3). The automatic recoating works by the automatic coating machine were applied to the slightest-class cleaning, the intermediate coat of 30 micrometers and the finish coat of 25 micrometers.

The quality control method in the main tower is the same as that in the stiffening girder. According to the measurement results by the dry coat device, the finish coat and the intermediate coat by the automatic coating method have smaller variations than those by the manual coating method. At the main towers of the Ohnaruto Bridge with corner-cut cross sections, the coating efficiency of automatic recoating is 7 times as high as that of manual recoating in the surface treatment, 4 times in the intermediate coating, and 4 times in the finish coating (Figure 4).

The conclusions of the overall recoating in the Ohnaruto Bridge are summarized as follows:

1. Based on the quantitative data of coating investigation and coat deteriorating curve, the overall recoating for the stiffening girders and the main towers was planned and carried out as the preventive maintenance.
2. The overall recoating for the stiffening girders was carried out from 1998 to 2004. The dry coat device was used in order to control the coat thickness, and the improved scaffolding was used for diagonal members in order to reduce the weight and the cost.
3. The overall recoating for the main towers was carried out from 2004 to 2005. The dry coat device was used to control the coat thickness, as same as the stiffening girders. Especially, in the tower shafts, the magnetic wheeled gondola and the automatic coating machine were used in order to improve the working ability/safety and to enhance the coating quality/efficiency.

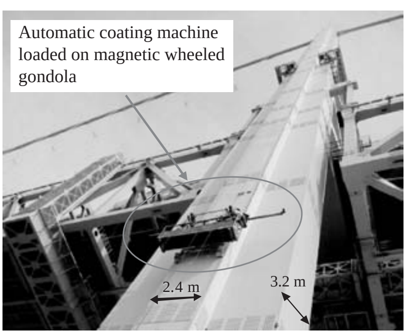

Figure 3. Automatic recoating on tower shaft, using the automatic coating machine.

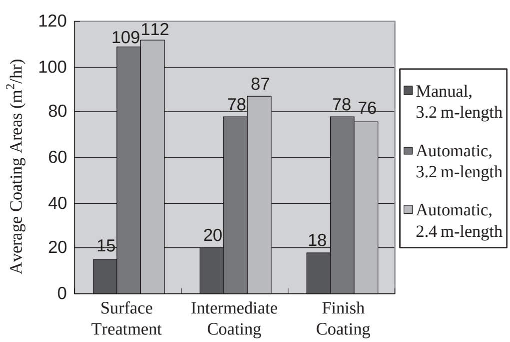

Figure 4. Coating efficiency of automatic/manual recoating at tower shaft of 3P.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Experimental research on the prestressed concrete main beams of road bridge strengthened by CFRP tapes under static loads at different repair stages

A.G. Mordak
Scientific-Research Center for the Development of Bridge Industry MOSTAR, Wroclaw, Poland

Z. Manko
Wroclaw Univ. of Technology, Wroclaw, Poland

ABSTRACT: This paper presents the results of the research on a five-span prestressed concrete road bridge under static load performed at different stages of its repair. All bridge spans reinforce were conducted by CFRP tapes glued on bottom flanges of main beams, and external flat steel stirrups, as well as overlaying a new concrete deck layer.

1 INTRODUCTION

This paper presents research on the concrete spans with the prestressed main beams of the road bridge on the Nysa Klodzka river, situated in Paczkow. The research was conducted at different stages of repair works, that is, before performing the main research on static and dynamic field load tests, which was aimed to determine the efficiency of the applied repairing methods. The aim of this paper is to present the program, process and results of the research, static-strength calculations and analysis of obtained results. The aim of conducted research was to determine the behavior of the chosen spans structures subjected to considerable static loads at various indirect phases of the performed bridge repair. The research allowed to find out on which elements of the load-carrying structure of span the biggest forces were exerted during the progress of repair works on one half of the bridge.

2 DESCRIPTION OF THE OBJECT AND ITS STRENGTHENING

The examined bridge was built from 1957 to 1959. It is a concrete five-spans bridge with simply supported spans, rectangular in plan spans, slab-beams structures, post-tensioned in the transverse and longitudinal direction with the load-carrying structure constructed of prefabricated post-tensioned prestressed concrete main beams (prototype "Plonsk") made *in situ*. Particular effective spans are 18.00 m. They were based on the bridge abutment and pier (spans I and V) and on two bridge piers (spans II, III and IV). The strengthening of the particular bridge spans was accomplished by gluing the tapes made of carbon fibers CFRP to the bottom flanges of main beams and constructing an additional reinforced concrete deck plate of variable thickness 0.12–0.185 m and adding the outer stirrups in the form of steel flats of 5×50 mm section, with axial base by every 0.35 m.

3 SPANS RESEARCH RANGE

Experimental research of the chosen spans of the bridge were conducted only for the same single scheme of the static load, that is, for the truck with its middle axle situated at midspan and directed in front of the middle span. During the bridge repairs, the research was conducted at eleven different stages (phases). In the project of the research under the static and dynamic load it was decided that four trucks KAMAZ 5511, with maximum loading capacity up to 101.5 kN, will serve as loading trucks. One of them was used at the particular stages of the research.

4 RESULTS AND ANALYSIS OF EXPERIMENTAL RESEARCH

Stage I. At stage I of the experimental research, the outer span I of the bridge was loaded on closed for traffic half of the bridge by the truck. On this half of the roadway only repair works were performed there at that time. Stage II. At stage II, the upper side of the span structure on the same half of the bridge was in the same technical condition as at stage I but on the bottom flanges of the main beams, CFRP tapes were already glued, which strengthened the load-carrying structure of the span I. The loading truck was placed similarly as at stage I. Measurements in Research Stage III. During the stage III, quite significant changes in spans structure could already be observed in comparison to the two previous stages. They were caused by laying concrete on the new bridge deck plate. Additionally, stirrups made from steel flats with sections of 50×5 mm, in axle base by every 0.35 m, were installed on the main beams. Research Stages IV and V. At the next stages of the research (IV and V) bridge's extreme span I was loaded by setting the load on the half of the roadway, where repairs had been finished (i.e., from the side of the headwater) and later on the non-repaired half, that is, from the side of tailwater, where the old road layer was still not removed. Measurements at Stages VI and VII. At the next stages of the research, span II was loaded by placing the ballast similarly as at stages IV and V, first on the half of the bridge where repairs had already been finished, that is, from the side of the headwater (stage VI), and later on the currently repaired half of the roadway, that is, from the side of the tailwater (stage VII), where the old road layer was still not removed. Measurements at Stage VIII. At stage VIII, the extreme span I was loaded by placing the loading truck on the currently repaired half of the roadway from the side of the tailwater. Repair of the half of the roadway from the side of the headwater had already been accomplished, while half of the bridge loaded at this stage of the research was at the same repair stage as its counter part from the side of the headwater at stage I of the experimental research (surface layers of both roadway and sidewalk were removed together with protective and leveling concrete). Measurements IX. Similarly to stage VIII, at this stage the extreme span V was examined (research stage IX). During research, time courses of deflections and strains of the main beams and strains in the middle strengthening tapes of all six main beams in their midspan were measured. Measurements X. At stage X of the experimental research, essential measurements were made by placing the load on the half of the roadway from the side of tailwater in midspan of span II. The second half of the roadway was used by standard bridge traffic. Repair works were practically finished, but there were preparations in progress for exchanging bearings, because rails set below extreme bridge crossbeams supported bridge's load-carrying structure.

The average deflection values of spans cross-sections were smaller than calculated ones in all cases. For the possible load schemes to execute during the repair works correspond average measured and calculated elastic deflection relations for particular stages were as follows: I – 0.696, II – 0.752, III – 0.895, IV – 0.904, V – 0.925, VI – 0.906, VII – 0.900, VIII – 0.588, IX – 0.702, X – 0.991, XI – 0.969. The measured main beams deflection distributions in span cross-sections were different than calculated ones. It provides that a real torsional stiffness of the bridge was higher than assumed in static calculations.

5 CONCLUSIONS

The span structures with the post-tensioned prestressed concrete main beams strengthened by CFRP tapes made of carbon fibers and outer stirrups in form of steel flat did not aroused any reservations as far as sizes of section forces, displacements and strains obtained from studies and calculations are concerned. Deflection and strain values of the main beams obtained from the measurements did not result in any doubts, as far as the strength and carrying capacity in tested spans and in behavior and interaction of the bridge in eleven repair stages are concerned.

The differences in the expected deflections and strains of main beams in a relation to the obtained from measurements after the repair of one half of bridge span provides that it was well interaction between a new bridge deck layer and pavement layers of roadway. However the applied of strengthening of main beams with CFRP tapes and outer stirrups did not call out the significant changes in deflections and strains values of main beams.

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Strengthening steel beams using bonded carbon-fibre-reinforced-polymers laminates

D. Linghoff, M. Al-Emrani & R. Kliger
Department of Structural Engineering, Chalmers University of Technology, Gothenburg, Sweden

ABSTRACT: In recent years, carbon-fibre-reinforced-polymer (CFRP) laminates have been used in the repair and/or strengthening of existing steel structures. One advantage of using CFRP is its high tensile strength and stiffness compared with its low self-weight. This paper describes the research of strengthening steel structures with CFRP laminates under static conditions and comprises both laboratory tests and simplified analytical methods for predicting the capacity of steel girders strengthened with bonded CFRP laminates.

The analytical solutions consist of calculation of bending capacity, normal stress distribution over the cross-section and interfacial shear stresses. By bonding CFRP laminates to the tensioned flange of a steel girder, it is possible to increase the moment capacity. The forces which are acting over the strengthened section, will cause interfacial stresses in the bond line. These interfacial stresses may cause debonding between the steel substrate and the laminate why these stresses must be considered in the design process. Simplified analyses were performed using Matlab to calculate the difference in stiffness and moment capacity depending on quantity and stiffness of the applied laminates. These solutions are based upon equilibrium statements over the section. The simplified analyses show that it is possible to increase the moment capacity by using bonded CFRP laminates to steel girders to a level of 20%, see Figure 1.

By using developed simplified solution described above, it is also possible to calculate to which extend the steel section has reached yielding. The solution shows for studied cross-section that a maximum increase in terms of the moment capacity is about 20%, and, as a result, nearly the whole cross-section will be in compression. Tension will almost only be transmit by the CFRP laminate. To increase the moment capacity further, strengthening has to be made on the compressed area. Also, interfacial shear stresses in the bond line for the studied beams were calculated using an existing solution for calculation of interfacial stresses for arbitrary substrates.

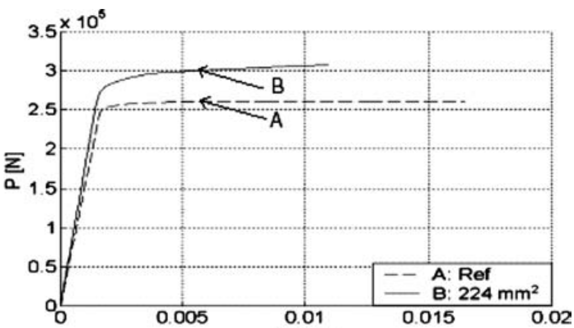

Figure 1. Load (P)–Strain (epsilon) plot for an unstrengthened beam (A) and a strengthened beam (B) with a total area of CFRP equal to 224 mm^2.

With results from the analytical solutions laboratory tests were conducted. The test specimens consist of CFRP laminates and epoxies of different material properties, which were attached to the tensioned flange of undistorted steel beams with H-shaped cross-sections. All beams were sand blasted, cleaned and coated with a primer directly afterward. When the primer had cured, the epoxy was applied to the CFRP laminate and then bonded to the steel substrate by a hard roller. At the ends, the redundant epoxy was tapered shaped to reduce the stresses. The strengthened beams were tested in static four points bending where load-deformation of the beam and strain both in the steel section and the laminate was recorded.

The dimension of the steel beam cross-section were calculated and selected from available sizes of CFRP laminates. In addition, the ratio between height and length was selected to be similar as for ordinary bridge girders. The steel beams had an HEA180 profile and a length of 2000 mm. The material properties of the steel were provided by material tests performed in connection to the bending tests and were 330 MPa and 212 GPa for strength in tension and modulus of elasticity (MOE), respectively. The material properties for the CFRP were in the range of 1500–3300 MPa for the strength in tension and 165–450 GPa for the MOE.

The reason for using different CFRP and epoxy was to see how the choice of stiffness, strength and quantity of the CFRP would affect the strengthened girder in Serviceability Limit State (SLS) and Ultimate Limit State (ULS).

For the tested specimens there were no problem with debonding of the laminate due to high interfacial stresses, except for one beam were the failure was between the primer and steel substrate. This failure has to be studied further to avoid wrong combinations of materials. The major failure modes for the tested beams were rupture of the CFRP laminate due to high stresses. Neither the analytical solutions indicated that failure should appear due to high stresses near the end of the CFRP laminate.

According to the analytical solutions there are opportunities to increase the quantity of CFRP laminate on the tensioned flange and thus increase the capacity of the steel beam further, presumed that the section is able to transmit the compression stresses.

From the laboratory tests, it can be seen that use of a high strength CFRP with steel equivalent stiffness produces the most preferable behaviour of the strengthened steel beam. An increase of stiffness can be obtained by using either a stiffer CFRP or a greater quantity of CFRP. The disadvantage with using CFRP with high modulus is the decrease in strength that causes a more brittle behaviour of the strengthened beam. Additionally, the laboratory test results show that after the strengthened steel beam has started to yield no exponential increase of the shear stress at the end is obtained. On the contrary, a decrease of the shear stress is observed at high load levels.

The aim of this research is to study the behaviour of the strengthened beam in serviceability limit state and ultimate limit state for different configurations of CFRP laminates and epoxies. Additionally, the failure modes and the interfacial shear stresses in the bond line are studied. The objectives with these results are to develop simplified calculation methods for design purpose and achieve suitable methods for repair and/or strengthening of steel structures, regarding both design criteria and practical applications.

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Rehabilitation of fatigue cracks in welded gusset joint using CFRP strips

Hitoshi Nakamura
Tokyo Metropolitan University, Hachioji City, Japan

Hiroyuki Suzuki
Meisei University, Hino City, Japan

Ken-ichi Maeda
Tokyo Metropolitan University, Hachioji, Japan

Takao Irube
TTK-Corporation, Tokyo, Japan

1 INTRODUCTION

In recent years, many fatigue failures have been reported in steel bridges, caused by an increase in traffic volume and heavy vehicles. The fatigue cracks are often initiated at welded joints, and in such locations, the repair workability is poor, hence a convenient and efficient construction technique is demanded in the fieldwork. To combat such problems, a Carbon Fiber Reinforced Polymer strip (henceforth a CFRP strip) is expected to become a new material used to repair and strengthen steel infrastructures.

In this study, the effect of conducting repairs by bonding CFRP strips is examined experimentally for fatigue cracks initiated in out-of-plane welded gusset joints. The effect of the bonding method and the number of layers of CFRP strips on the fatigue life is clarified.

2 EXPERIMENT PROCEDURE

In order to examine the effect of the repair of fatigue cracks, out-of-plane welded gusset joint specimen were fabricated, as shown in Figure 1. The used CFRP strips are made from a unidirectional reinforced material of 1.2 mm thickness. Fatigue tests were carried out under constant amplitude loading. The uniform nominal tensile stress occurred in the specimen.

First, in order to initiate a fatigue crack at the weld toe, cyclic loading was carried out. The fatigue test stopped, when the crack length a reached 15 mm from the center of the specimen on one side.

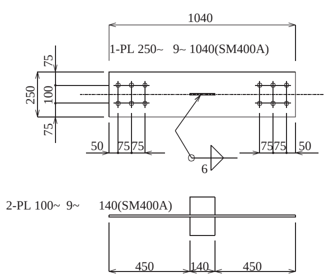
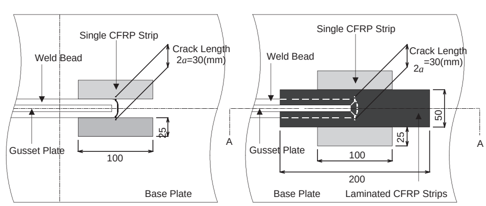


Figure 1. Specimen configuration.



(a) Series S (b) Series M

Figure 2. Bonding method of CFRP strips.

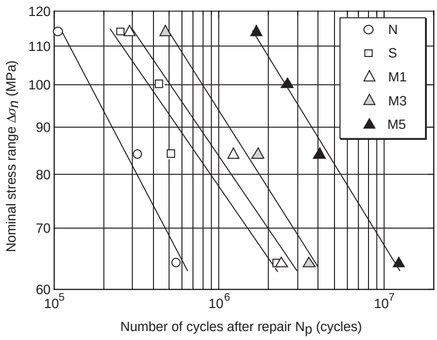


Figure 3. S–N_p diagrams.

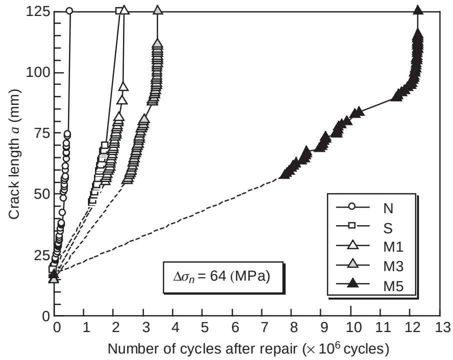


Figure 4. Crack propagation.

Next, the specimen was removed from the loading system, and repair was effected via the following methods. The bonding positions of the CFRP strips in Series S and M are shown in Figure 2.

In Series S, four-single CFRP strips (25 × 1.2 × 100) were placed on both sides of the gusset plate, and the repair was carried out using the epoxy resin adhesive, as shown in Figure 2(a).

In Series M, the CFRP strips (50 × 1.2 × 200) slit in rectangles were very close, with each individual layer bonded to the weld bead using epoxy resin adhesive. The number of laminations varied between 1, 3 and 5 layers. Four-single CFRP strips (25 × 1.2 × 100) were adjacently bonded to both sides.

3 EXPERIMENTAL RESULTS

In Figure 3, an S–N diagram was described using the number of cycles from the restart after repair to collapse, N_p. In Series S of single CFRP strips, a significant improvement in the fatigue life after repair was obtained in comparison with Series N of non-repair, despite the stress range.

In Series M, the fatigue life drastically improved as the layers of laminated CFRP strips increased. In M5, the fatigue life improved remarkably in comparison with Series N, and a sufficient repair effect was obtained. This is attributed to the fact that the crack opening was repaired by the high rigidity of laminated CFRP strips, and the opening displacement was reduced sufficiently. However, in this study, the fatigue limit was not obtained by the repair of M5 within the stress range (64–114 MPa), although the fatigue life greatly exceeds 10 million cycles within the lowest stress range, 64 MPa.

Figure 4 shows the relationship between the crack length and the number of cycles after repair in $\Delta\sigma_n = 64$ (MPa). In Series S and M, the fatigue life after repair was shown to be improved. In particular, crack propagations were delayed in the bonding area of CFRP strips, indicated by the dotted line, in comparison with Series N. In Series M, this tendency was indicated to a remarkable extent, and the sufficiently delayed property of the crack propagation also traversed the bonding area of the CFRP strips when the number of layers of laminated CFRP strips increased.

4 CONCLUSIONS

For fatigue cracks initiated at the weld toe of an out-of-plane gusset joint, the repairs were carried out using CFRP strips and epoxy resin adhesive, and the effect of the bonding method of CFRP strips on the fatigue life was examined experimentally. The following conclusions were obtained:

(1) It was confirmed that as the number of laminations of CFRP strips increases, the post-repair fatigue life improved considerably and that a sufficient effect of repair was acquired in the case of 5 layers.

(2) The laminating of CFRP strips was found to be effective in preventing peeling and sharing axial force.
(3) No fatigue limit was obtained from the target stress range. The proposed method was positioned as a first aid repair to prolong the fatigue life, since no crack recurrence was prevented.

Therefore, it was indicated that the proposed repair method was sufficiently applicable for first aid repair.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Alcácer do Sal Bridge – rehabilitation and strengthening

T.P. Mendonça, M.P. Almeida & P.P. Paulo
Betar Consultores, Lisbon, Portugal

ABSTRACT: The design of the bridge at Alcácer do Sal was based on standards that are no longer applied in Portugal. The structure is extremely complex for the calculation methods available at that time. The owners of the bridge are now giving special importance to the verification of these structures in accordance with current standards and calculation tools. The project of rehabilitation and strengthening developed for this bridge means that it now complies with the required standards of durability and resistance.

The bridge at Alcácer do Sal crosses the River Sado at Km 0 + 050 on the E.N. 120. The bridge was built between May 1941 and November 1945 in substitution of the old wooden bridge that existed before.

This is the only movable bridge in a National Road in Portugal.

The bridge connects the two sides of the River Sado with a total length of 107.45 m, including the end supports. It is made up of three spans, with the end spans measuring 42.7 m. The centre span, previously movable, is 14.6 m long.

The steel superstructure is of the Bowstring type and each span is composed of two trellised beams that use a lower level chord to support a grill made of girders on which the reinforced concrete slab is seated.

A Principal Bridge Inspection was carried out during a first phase. This was basically a visual inspection to check any existing anomalies and estimate the cost of any repairs to the structure that we may have felt necessary.

A Special Inspection was carried out later, which not only made it possible to confirm the damages discovered in the Principal Inspection but also allowed for an exhaustive and detailed survey to be made of every existing anomaly.

Various Tests were carried out to characterize the material used for the superstructure, which were: Traction Tests, Fatigue Tests, Shock Tests, Bending Tests and Chemical Analysis.

Based on the available detailed survey and geo-technical prospection data, various three-dimensional models of the structure where analysed on the SAP2000 computer programme, so as to simulate the global behaviour of the structure.

Figure 1. General view of the bridge.

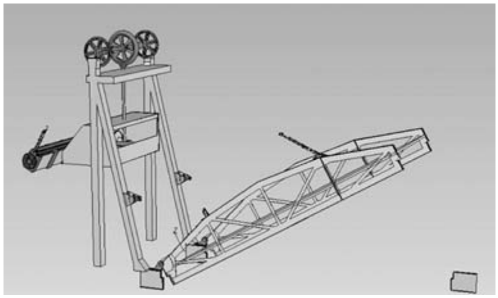

Figure 2. System of raising/lowering the middle section.

Thus it was seen that only the lower chord flanges of the superstructure need to be reinforced due to problems of accentuated corrosion levels.

In the analysis carried out for seismic loads it was verified that the piers are not safe. Significant contribution to this unsafe scenario is given by the low level of confinement that the silt and mud layers are unable to provide for the piers, and the considerable mass that they represent.

The strengthening of the piers is quite complex due to the amount of water, which can reach a depth of 12.0 m, and the use of cofferdams is both complicated and expensive.

Thus the solution was to share the seismic force from the platform between the piers and end supports, taking some of the burden off the piers and therefore making the bridge safe during seismic action.

Four 500 kN oleodynamic, histerectic type damping mechanisms were adopted to connect the platform to both end supports (two per end support), so as to allow these to contribute to the overall resistance of the bridge when subject to seismic loads in its longitudinal direction.

The system of raising/lowering the middle section has been inoperable for many years and there are no elements to show its correct working order. Therefore an exhaustive survey was made of the existing system in order to carry out the work necessary to make it safe.

The work carried out on this bridge mean that it will be ready for loads and safety standards imposed by current regulations without affecting its patrimonial value.

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Vouga Bridge – rehabilitation and strengthening

T.P. Mendonça, M.P. Almeida, A.R. Vieira & V.R. Brito
Betar Consultores, Lisbon, Portugal

ABSTRACT: The design of bridge over the River Vouga was based on standards that are no longer applied in Portugal. The structure is extremely complex for the calculation methods available at that time. The owners of the bridge are now giving special importance to the verification of these structures in accordance with current standards and calculation tools. The project of rehabilitation and strengthening developed for this bridge means that it now complies with the required standards of durability and resistance.

The viaduct over the River Vouga is situated on the A1 Motorway to the North between km 256 + 911.500 and 257 + 886.133, on the Aveiro Sul/Albergaria sub-stretch and facilitates the crossing of the River Vouga valley, which also carries the Vouga Valley railway line and the E.N. 230 main highway.

Figure 1. Partial elevation of the bridge.

Figure 2. Partial elevation of the access viaduct.

The rehabilitation study considered the evaluation of the state of conservation of the structure and verification of its structural safety in the light of current standards. The solutions for repair and/or strengthening could not jeopardize any future widening of the bridge.

The bridge is made up of two separate structural solutions: the bridge, which is 501.75 m long and the access viaduct, which is 488.63 m long, giving a total length of 990.38 m.

The Principal Bridge Inspection is a visual inspection and recording of its working conditions, as well of its elementary parts (Components).

The Special Inspection provided an exhaustive survey of the whole bridge. A detailed report was made, focusing on all deficiencies uncovered by this Special Inspection and with a detailed mapping of all pathologies.

The bridge was built in 1983 according to RSEP standards, whose definition of some loads is different from those in current standards.

In defining load conditions for safety evaluation of the bridge, the RSA standards where applied, except when seismic loads where to be considered, for which, by the imposition of BRISA, the Eurocode 8 and the respective National Application Document had to be followed.

Analysis of the existing structure indicated that some elements needed reinforcement. After structure reinforcement we developed a new analysis to verify structural safety.

The principal repairs to be carried out on the structure are:

- Concrete repairs
- Injection of cracks with epoxy resin.

The interventions allowed the clarification of the state of conservation and definition of the rehabilitation and reinforcement solutions in order to prepare this bridge for the loads imposed by current standards without jeopardizing any future plans for widening the bridge.

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Safety evaluation based on required strength for reinforced concrete members

J.-H. Cheon, D.-J. Seong & H.-M. Shin
Department of Civil and Environmental Engineering, Sungkyunkwan University, Suwon, Kyonggi-do, Korea

S.-C. Lee
Korea Infrastructure Safety and Technology Co., Koyang, Kyonggi-do, Korea

ABSTRACT: The Purpose of this study is to offer an appropriate and reliable safety evaluation method for the reinforced concrete members like as reinforced concrete flexural beams reinforced concrete columns, etc. A nonlinear finite element analysis program named RCAHEST (Reinforced Concrete Analysis in Higher Evaluation System Technology) was used in order to evaluate the ultimate strength analytically for the reinforced concrete members that have complicated mechanical behaviors. The nonlinear material model for the reinforced concrete is composed of models for characterizing the behavior of the concrete, in addition to a model for characterizing the reinforcing bars. The proposed numerical method for the safety evaluation of reinforced concrete bridge structures that is consisted of reinforced concrete member is verified by comparison with reliable experimental results.

1 INTRODUCTION

The design method of the reinforced concrete structures is converting from the current determinate strength based design method to the reliability based design method worldwide, and active studies have been undertaken in the US, Europe, and Japan etc. The main area of these studies is to secure the non-linear analysis technology with high reliability including the information of deformation capacity as well as the intensity of the structure. In this study, the data for the reinforced concrete deep beams and reinforced concrete columns tested by many researchers are used to verify the applicability of the nonlinear finite element analysis program (RCAHEST, **R**einforced **C**oncrete **A**nalysis in **H**igher **E**valuation **S**ystem **T**echnology). The RCAHEST was developed by Kim and Shin (Kim, T. H. et al. 2002a, b), at the Department of Civil and Environmental Engineering, Sungkyunkwan University. The proposed structural element library RCAHEST is built around the finite element analysis program shell named FEAP, developed by Taylor (Taylor, R. L. 2000). A full description of the nonlinear material model for reinforced concrete is given by Kim et al. (Kim, T. H. et al. 2002a, b).

2 NUMERICAL EXAMPLES

For the reinforced concrete deep beams, 102 experimental results are compared with the finite element analytical results. Data from experiments by Clark A.P. et al. (1951), and Smith et al. (1982) are used. For the reinforced concrete columns, 105 experimental results are compared with the analytical results. Data from experiments by Ang et al. (1998), Jung Young-soo et al. (2001), Lee Jae-hoon et al. (2001 and 2002), Guney Ozcebe and Murat Saatcioglu (1987), Chien-Hung Lin et al. (2004) and James F. PFISTER (1964) are used. These specimens's failure mode is variable under a variety of reinforcement ratio, loading conditions and aspect ratio. For the detailed information on the structural properties and dimensions of the specimens may be founded in the references.

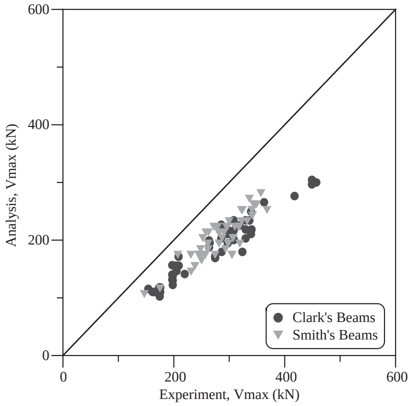

Figure 1. Comparison with experimental and analytical results for the reinforced concrete deep beams.

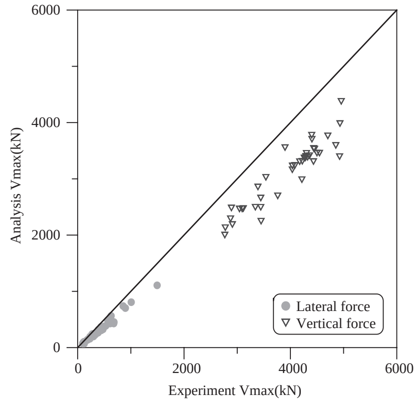

Figure 2. Comparison with experimental and analytical results for the reinforced concrete columns.

The regularity on the ratio of ultimate strength from the analysis and experiment is verified for each case, and in order to secure the reliability index, applicable to the structure with very large importance of the Euro Code, additional strength reduction coefficient which is apply to analytical result is calculated from the reliability analysis, respectively. The calculated strength reduction coefficient is 0.7 for the reinforced concrete deep beams and 0.76 for the reinforced concrete columns, respectively. The result is shown on Figure 1–2 for the each case.

3 CONCLUSIONS

In this study, on the reinforced concrete columns and deep beams, the ultimate strength is evaluated with RCAHEST that was developed by one of the authors, and present the method to evaluate the safety based on the reliability theory. Comparisons with experimental data, the following conclusions are reached.

1. The reliability on ultimate strength predicting is verified by applying the RCAHEST on the reinforced concrete deep beams and the reinforced concrete columns with various variables.
2. Through this study on the reliability analysis a new strength reduction coefficient is proposed for applying RCAHEST to the reliability based design method.
3. More efforts should be directed to apply the proposed procedure to the standard design code.

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Application of CFRP sheets with high fiber density in strengthening RC slabs subjected to fatigue load

H.K. Chai, H. Onishi & S. Matsui
Department of Civil Engineering, Osaka University, Suita, Osaka, Japan

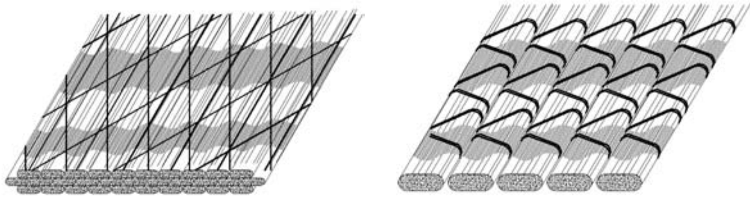
ABSTRACT: Research and development on the strengthening of deteriorated reinforced concrete (RC) slabs of highway bridges by carbon fiber reinforced plastic (CFRP) sheets have been extensively carried out in recent years. One important property of the CFRP sheets that affects the effect of the strengthening work is the tensile stiffness, which is determined by their Young's modulus and unit cross sectional area. Therefore, it is anticipated that CFRP sheets with normal Young's modulus but high fiber density (high unit cross sectional area) can achieve strengthening effect equivalent to those with higher Young's modulus but lower fiber density. In other words, the low Young's modulus of the CFRP sheets can be compensated by its high cross sectional area to acquire desire effect of strengthening.

The CFRP-concrete bond behavior determines the effectiveness of the strengthening. Thus, in the case of strengthening RC slabs subjected to fatigue loading, it is essential to investigate the durability of the bond and to clarify the debonding phenomenon.

In this paper, investigations are carried out on the strengthening effect of the CFRP sheets with fiber density of 600 g/m^2, which is higher than those of the commonly used CFRP sheets. The high-fiber-density CFRP sheets have two different knit structures, namely the laminate structure and the warp-knit structure, as shown in Figure 1. The warp-knit structure is introduced to increase the penetration of epoxy resin into the gap between bundles of wrapped carbon fibers, so that a stronger bond can be formed.

Small-scale bond tests were carried out to investigate the durability of the CFRP-concrete bond. The testing instrument as shown in Figure 2 applied flexure load to the concrete prisms bonded by CFRP sheets. Under repeated loading, failure of the specimens was characterized as complete detachment of sheet at one of the concrete prisms of the specimens, which occurred abruptly when the debonding area became considerably large. Experimental results verified the excellent bond durability by single layer of the 600 g/m^2 warp-knit type CFRP sheet, in which the fatigue cycles was approximately 3.3 ~ 5.0 times that of bond by double layers of the 300 g/m^2 laminate type CFRP sheets.

To investigate the effectiveness of the high-fiber-density CFRP sheets in strengthening RC slabs, fatigue tests on full-scale slab specimens were carried out by using the wheel running machine as



(a) Laminate structure (b) Warp-knit structure

Figure 1. Knit structure of CFRP sheets.

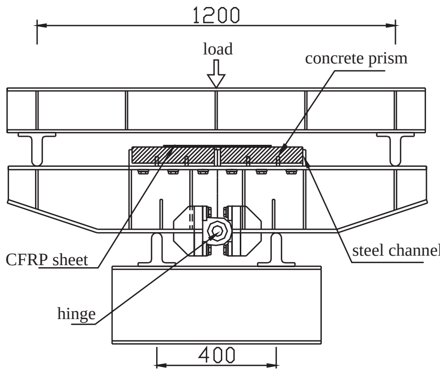


Figure 2. Instrument for bond test.

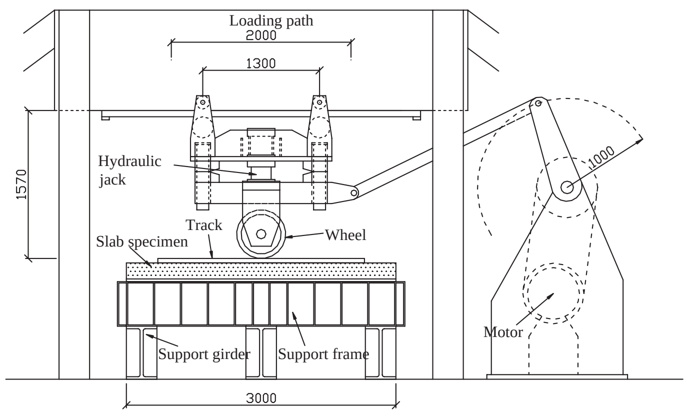


Figure 3. Wheel running machine.

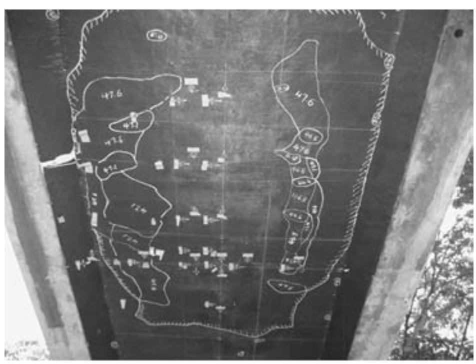


Figure 4. Debonding of CFRP sheet.

presented in Figure 3. The testing machine simulates repeated horizontally moving wheel load similar to traffic load by heavy trucks on slabs.

In the full-scale slab fatigue tests, the slab specimens failed in the form of punching shear of concrete. The debonding of the CFRP sheets from the tension surface of the specimens occurred as shown in Figure 4. By comparing the strain results of the CFRP sheets in small-scale prism tests and those of slab fatigue tests, it is concluded that the debonding of the CFRP sheets did not occur under flexure during the fatigue loading. It was actually initiated by peeling stress due to punching shear of concrete at the ultimate failure. The high-fiber-density CFRP sheets generally exhibited an excellent strengthening effect with fatigue life extension ratio of RC slabs at least equivalent to that of the slabs bonded with double layers of 300 g/m^2 laminate type CFRP sheets. Furthermore, the extension in fatigue life of the strengthened slabs increases substantially if the concrete properties are good. Besides, equivalent strengthening effect can be obtained by CFRP sheets of different Young's modulus and fiber density, with the condition that they have similar tensile stiffness.

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Black river parkway viaduct bearing replacement

H.S. Viljoen
Western Cape Provincial Administration, Roads Infrastructure Branch, Cape Town, South Africa

A.A. Newmark & B.E. Mawman
BKS (Pty) Ltd, Cape Town, South Africa

ABSTRACT: This Bridge, which is located on the Black River Parkway, a freeway near Cape Town in the Western Cape, South Africa, crosses several railway lines and a major arterial road. The 26 span prestressed concrete beam and slab type bridge has an overall length of 378 m and an overall width of 26 m. Construction was completed around 1965. The structure has a maximum transverse super elevation of 6.8% and a longitudinal grade of up to 3.2%. When the structure was originally constructed the fixity of the deck was obtained by using thin rubber bearings, which were placed directly on the steep transverse and longitudinal slopes of the supporting structure, which consist of columns with transverse trestle beams (A total of 1040 bearings). Significant displacement of the bearings and deck as well as resulting damage to the structure necessitated the replacement of bearings and associated repairs. The final bearing replacement proposal consisted of purpose made structural steel jack supporting units, supported directly from the trestle beams. This adjustable configuration eliminated problems related to height and founding conditions. A detailed analysis of jacking force/deflection recordings using calibrated pressure and deflection dial gauges, was used to determine the final jack release preset for the beam soffits. Each jacking operation required careful analysis of jacking forces and deflections in order to ensure that allowable structural deformations were not exceeded and that load was transferred effectively to the new bearings. Interface compatibility was achieved by injection of an epoxy mortar paste and allowing curing before release of jacking loads. The bearing replacement techniques developed for this project ensured minimal disruption of traffic on one of the busiest routes in Cape Town.

1 BACKGROUND

The 26 span prestressed concrete beam and slab type bridge has an overall length of 378 m and an overall width of 26 m. Construction was completed around 1965.

The super structure consists of simply supported prestressed precast concrete T-beams and infill slabs. The overall deck depth is 915 mm and the span length varies from 7 m to 21 m in length, but is generally about 14.8 m.

The substructure consists of piers with top transverse trestle beams supported on circular or oval columns. A solid wall type abutment is provided at the northern and southern ends. The structure is founded on piles with each circular or oval column provided with a separate pile cap. The height of the substructure above the pile caps varies from 4.5 m to 11.5 m.

The structure has a maximum transverse super elevation of 6.8% and a maximum longitudinal grade of 3.2%. When the structure was originally constructed the fixity of the deck was obtained by using very thin rubber bearings, 6 mm thick, at the fixed ends and thicker rubber bearings, 19 mm thick, at the free ends to allow thermal movement. The bearings were placed directly on the steep transverse and longitudinal slopes of the supporting structure which had no horizontal bearing surfaces at bearing locations as should normally be provided for this type of structure.

Figure 1. Spalling.

Figure 2. Displacement and contact of bearing surfaces.

Significant displacement of the bearings and deck as well as resulting damage to the structure necessitated the replacement of bearings and associated repairs. (Refer Figure 1 & 2)

2 BEARING REPLACEMENT PROCEDURE

The structure overall height and the absence of suitable founding conditions were the main considerations in the design of the jacking system used.

The final proposal consisted of a purpose made structural steel supporting structure (Steel I-beams and H- Beams), which made provision for two jack locations on both sides of a trestle beam. The absence of transverse diaphragms at these locations permitted the supporting of this structure on the top of the trestle beam and offsetting the structure center relative to the trestle beam made provision for different dead loads in adjacent spans. This configuration also eliminated problems related to height and founding conditions.

The jacks were provided with a mechanical lock-off system to maintain the deck at the required jacking displacements after jacking hydraulically.

The precast deck beams were first jacked to a height of about 15 mm above the existing bearing surfaces to allow removal of the existing rubber bearings and to clean and prepare the interface surfaces. The new bearings were then wedged to the underside of the beams installed using compressible wedges to keep the bearing to the underside of the beam and an epoxy mortar bedding was injected to the underside of the bearing pad. The precast beams were lowered into their final position and the jacks were released completely after curing of the bedding.

Excessive spalling at the ends of some of the trestle beams, as mentioned above, required extension of the lower trestle end and repair to the upper ends.

3 CONCLUSIONS

Having seen the effects of placing the precast beams on sloping supports, whether transversely or longitudinally, it should not readily be assumed that the friction between the rubber bearings and the concrete beams/support structure is sufficient to prevent permanent displacement of a bridge deck in the long-term.

The bearing replacement techniques developed for this project ensured minimal disruption of traffic on one of the busiest routes in Cape Town as well as the cost efficient extension of the service life of the structure.

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Rehabilitation of the U.S. route 46 bridge over Overpeck Creek

A.P. Ranasinghe & G. Khaitan
McCormick Taylor, Inc., Edison, New Jersey, USA

ABSTRACT: The U.S. Route 46 Bridge over Overpeck Creek in New Jersey, USA needed significant rehabilitation due to the poor condition of its superstructure system. Constructed in 1928, it is a six-span, simply-supported, concrete-encased 3-girder / floorbeam system with a four-girder double-leaf bascule span. The bridge substructure is supported on timber piles.

The project was one of the first to be on the New Jersey Department of Transportation's hyper-design/hyper-build schedule. This was necessary to avoid repeated maintenance problems on the heavily traveled structure (40,000 vehicles per day). The goal behind hyper-build is to streamline the design and construction phases of a traditional project. This not only saves time and money during both phases, it also reduces associated motorist costs, estimated for this project at $19,000 per day. On this project, delivery time was reduced from 48 months to 6 months.

A thorough study was conducted to choose between four different precast superstructure alternatives, namely the Effideck™ Deck Replacement System, Exodermic™ Bridge Deck System, Orthotropic deck system, and the Inverset™ Bridge System. All these options were evaluated across several key criteria, such as cost, construction duration, ease of erection, ease of fabrication, rideability, etc. Eventually, Inverset was identified as the best alternative for replacing the existing steel superstructure. This will add 50 years to the serviceable life of the bridge. Staged construction not only permitted the bridge to remain open during demolition and erection activities, there were no traffic detours, minimizing any impact to commuters.

Each Inverset unit is comprised of a high-strength (5000 psi) reinforced concrete slab on top of two steel beams. The beams are prestressed using a unique upside-down casting technique. Composite action occurs immediately upon turning the finished units right side up. The composite section supports the superstructure, live loads, and superimposed dead loads. Although initially more expensive, because superstructure elements were already built off-site, the Inverset system was assembled rapidly during construction and was ready for use immediately upon installation. Typically, three or four units were installed each day.

The rehabilitated bridge carrying U.S. Route 46 Westbound over Overpeck Creek consists of two 13′-0″ lanes, two 12′-0″ lanes, and one 6′-6″ sidewalk. The overall deck width is 64′-9″.

The Inverset prefabricated superstructure units serve as both the deck and superstructure of the rehabilitated bridge. The units were designed for 2004 AASHTO LRFD Specifications and NJDOT Structures Manual. Each panel is 9′-0″ to 9′-4″ wide and there are seven panels per span for a total of 42 for the bridge. The maximum weight per unit is 65 tons. Shear connectors are used to supplement the composite action between the beams and the 8.5 inch reinforced concrete deck slab. Non-shrink grout is used between adjacent Inverset panels. The units were pre-installed at the fabricator's yard to ensure proper fit in the field.

A comprehensive multimode spectral analysis of the existing structure was performed using a finite element model. Seismic design parameters included an acceleration coefficient of 0.18 and seismic performance zone 2. The resulting capacity/demand calculations revealed that the existing rocker bearings and timber piles would be overstressed during the design seismic event and new seismic isolation bearings were needed. A total of 168 isolation bearings are used in this bridge.

Due to the reduced superstructure depth of the Inverset system in comparison to the existing steel girders, there was extra clearance above the existing substructure and bottom of steel superstructure. To make up for this difference, a total of 86 precast high-strength concrete pedestals were used,

most of them supporting two bearings each. The height of the pedestals varies from 3′-3″ to 14′-9″. The pedestals are anchored to the existing substructure by threaded #22 rebars embedded to develop sufficient pullout capacity.

The existing piers and abutments needed several repairs. Rapid hardening concrete was used to reconstruct the abutment headers and backwalls. All areas of unsound, deteriorated and spalled concrete were repaired. Special procedures were followed to perform underwater repairs. Pressure injected grouting was used to repair concrete cracks both above and underwater. In several areas concrete was removed to adjust for the new superstructure and precast pedestals.

A custom-made cap & ball 2-rail metal railing was used on the vertical parapet on the northern end of the bridge. Concrete with colored pigments was used on the northern parapet and the northern face of the median barrier to yield a pleasing finish.

Due to the fact that the existing eastbound structure had no barrier on the northern side (the side adjoining the new westbound structure), a 7′-3″ wide median with two parapets was constructed on the southern end of the westbound structure. Since the curb elevations for the two structures were different, the height of the median barrier varies between the two. A four inch concrete cap prevents water or debris from entering the cavity between the two barriers.

Prior to placing the prefabricated superstructure units, the existing bridge deck and support beams were removed and the piers were modified by installing the precast pedestals. Both the demolition and erection activities were carried out using two Mi-Jack Travelift temporary gantry cranes working in tandem, each having 50 tons lifting capacity.

Due to the intricate nature of the construction and the inherent complications laden in rehabilitative work, several problems were encountered. Two of the notable ones included:

- Due to staged construction, some bascule girders needed to be secured. A special hold-down mechanism was designed for this purpose.
- In several areas the existing concrete in the substructure units had deteriorated and had to be removed and replaced. Concrete cores taken beyond the areas of deterioration indicated a minimum strength of 4500 psi.

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Renovation problems of historical concrete bridges

G. Borończyk-Płaska & W. Radomski
Warsaw University of Technology, Institute of Roads and Bridges, Poland

ABSTRACT: Historical bridges are a great part of bridge population in many countries. Many of these bridges are structurally deficient and/or functionally obsolete and, therefore, require to be rehabilitated or modernized, including strengthening and change of the geometrical parameters of the structures. Criteria for classification of the historical values of the old bridges are presented and discussed based on the Polish regulations. Selection criteria of the methods for rehabilitation and modernization of the historical bridges, especially concrete ones, are presented. The relevant examples of bridge rehabilitation and modernization taken from the Polish engineering practice are given.

1 INTRODUCTION

Old bridges are a great part of bridge population in many countries, including Poland. More or less part of these structures are of the considerable historical values. In spite of this, they are usually under normal service and, therefore, should meet all the save and functional conditions required for contemporary traffic. On the other hand, however, the historical bridges are in general structurally deficient and/or functionally obsolete and, therefore, require to be rehabilitated or modernized, including strengthening and change of the geometrical parameters of the structures, e.g. widening of the bridge deck.

Depending on the historical values and utilization function of the bridge, the difference requirements concerning its rehabilitation and modernization can be formulated. The paper is mainly focused on the situations when the historical bridge requires to be rehabilitated and/or modernized for normal service. The selection of the renovation methods applied is usually based on the two groups of requirements, namely – conservation and structural ones, which both should be met.

In general, conservation requirements concern preservation of the historical nature of the bridge by any or a little changes only in its original appearance, depending on the real historical values of the bridge. Structural requirements concern generally desired load-carrying capacity of the structure, its save utilization, durability, etc. These requirements are discussed below.

2 WHAT IS HISTORICAL BRIDGE?

The answer to this question can be based on the formal regulations as well as on the local or national tradition or on both of them. National and international regulations and recommendations concerning the objects of the great historical values, e.g. the rules of inclusion of the objects in the National Register of Historic Objects and Places existing in many countries, are rather of general character and do not concern directly bridges. That is why classification of the bridges into the historical ones is always more or less subjective. Some of the criteria of classification formulated on base of Polish regulations, are presented.

3 HISTORICAL BRIDGES IN POLAND

Registration system of the historical bridges is not sufficiently clearly developed in Poland. The bridges are included to the National Register of Historical Objects together with other relics or historical places and, unfortunately, there are not the separate regional or national lists for the historical bridges themselves. The results of the study concerning selection of the historical bridges from the official registers as well as finding some old bridges not included in them are summarized. Profits and restriction connected with inlusion of a given bridge in the National Register of Historical Objects are mentioned.

4 RESTORATION AND STRENGTHENING OF HISTORICAL MASONRY AND CONCRETE BRIDGES

In general, restoration and strengthening of the historical concrete bridges for their normal use may concern different, more or less complexed, situations. Sometimes both the load-carrying capacity of the bridge and its geometrical parameters do not meet the traffic conditions and the bridge requires to be strengthened, geometrically modernized and repaired because of cracks, material losses, corrosion, etc. It should be noticed that the change of geometrical parameters of the bridge always leads to the more or less evident change in its original appearance. Therefore, the relevant strengthening and modernizing operations can be performed on the old bridges of rather not the highest historical values where conservation of the original bridge appearance is normally the decisive factor. For strengthening of historical bridges, those methods should be selected, application of which do not result in the change of bridge original appearance. The selection should be always done according to the individual structural solutions of the bridge and its historical values as well as according to the conservation and traffic requirements. These requirements are formulated in each case by the relevant transport administration which defines all the conditions for bridge utilization as well as by the relevant conservation organizations and authorities, national or local ones. The Authors presents some methods of structural strengthening of both vault and concrete slab, grider or frame bridges, wchich anable to conserve their original appearance. The most modern methods of strengthening such as external plating (i.e., steel plate or steel flat bonding and/or bolting), external bonding of CFRP strips, shaped elements and/or fabrics and near surface mounted (NSM) CFRP reinforcement, which can be used for strengthening both vault masonry and concrete historical bridges as well as concrete slab, girder and frame historical bridges are briefly described.

5 CASE STUDIES

The problems presented above on rather general form are exemplified below by two historical concrete bridge structures in Poland, namely: bridge over the Rega river in Gryfice, north-west part of Poland and the Karowa street serpentine viaduct in Warsaw. Both of them are included in the National Register of Historical Objects.

6 FINAL REMARKS

In the paper the restoration problems of historical bridges are touched only. The problems are very complex and of important technical, economical, social and cultural meaning. The last decade has shown that the interests concerning the renovation of historical bridges are evidently increased in many countries including Poland. The historical bridges are considered as an important element of national and – in some particular cases – world heritage. Therefore, the special technical solutions are developed to meet all the conditions related both to safe utilization of the historical bridges and preservation of their historical values.

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Study of stress distribution of cracked steel plate with single sided CFRP material patching

Angus C.C. Lam, J.J. Roger Cheng & G.D. Kennedy
Department of Civil & Environmental Engineering, University of Alberta, Edmonton, Canada

Michael, C.H. Yam
Department of Building and Real Estate, Hong Kong Polytechnic University, Hong Kong, China

ABSTRACT: In a cracked element, all stresses must flow around the crack, inducing high stress concentrations at the crack tip that are responsible for crack propagation. As the development and application of fiber reinforced composite materials to different engineering structures are increasing gradually nowadays, composite fiber patching techniques are being considered as alternatives to traditional methods of strengthening and fatigue crack repair in steel structures. An experimental program was conducted to study the stress distribution in a patched element, and the effect of varying patch dimensions and stiffness on that stress distribution. Test results showed that single sided patching decreases crack tip strains significantly in the patched face, and increases them in the unpatched face. Finite element analysis of the test specimens were carried out and compared with the test data. The finite element results showed that the stress intensity factor of cracked steel plates with CFRP patching were reduced significantly.

1 INTRODUCTION

Composite fiber patching techniques are being considered as alternatives to traditional methods of strengthening and fatigue crack repair in steel structures. Layers of fiber are impregnated with epoxy and bonded to the surface of a damaged structure, stiffening the area and restricting crack opening. In this paper, the experimental results of the effects of patching on stress distribution through a cracked plate were presented. Varying parameters such as the length, width and stiffness of patching were included.

2 EXPERIMENTAL DESIGN

2.1 *Test specimens*

Ten tests were performed to study the effects of patching on stress distribution through a cracked plate. Nine of the "crack" plates were then patched on one side with carbon fiber composites prior to testing. The final plate was tested without a patch for reference. Six layers of fiber were applied to the majority of the specimens, providing a patch to adherend stiffness ratio, E_pt_p/E_st_s, of 0.16. A typical patched specimen is shown in Fig. 1.

3 FINITE ELEMENT ANAYLSIS OF TEST SPECIMEN

The specimens were analyzed by using finite element method. Eight nodes shell elements were used in the model. At the location of crack tip, collapsed element with mid nodes located at quarter

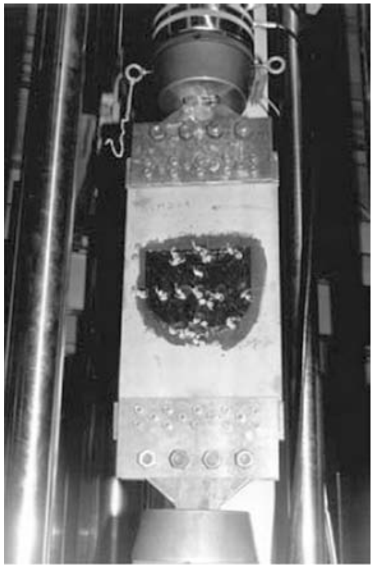

Figure 1. Typical gauged specimen, bonded patch test.

point were used in order to obtain the stress intensity factor, SIF (K) at crack tip numerically. The strain results of the finite element model compared well with the test results. For the case of specimen with $L = 100$ mm, $w = 160$ mm and $n = 6$ (R116), the SIF of crack tip on patch side was reduced from 36.71 to 23.64 $MPa\sqrt{m}$ (36% reduction).

4 SUMMARY AND CONCLUSION

Experimental testing was used to study the effects of a bonded, single sided composite patch re-pair on the flow of load through a steel plate with an internal, through-thickness crack. The objective of the program was to determine the changes in strain distribution that occur when a bonded patch is applied to one face of a cracked plate subjected to tensile loading. With the current test results, it was shown that (1) on the patched face of a cracked steel element, load flows through the patch across the crack and transfers quickly back into the steel, (2) single sided patching decreases crack tip strains significantly in the patched face, and increases them slightly in the unpatched face, (3) compared with an unpatched plate, the strain distribution for a patched plate through the center of the plate thickness along an axis parallel and close to the axis of an internal crack shows lower extremes and smoother gradients – strains near the center-line of the crack are higher, and near the crack tip are lower. Loads are also drawn back to the low strain area around the crack centerline more quickly in a patched plate. These two factors enable constant stress distribution across the width of the patched plate to be re-established in a shorter distance than for an unpatched plate, (4) current test results showed that patch width has little effect on strain distribution in the plate. Meanwhile, higher stiffness patched reduces crack tip strain more effectively and (5) The cracked steel plates with single side CFRP patching were analyzed by finite element method. The strain results of the finite element models with three layer technique compared well with the test data. With the finite element models, SIF of the cracked plate with CFRP patching were computed. In current study, a maximum reduction of SIF (55% reduction) was obtained for specimen R116 on the patched side. However, the SIF of crack tip on unpatched side was not reduced effectively. It was shown that the SIF of crack tip on unpatched side was almost the same as value of the specimen without CFRP patching.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Strengthening of composite beams with external tendons using a rating factor equation

D.H Choi, S.H. Chung & D.M. Yoo
Department of Civil Engineering, Hanyang University, Seoul, Korea

M.Y. Han
Department of Civil Engineering, Ajou University, Suwon, Korea

ABSTRACT: The prestressing technique using external tendons can be considered an effective method of strengthening bridges that are deteriorating due to increasing overloading and progressive structural aging. This technique is not only easy to perform but it is also convenient to maintain the tendon because it is exposed in the air; it is a practical and cost-effective strengthening method (Troitsky et al. 1989). The advantages of the technique are the enlargement of the elastic range of behavior, the increment of the ultimate capacity, and improvement of the fatigue and fracture strength (Saadatmanesh et al. 1989a, b, c). Therefore, prestressing is applied to the reinforcement of many concrete girders (Harajli 1993, Ng 2003). In order to achieve an more economical and effective strengthening, design variables such as configuration of tendon, the number of tendons and the initial tendon force have to be determined effectively to reinforce an existing bridge with external tendons.

In this paper, external tendons are used for strengthening of steel-concrete composite beams. An analytic expression for the increment in the initial tendon force is derived using the virtual work method for configurations of straight and draped tendons under external loads. Considering the initial tendon force and its increment under external loads, a new rating factor equation is introduced. A systematic procedure has been developed to calculate the number of tendons and the initial tendon force using the proposed rating factor equation. A design example is given to demonstrate the effect of the proposed equation on increasing load-carrying capacity in existing steel-concrete composite bridges.

Stress distribution in any cross section of a composite beam strengthened with external tendons under each stage of loading is calculated. Dead load and live load cause compressive stresses in the concrete slab and top flange, and tensile stress in the bottom flange of the simply supported beam. Tendon force causes compressive stresses in all cross section of the composite beam; the negative moment due to tendon force causes compressive stresses in the upper cross section and tensile stresses in the lower cross section of the neutral axis. Under live loading, there is an increment in the initial tendon force. Accordingly, the increment of tendon force causes stresses in the cross section of the composite beam.

An analytic expression for the increment in the initial tendon force for configurations of straight and draped tendons under external loads is determined using the virtual work (Saadatmanesh et al. 1989c, Troitsky 1990). External loads used are truck load (DB) and lane load (DL) that are prescribed in KHBS (2005). The finite element program, LUSAS (2005), was performed to compare with the virtual work method. The results show little difference between the two methods.

Considering the initial tendon force and its increment under external loads, a new rating factor equation is introduced. In the allowable stress method, in-service bridges strengthened with external tendons are evaluated by a rating factor equation as follows:

$$RF' = \frac{f_a - (f_{DL} + f_T)}{(f_{LL} + f_{\Delta T})(1+i)}$$

where f_a = allowable stress of the member; f_{DL} = stress due to dead load; f_{LL} = stress due to live load; i = impact factor (15/(40 + span length)); f_T = stress due to tendon force; and $f_{\Delta T}$ = stress due to increment of tendon force.

A systematic procedure has been described to determine both the number of tendons and the initial tendon force using the proposed rating factor equation after the material and configuration of the external tendon are selected.

A design example of a plate girder bridge constructed in 1973 with a span of 40 m was selected for enhancing load carrying capacity by introducing straight external tendons under the bottom of the lower flange. The existing bridge was originally designed for DB-18 truck loading. The number of tendon and initial tendon force are calculated using the proposed rating factor equation for upgrading the bridge that have a target rating factor of 1.2 for DB-24 truck loading.

The effects of the proposed equation for increasing load-carrying capacity in existing steel-concrete composite bridge are summarized as follows: (1) the stress in the lower flange due to increment in the initial tendon force is 5.1% of that due to initial tendon force; (2) the rating factor considering the increment in the initial tendon force is 1.20, whereas it is 1.18 if the increment is neglected.

In this study, the increment in the initial tendon force for configurations of straight and draped tendons under external loads are derived in an analytic expression. And a systematic procedure has been described to calculate the number of tendons and the initial tendon force using the proposed rating factor equation.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Evaluation of safety for repair work with welding – features of thermal stress generated by cutting

S.H. Lee
Joining & Welding Research Inst., Osaka Univ., Osaka, Japan

K.H. Chang
Dept. of Civil & Environmental Eng., Chung-Ang Univ., Seoul, Korea

Y.-C. Kim
Joining & Welding Research Inst., Osaka Univ., Osaka, Japan

1 INTRODUCTION

It is very important whether structure is safe during repair work. There are abundant methods to repair steel bridges. These of steel bridge generally include cutting, bolting and welding procedures. So, a chain of confirmation against the safety of structure is necessary.

Safety evaluation method, which has regard to heat effect, is necessary to confirm the safety of structure during the repair work. Using a welding and bolting in the repair work except attaching an additional member is accompanied with the cutting procedure. So, in order to repair steel bridge with welding or bolting, it matter whether structure is safe during cutting or welding.

2 CONDITIONS FOR ANALYSIS

2.1 *Analysis model for cutting*

The cutting model was decided with reference to existing welding model. Table 1 and Figure 1 show the cutting model for investigating the features of cutting residual stress. In this table, three types of thickness of the steel plates were used. Those were usually used to construct a steel bridge in former days.

Table 1. Cutting conditions.

Model	Range	L(mm)	B(mm)	L/B	$W_m(mm)$	Remarks
CR1M1_H	I	100	500	0.2	2	$M = 1.25$, $T_f = 1450(°C)$, $v = 5(mm/sec)$,
CR1M2_H		200	500	0.4	2	$\rho = 7.8(g/mm^3)$, $c = 0.13(cal/°C \cdot g)$,
CR1M3_H		200	400	0.5	2	$h = 6, 9, 12(mm)$
CR1M4_H		300	500	0.6	5	'H' of model is for the thickness ($h = 6, 9, 12$)
CR1M5_H		400	500	0.8	10	of cutting model.
CR1M6_H		100	500	1.0	2	
CR1M7_H		600	400	1.5	8	
CR2M1_H	II&III	400	200	2.0	5	
CR2M2_H		500	200	2.5	10	
CR2M3_H		600	200	3.0	8	
CR2M4_H		700	200	3.5	5	
CR2M5_H		1000	250	4.0	8	
CR2M6_H		900	200	4.5	8	
CR2M7_H		1000	200	5.0	10	

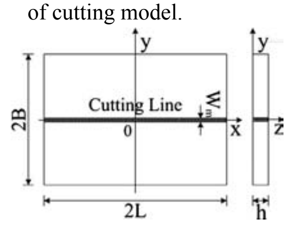

Figure 1. Analysis model for cutting.

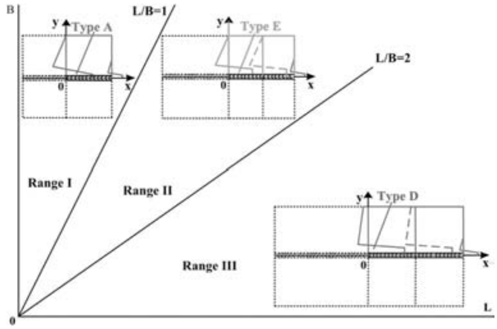

Figure 2. Pattern of residual stress generated by cutting.

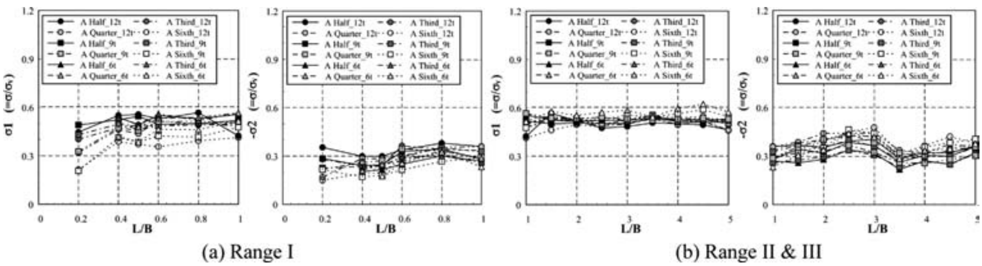

Figure 3. L/B dependence on the magnitude of cutting residual stress.

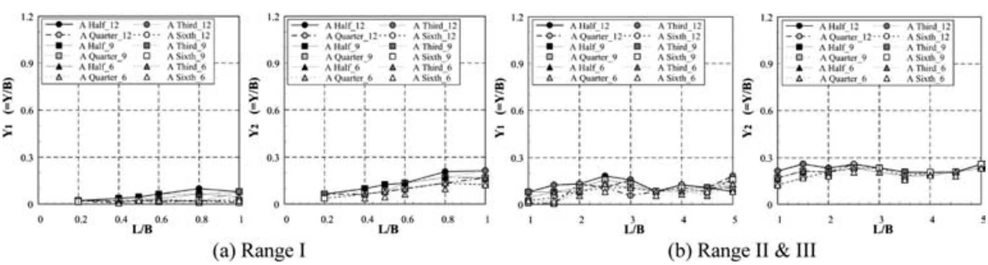

Figure 4. L/B dependence on the position of cutting residual stress.

3 FEATURE OF STRESS GENERATED BY CUTTING

3.1 *Distribution of residual stress*

It can be recognized that residual stress distribution generated by cutting is generally shown like Figure 2 based on the basic pattern of residual stress.

3.2 *Magnitude of residual stress*

Figure 3 shows the L/B dependence on the magnitude of residual stress generated by cutting.

Table 2 shows the estimate equation of the magnitude of critical residual stress generated by cutting. The effect of thickness of plate is also considered in these equations.

3.3 *Position of residual stress generated by cutting*

Figure 4 shows the L/B dependence on the position of residual stress generated by cutting.

Table 2 shows the estimate equations or mean value for the distance from cutting line generated by cutting. The effect of thickness of plate is also considered in these equations.

Table 2. Estimate equation of the cutting residual stress and distance at each break point.

		Estimate equation of residual stress		Equation or mean value of distance	
Range	Position	$\sigma 1$	$\sigma 2$	Y_1	Y_2
Range I (L/B < 1)	At a Half	$(0.52 + 262.5e^{-1.64h})\sigma_Y$	$(0.003 + 0.113h^{0.444})\sigma_Y$	$0.139B(1 - 0.471^{(L/B)})$	$0.249B(1 - 0.267^{(L/B)})$
	At a Third	$(0.49 + 17.33e^{-0.98h})\sigma_Y$	$(0.103 + 0.082h^{0.396})\sigma_Y$	$1.167B(1 - 0.951^{(L/B)})$	$0.268B(1 - 0.356^{(L/B)})$
	At a Quarter	$(0.5796e^{-0.018h})\sigma_Y$	$(-0.052 + 0.258h^{0.15})\sigma_Y$	$0.0011B \times 9.735^{(L/B)})$	$0.276B(1 - 0.465^{(L/B)})$
	At a Sixth	$(0.6012e^{-0.038h})\sigma_Y$	$(0.543 - 0.267h^{0.021})\sigma_Y$	$(0.019 + 0.00007(L/B)^{9.07})B$	$0.263B(1 - 0.513^{(L/B)})$
Range II & III (1 < L/B)	At a Half	$(0.5626 \times 9.892^{h})\sigma_Y$	$(-1.396 + 1.527h^{0.05})\sigma_Y$	$(-1.056 + 1.098h^{0.03})B$	$(0.18 + 0.0067h^{0.797})B$
	At a Third	$(0.5632 \times 0.991^{h})\sigma_Y$	$(-1.014 + 1.116h^{0.09})\sigma_Y$	$(-1.492 + 1.525h^{0.02})B$	$(0.205 + 0.00002h^{2.92})B$
	At a Quarter	$(0.5612 \times 0.992^{h})\sigma_Y$	$(-1.712 + 1.814h^{0.06})\sigma_Y$	$0.106B$	$(0.182 + 0.006h^{0.776})B$
	At a Sixth	$(0.6431 \times 0.983^{h})\sigma_Y$	$(-0.148 + 0.425h^{0.10})\sigma_Y$	$(-1.214 + 1.223h^{0.03})B$	$(-0.0047 + 0.148h^{0.17})B$

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost – Cruz, Frangopol & Neves (eds)

Rehabilitation of the Barra Bridge – the strengthening side

A. Rito, M. Loureiro & S. Bispo
Armando Rito, Lda., Lisbon, Portugal

T. Ripper
Leb, Lda., Cascais, Portugal

ABSTRACT: The Barra Bridge over the Mira Canal, in Aveiro, designed by Professor Edgar Cardoso in 1971, opened to traffic in 1975. Soon after, an excessive deflection of the central span longitudinal cantilevers was detected. Since then, several inspections have been carried out, revealing the existence of many other problems, especially relevant to the conservation of the bridge. The structural analysis revealed the need to proceed with a global and local strengthening of the structure. Various solutions were adopted, such as the installation of external pre-stressing in the deck and in some piers, the piers strengthening with carbon fiber laminates, the strengthening of the box girders with structural steel, the installation of viscous fluid dampers and other local strengthening solutions.

1 INTRODUCTION

The present paper refers to the rehabilitation of the Barra Bridge over the Mira Canal, in Aveiro, inserted in the national road E.N. 109-7. The bridge was designed by Professor Edgar Cardoso in 1971, and was opened to traffic in 1975. The objective of the strengthening design was to adapt the bridge to the new code rules, notably, new live loads, seismic actions and durability aspects, thus securing the improvement of the bridge performance levels.

The various inspections and the structural analysis data were carefully analyzed, the global and local load capacity and response levels of the structure were evaluated and, in consequence, the necessary strengthening operations to accomplish the defined objectives were established.

2 DESCRIPTION OF THE BRIDGE

The bridge has a total length of 578 meter (Fig. 1), divided into three double box girder central spans and two access viaducts with current spans of 32 meter composed by four longitudinal beams connected by the top slab. The central span has two 34 meter long cantilevers connected by an isostatic span. The piers are portal frames bearing on two caissons.

The abutments are hollow boxes. The deck horizontal forces are transferred by struts/tie to the raft foundation that surmounts the caissons.

Figure 1. General view of the Barra Bridge.

3 STRUCTURAL PROBLEMS DETECTED

The first sign of the Barra Bridge's less adequate performance was detected soon after its opening to traffic, with the development of an excessive deflection of the central span cantilevers. Since then, several inspections have been carried out, showing the existence of many other problems concerning mainly the maintenance of the bridge. In fact, the bridge's advanced deterioration suggested that a major structural repair work would be needed. Also the structural analysis showed the need for global and local strengthening so that the structure could comply with the existing code rules.

4 STRENGTHENING AND MAINTENANCE SOLUTIONS ADOPTED

The deck will be strengthened with the installation of longitudinal external tendons in such a way that the acting forces will be reduced and the tension stresses will be eliminated, thus ensuring that the service limit states are verified. The top slab will be strengthened with carbon fiber laminates on the fixed-end cross section of the cantilever free edge zone, in such a way that the ultimate limit state of bending in the transversal direction will be assured. Due to the insufficiency of compression flange area on the box girder spans, the bottom slabs will be longitudinally strengthened with structural steel connected to the slab by steel connectors and perpendicular to the slab fixing devices. Due to the lack of shear reinforcement in the cantilever and adjoining spans, the box girder webs will be strengthened with vertical pre-stressing bars, longitudinally spaced by 1 meter. In order to ensure that the ultimate limit states of shear and bending near the isostatic span supports are verified, these zones will be shear strengthened with 45° slope pre-stressing bars, and bending strengthened with horizontal bars. Longitudinal and transversal bracing devices constituted by bars and neoprene blocks, allowing solely rotations around transversal and vertical axes, will also be installed in the isostatic span joints. One of the objectives of the Rehabilitation project was to level the bridge platform to its original configuration by strengthening and filling. However, since that was not possible, it was decided to raise the simple supported span by 7 cm which, together with the fillings of the cantilevers that support it and the filling of the isostatic span itself, represent a leveling that will stay only 7 cm bellow the original grade line at its most unfavorable point. The footways were widened to allow the simultaneous circulation of pedestrians and cyclists. Due to the modification of the structural system and the deck widening, the expansion joints will be replaced. All the bearings will also be replaced due to their state of degradation.

To assure the ultimate limit states, the piers' shafts and higher beams will be strengthened with carbon fiber laminates. Whenever possible, the bond anchorage of the laminates is assured. When that is not possible due to the members' configuration, mechanical anchorage is achieved by pre-stressing bars and/or superposition with reinforcement bars. Fixing devices consisting of four tendons of three 15 mm strands were adopted to keep the deck anchored to the transition piers when negative reactions are developed. In order to cancel the tension forces transmitted to these piers by the deck's reaction, their shafts will be pre-stressed.

For the diminution of the seismic forces induced by the deck into the abutments, the solution adopted consists of a total release of the deck at the abutments, fixing the isostatic span in the longitudinal direction and introducing viscous fluid dampers between the deck and abutments.

5 CONCLUSIONS

The Barra Bridge is a structure that, due to its size and to the complexity of the structural problems detected, required an in-depth study on how to proceed with its rehabilitation and strengthening. This study consisted of the development and optimization of strengthening solutions, bringing together the needs of global strengthening with the minimization of the individual tasks needed to put those reinforcements into effect. The diversity and peculiarity of some structural problems detected, imposed the need to search for new strengthening techniques, exploring the potentialities of new materials and of different types of interaction between materials so as to achieve efficient, lasting, and practicable strengthening solutions.

*Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)*

Rehabilitation of the Barra Bridge – the repair side

A. Rito, S. Bispo & M. Loureiro
Armando Rito, Lda., Lisbon, Portugal

T. Ripper, P. Marques & J.N. Ferreira
Leb, Lda., Cascais, Portugal

ABSTRACT: The requirements for the best compliance of the Barra Bridge Rehabilitation Project, on its repair side, had obliged the most precise definition the areas to be treated and the products and methods to be applied, following the recent CEN standards and other international recommendations, being the tasks defined according to the degradation levels locally and globally detected and to the extension of the works to be performed in each case.

1 OBJECTIVES

This text aims to present the most relevant aspects of the repair side of the rehabilitation Project of the Barra Bridge, that regards the improvement of its durability in such a way to:

- Stop and reverse the basic mechanism of degradation of the structure;
- To retrofit the structure in order to enable it for a better response, in a near future, against the aggressiveness of the environment, thus improving the structure to a service life period compatible to its social and economical relevance;
- To level the construction's performance up to the requirements of the recent European codes.

2 BRIEF DESCRIPTION OF THE BRIDGE

The Barra bridge has a total length of 578 m and is entirely constructed in reinforced concrete – being prestressed the main elements of the deck – had its structural project developed by the worldly known Prof. Edgar Cardoso in 1971. The bridge is kept in service since 1975.

3 CHARACTERIZATION OF THE ENVIRONMENT

The interpretation of the corrosion phenomena obliges the knowledge of the atmosphere constituent characteristics, which implies the climate characterization of Portugal, with special prevalence in the west coast area that includes Aveiro city, where the Barra Bridge is located.

Thus considering the highest values of relative humidity of the air (over 80%) all along the Portuguese west coast, the factor humidity time, related to the number of hours through which the relative humidity of the air exceeds 80% and the temperature is over 0°C, stands as the main climatic element that influences the atmospheric corrosion.

The Brooks index for this construction reflects a moderate degree of deterioration ($2 < I < 5$) which, however, represents itself as the higher in Portugal. Concerning to the corrodibility based on climatic parameters the category for this bridge is C3/C4 from a total of five categories.

The last approach regards the corrodibility based on the corrosion velocity of the steel. Facing this approach the corresponding category is C4.

4 IDENTIFICATION OF THE AGGRESSIVE AGENTS AND CHARACTERIZATION OF THE MECHANISMS OF DEGRADATION

The main information that had resulted from a package of technical tests carried out is:

– Regarding durability parameters, the concrete used in all structural elements shall be classified as a poor or medium quality one, since its permeability coefficient is extremely high, reaching, on the most external 3 cm, values higher than 10^{-10}, the open porosity index goes up to 11% and the chloride concentration varies between 0,4% and 0,9%;
– The evaluation of the rebars corrosion electrical potential as well as the electrical conductivity tests performed had confirmed that there is already active corrosion on the concrete surfaces at the splash zone;
– The thickness of the rebars concrete cover ran from 35 m (at the cantilever slabs) to 32,7 mm (columns and frames) and to 28, 1 mm (longitudinal beams);

From the analysis of the results of the performed tests emerges the conclusion that the most critical situation belongs to the splash zone elements. On the superstructure the worst situations are clearly related to the areas with less concrete cover, more intense concentration of internal rebars, more exposition to the aggressiveness of the environment.

Thus it is reasonable to conclude that the basic mechanism of degradation is the corrosion of the reinforcement rebars caused by the action performed, from the exterior to the interior, by the chloride ions which are transported by air. This was made easier due to the high porosity of the concrete that has also high capillarity, to the minimum thickness of the rebars' concrete cover, to the very high water – cement ratio and to the absence of any kind of protective coating.

5 REPAIR PROJECT CRITERIA

The repair side of the Project has adopted a multi-strategy of intervention based on surgical repair tasks plus the addition of corrosion inhibitors and plus a protection based upon the use of special coatings, impermeable to water, but permeable to the water vapour, and with high capacity to prevent the ingression of the aggressive environment agent, chloride ions, especially.

The understanding of each and every repair task specified implies the observation, simultaneously, of the drawings and the written specifications of the Project. In order to enhance the value of these written specifications the idea was to establish a straight link between them and the drawings that for those purposes had also included detailed schemes of the repair methods, such as exemplified for the columns, in the figure that follows.

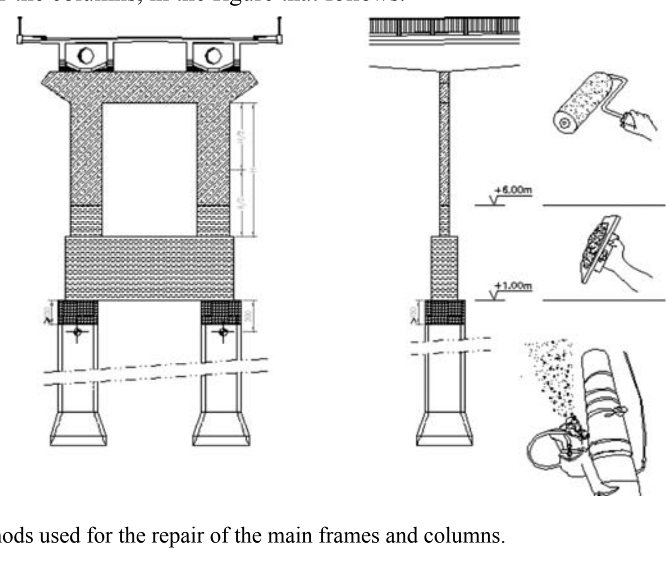

Figure 1. Methods used for the repair of the main frames and columns.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Numerical analysis of two-way concrete slabs with openings strengthened with CFRP

P. Rusinowski
Technical University of Denmark, Department of Civil Engineering, Kgs. Lyngby, Denmark and Norut Technology Ltd., Narvik, Norway

B. Täljsten
Technical University of Denmark, Department of Civil Engineering, Kgs. Lyngby, Denmark and Luleå University of Technology, Division of Structural Engineering, Luleå, Sweden

O. Enochsson, T. Olofsson & J. Lundqvist
Luleå University of Technology, Division of Structural Engineering, Luleå, Sweden

ABSTRACT: Carbon Fibre Reinforced Polymers, CFRP, offer excellent corrosion resistance to environmental agents as well as the advantages of high stiffness-to-weight and strength-to-weight ratios when compared to conventional construction materials. Perhaps the biggest advantage of CFRP is its tailorability. One common application for CFRP sheets is to strengthen slabs and walls when openings are to be made. In spite of this, there have not been many studies reported on slabs with openings strengthened with CFRP and especially, not with distributed loading. This paper presents numerical analyses of simply supported two-way concrete slabs with openings strengthened with CFRP sheets. The finite element program ABAQUS is utilized for the analyses. The analyses are compared with full-scale laboratory tests and show a good agreement.

1 INTRODUCTION

Reinforced concrete slabs and shells are commonly used structural elements in constructions. Due to changes in use and new functionality demands, existing structures often need to be rebuilt. Installation of elements, such as staircases, elevators, ventilation systems, doors or windows requires new openings in existing slabs or shells. External bonded FRP sheets are becoming an increasingly used in reconstruction.

The simplified calculation method of FRP strengthening for slabs with openings, applied also in this paper, uses the instructions, given by the building codes, concerning required amount of steel reinforcement in slabs cast with openings. The method assumes that bands around the opening, where additional reinforcement is applied, are separate reinforced concrete beams. When FRP strengthening is used, the strengthened sectional area can be calculated by simply converting the required steel reinforcement into FRP.

The entire research program investigates the effect of different size of openings and different strengthening arrangements. The biggest advantage of the experiments carried out in this study is the application of line supports and submitting uniformly distributed. This configuration corresponds well with adopted existing design methods of traditional concrete slabs.

The aim of the study presented in this paper is to model the behaviour of CFRP strengthened concrete slabs using numerical analysis and to compare the result with experimental tests. The entire experimental program contains 11 slabs with different sizes of the openings and different strengthening arrangements. This paper, however, focuses only on numerical modelling which is compared with three of the slabs from the experiments. Figure 1 shows the types of specimens considered in this paper.

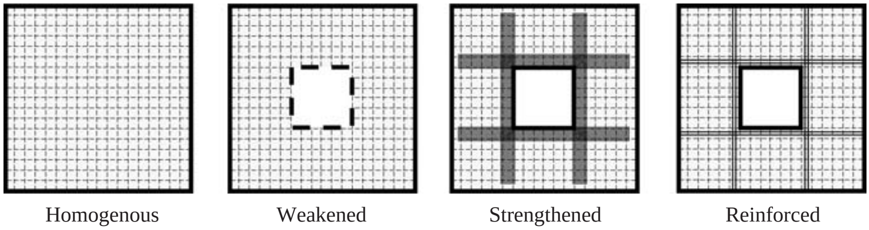

Figure 1. Considered slab configurations. All slabs are quadratic with a side length of 2600 mm and a thickness of 100 mm. The openings (850 × 850 mm) are located in the middle of the slab. The "Reinforced" slab is only studied numerically.

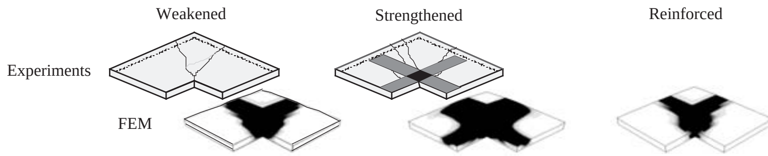

Figure 2. Comparison between the final propagation of cracks in the experiments and the maximum principal strain achieved in the FE.

Explicit integration was used in FE-analysis as a solver technique, which is originally a dynamic method but may be used also to solve static (so-called quasi-static) analysis. A damaged plasticity constitutive model for concrete is used in the FE-calculations. The CFRP material is considered to be elastic.

The interface between the concrete slab and the CFRP is modelled as ideal bonding. This property is obtained by using interaction "tie" between surfaces, which locks the distance between adjacent nodes. It is assumed that carbon fibres are applied directly on the concrete surface, i.e. no intermediate layer modelling the epoxy bond is considered. The material property of carbon fibres is assigned using orthotropic material "lamina" in ABAQUS.

Similarities in comparison of crack distribution between experiments and numerical analyses are shown in Figure 2. The plastic region in CFRP strengthened slab is more widely spread compared with the other analysed and tested slabs, which may be the effect of distributed strengthening.

2 DISCUSSION AND CONCLUSIONS

The FE calculations used in this study show good agreement with the experiments although relatively simple model was applied. In opposite to the experiments, however, CFRP failure was not reached in the numerical calculations and the possible reason is the model of the concrete behaviour in tension. In the numerical calculations cracks are distributed over a larger region, while in the reality, a few discrete cracks propagate and the stresses are concentrated in the reinforcing steel and CFRP sheets.

The model of CFRP strengthening assumes an ideal bonding, i.e. no interlayer imitating the epoxy adhesive is considered. This entail that the anchorage length cannot be investigated. Furthermore, the stiffness of CFRP is assumed to be equal in compression and tension and this is suitable for cases where CFRP sheets are only stretched. For more complex problems, as two-way concrete slabs or structures subjected to cycle loading, different material properties of CFRP in tension and compression should be assigned.

Apart from these observations, the explicit FE analysis gives more insight to the strengthening effect of the CFRP sheets. It also gives more confidence to the future utilization of the non-linear FE analysis in order to evaluate larger slabs with different opening configurations.

For further investigation of two-way concrete slabs with openings it is necessary to study the bond behaviour between the CFRP and the underlying concrete in detail. Furthermore, better models of the crack localisation in the concrete are needed to be able to accurately predict the ultimate failure.

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Strengthening of concrete structures by external prestressing

H. Nordin
Luleå University of Technology, Sweden

B. Täljsten
Technical University of Denmark, Denmark; Luleå University of Technology, Sweden

ABSTRACT: Rehabilitation and strengthening of existing concrete structures has become more and more in focus during the last decade. All over the world there are structures intended for living and transportation. The structures are of varying quality and function, but they are all ageing and deteriorating over time. Some of these structures will need to be replaced since they are in such a bad condition. However, it is not only the deterioration processes that make upgrading necessary, errors can have been made during the design or construction phase so that the structure needs to be strengthened before it can be used. The causes for repair and/or strengthening can be many, but normally deteriorated concrete, steel corrosion, change of use, increased demands on the structure, errors in the design or/and construction phase or accidents are governing factors. Many methods to repair or/and strengthen concrete structures exists such as concrete overlays, shotcrete, use of external prestressed tendons, just to mention a few. Prestressing is in particular interesting with several comparable advantages to other methods. In this paper the use of prestressing for repair and strengthening are briefly discussed and tests on concrete T-beams with external Prestressed tendons of steel or CFRP (Carbon Fibre Reinforced Polymer) are presented. The tests shown that prestressing is a very effective way to increase the existing load carrying capacity of existing concrete members. The presented project is a small part of a larger European funded project, the "Sustainable Bridges", where the aim is to evaluate the load carrying capacity and life of existing railway bridges with the purpose to increase existing load carrying capacity with 25% and the train speed to 350 km/h.

The tests were carried out at Luleå University of Technology (LTU). The test specimens were concrete T-beams with a length of 6 meters, see figure 1. The beams were loaded under four-point bending, the load was applied with deformation control at 0.2 mm/s until failure or to a point where the beam no longer could carry any more load. A total of eight beams were tested during the series. The strengthening techniques used were externally prestressed steel rods, externally prestressed CFRP rods and Near Surface Mounted Reinforcement (NSMR) CFRP rods with and without prestress.

For the steel tendons a traditional steel wedge anchor was used, but that was not possible for the CFRP tendons, as normal steel wedge anchors would crush the FRP tendons. For the tests an anchor was developed using a nylon wedge. To get better effect of the anchor, the tendons had quarts sand glued on them in the zone for anchoring. As those anchors would not be able to take as high forces as a steel anchor on a steel tendon six tendons were used instead of two. However, as the prestressing was applied it became clear that it would not be possible to achieve the same prestressing force as with the steel tendons.

With the exception of the beam with external CFRP tendons all the tested beams behaved as expected. The strengthening effects for the prestressed beams were over 100% both for concrete cracking and steel yielding.

When looking at the post-steel yielding behaviour it is interesting to compare the beams strengthened with unbonded tendons and those with bonded tendons (Steel 3 and NSMR PS). The beams with bonded tendons and rods showed a better behaviour after steel yielding than those with

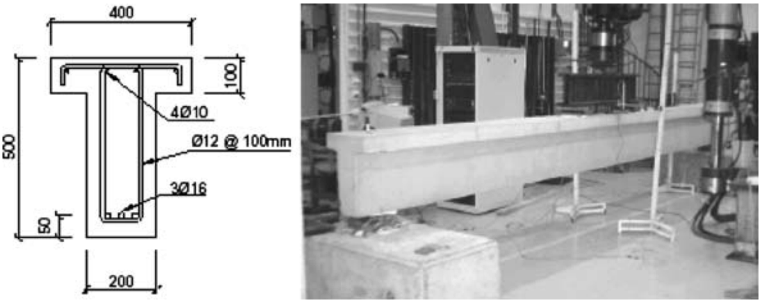

Figure 1. Cross-section of the beams and to the right the reference beam during testing.

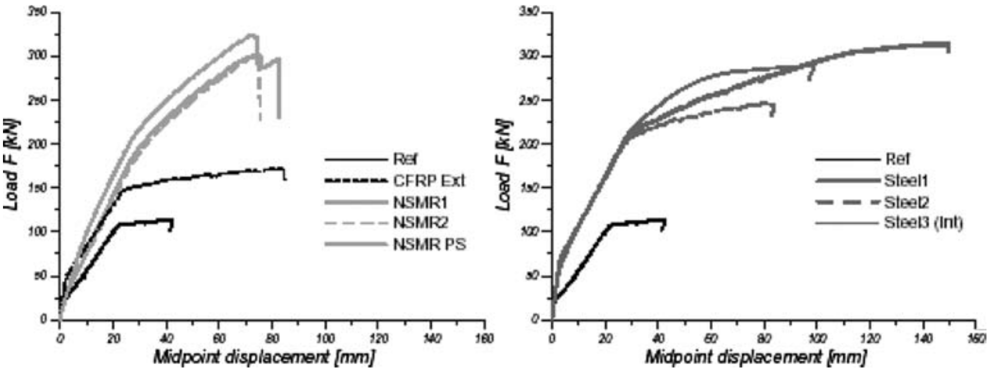

Figure 2. Load-displacement for the tested beams strengthened with CFRP (left) and steel (right).

unbonded tendons. The problems with the CFRP anchor during prestress continued during loading and the loads were much lower for that beam then predicted. In figure 2 the loads and displacements are shown.

The tests show a large increase in crack and steel yielding loads. The increase in load for steel yielding can be very important for a constructions life, the fatigue behaviour will improve and as a consequence the crack widths will be smaller which can result in increased durability. Together with higher crack loads the cracks also go smaller, this should also indicate a more advantageous behaviour in the service limit state (SLS).

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Applicability of welding for repair/reinforcement of overage bridges

Y.-C. Kim
Joining and Welding Research Institute, Osaka University, Osaka, Japan

H. Horikawa & M. Hirohata
Osaka University, Osaka, Japan

1 INTRODUCTION

In constructing steel bridges, although welding is 100% performed for joining in factories, bolts are generally used in the sites. As repair and reinforcement on bridges should be performed in the sites, bolts are mainly used for joining of members. It is necessary that the joining method in repair and reinforcement can be selected in the right way in the right place.

In this paper, a series of experiments, in which steel used for bridges constructed in around 1930 is welded using a general filler wire, are carried out. It is examined whether welding can be performed on steel used for a long term or not. Then, the mechanical characteristics of the welded joints are investigated.

2 EXPERIMENTS

Materials for experiment are the steels used for the railway bridge constructed in 1931 and the one used for the highway bridge constructed in 1927.

The steel used for the railway bridge is called as Steel R1 and the one used for the highway bridge is called as Steel R2 here. SM400A, which is the same grade as them, is selected as the criterion based on the obtained results of the tensile test.

The chemical compositions are analyzed on Steel R1 and Steel R2.

MAG (Metal Active Gas) welding is performed using a generally used filler wire (**Table 1**). It is checked that various welding cracks occur or not.

Vicker's hardness test, a tensile test, a bending test and a fatigue test are carried out so as to elucidate the mechanical characteristics of the welded joints[1].

Charpy impact test is carried out so as to obtain Charpy absorbed energy and brittle fracture percentage of the weld metal (WM), HAZ and the base metal (BM). The absorbed energy, ${}_{v}E$, of sub-size specimen was converted into that in full-size by using the following equation[2].

$${}_{v}E = bh^2 \times {}_{v}E \text{ (sub-size)}$$

where, b: width (mm), h: height (mm)

3 EXPERIMENTAL RESULTS AND CONSIDERATION

3.1 *Evaluation of weldability*

Table 2 shows the results of the chemical compositions.

Steels used for a long term contain more sulfur (S) and phosphorus (P) comparing with SM400A because it is considered that the technique of desulphurization in those days was inferior to that at the present.

Table 1. Mechanical properties of filler metal.

Yield point (MPa)	Ultimate strength (MPa)	Elongation (%)	Absorbed energy (J)
470	560	30	120

Table 2. Results of chemical composition analysis.

	Chemical compositions ($\times 10^{-2}$) (%)												
	C	Si	Mn	P	S	Cu	Ni	Cr	V	Mo	B	Pcm	Ceq
R1	11.3	1.0	44.0	5.7	3.9	20.6	4.0	3.0	0.1	0.0	0.0	14.8	19.4
R2	20.7	1.0	52.0	3.5	4.3	7.0	3.0	1.6	0.0	0.0	0.0	23.8	29.8
SM400A	15.0	10.0	69.0	2.4	0.7	1.0	2.0	2.0	0.1	0.5	0.1	19.5	27.5

The sound welded joints can be obtained without defect or crack by using a general filler wire without dealing with special treatment like as pre- and/or post-heating although the occurrence of weld cracking was anticipated. The weldability of steels used for a long term is by no means inferior to that of SM400A.

3.2 *Evaluation of mechanical properties*

According to the results, remarkable softening/hardening in the weld metal (WM), heat affected zone (HAZ) and the base metal (BM) is not seen in steels used for a long term.

Steels used for a long term are fractured in all of the base metals and JIS (Japanese Industrial Standard) of SM400A is satisfied. It is found that steels used for a long term have enough tensile properties comparing with SM400A have.

All of specimens can be bended 180-degree without cracks or fissures. Steels used for a long term have enough bending properties.

It is reconfirmed from the results of the fatigue test that welding defects are not generated in the welded joints without pre- and/or post-heating in welding for steels for a long term.

The absorbed energy of steels used for a long term is the largest in WM, and in the order, in HAZ and in BM (Fig. 1). That is, the absorbed energy of the base metal is the smallest and it is found that the absorbed energy is remarkably small comparing with that of SM400A.

The brittle fracture percentage of steels used for a long term is remarkably high comparing with that of SM400A. It is over 80% in all temperatures.

When welding is performed on the steels used for a long term in this experiment, it is elucidated that BM itself should be much noted rather than WM and HAZ should be.

4 CONCLUSIONS

The obtained results are as follows.

(1) Steels manufactured in around 1930 contained more sulfur and phosphorus comparing with contemporary steels contain.
Welded by using a general filler wire:
(2) No defects or cracks occurred without dealing with special treatment like as pre- and/or post-heating.

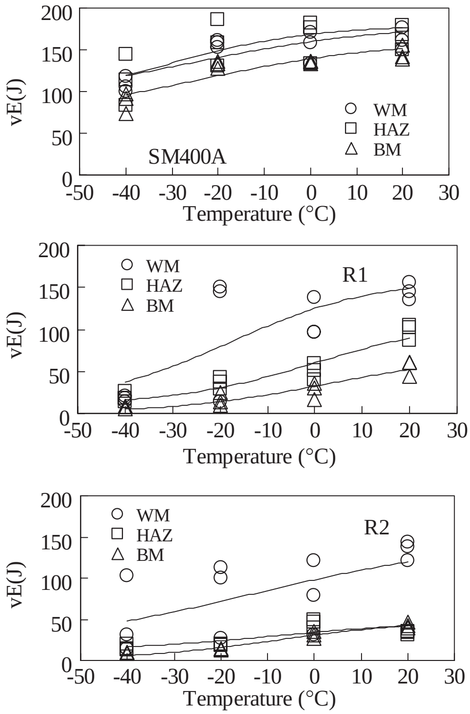

Figure 1. Charpy absorbed energy at test temperature.

(3) It was found that the welded joints with steels used for a long term had enough mechanical properties.
(4) It was reconfirmed from the results of fatigue test that no defects or cracks occurred without dealing with special treatment like as pre- and/or post-heating.
(5) Charpy absorbed energy ($_{V}E$) was the smallest in the base metal. It was elucidated that brittle fracture percentage of the base metal was extremely high comparing with that of SM400A.

Even if welding was used for joining in repair and reinforcement on the existing old steel bridges, there were no problems in the welded joints. It was rather necessary that the base metal should be much noted.

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Evaluation of reinforcement effect of deteriorated PSC beam through cutting its external tendons

B. Lee, C. Park & W. Lee
Korea Highway corporation, Hwasung, Korea

M. Kim
Sunkwunkwan University, Suwon, Korea

ABSTRACT: Recently, bridges for which maintenance needs to be mainly made are increasing rapidly on the expressways, due to the thickening of traffic, the increase of large-sized passing vehicles and the increase of the period for public use. The prestressed concrete (PSC) beam bridge, which is about 25% of the whole bridges on the expressways, is the bridge of a representative type which assumes the most proportions among bridge types.

In most of the bridges executed early in the construction of the expressways among the PSC beam bridges, various damages, such as the strength decrease and surface deterioration of concrete, and cracks due to the loss of prestress, occur.

Accordingly, various repair and retrofit methods are being applying for securing the stability of bridge and increasing its durability, and to enhance the load carrying capacity due to the performance enhancement or damage of bridge, the External Prestressing Strengthening Method, which introduces prestress from the outside using tendons, the attachment method, which attaches steel plate or fiber, such as carbon fiber, glass fiber and aramid fiber, etc among the methods, are being representatively used. The attachment type retrofit method, which attaches reinforcements on the lower part or side of the girder, for which the load carrying capacity lowers, and since reinforcements used in the method support not the effects of dead loads but only those of live loads, the efficiency of retrofit is impaired. Accordingly, for retrofitting the girder of PSC beam bridge, the external tendon retrofit method is widely being applied and is generalized. Since the external tendon retrofit method introduces prestress artificially from the outside and supports the effects of dead loads as well as those of live loads, the efficiency of retrofit is excellent. However, the characteristics of structural behavior due to retrofit is not considered enough at the design for retrofitting old bridge, and the quality control at the construction for retrofitting is not performed strictly.

In this paper, to analyze the behavioral characteristics and retrofit effects of the old PSC beam bridge to which the external tendon retrofit method was applied, loading tests and structural analyses for various cases were performed. The behavioral change of the bridge by the introduction of tensile force was evaluated reversely by cutting existing retrofitted external tendons by stages and measuring the deflection and strain of the bridge, and similarly the behavioral change of the bridge was evaluated reversely by performing loading tests before and after cutting tendons. Also, by analyzing the history of deflection and the natural frequency, the effects of external tendon retrofit were investigated.

The bridge for tests, which was completed in the 1970s, is two lane PSC beam bridge with the span length of 25 m and the width of 11.6 m. The girders of this bridge are five and it was found that the spacing between girders was not constant as 2.0 m $\sim$ 2.5 m, contrary to the girder spacing of 2.1 m in the plan. It was found from the results of the detailed safety evaluation that cracks occurred at the central part of the girder, and to secure the stability and durability of bridge, retrofit was performed by tensing tendons on the both sides of the girder.

The tensile force introduced at the time of retrofit construction was about 128 tonf, and the computed camber was designed to be 8 mm. However, at the time of the retrofit design of this

bridge, the whole bridge was not considered as a structural system, but design was made for the composite section of one PSC beam, the resisting section, and accordingly, the computed camber was predicted excessively.

Since for this bridge three years have passed after external tendon retrofit, it was evaluated that when considering the loss after retrofit, including the effect of concrete creep, about 14% of the tensile force introduced at the time of retrofit was vanished and at present the tensile force of about 110 tonf is being applied. It was assumed at the time of structural analysis that the same tensile force was being applied to all the external tendons.

To compare with the failure of external tendons and the results measured during the loading test, structural analysis was performed. For structural analysis, the general-purpose structural analysis program was employed, and the girder was modeled as a beam and the floor deck as a plate. Sections were modeled with dimensions measured actually. In structural analysis, the effect of external tendons cutting was considered by applying reversely the computed tensile forces, and static responses only were compared by loading statically loading vehicles in the loading test.

To investigate the retrofit effects by external tendons, external tendons on both sides of five girders were cut successively using oxyacetylene cutter. At the time of cutting tendons, the strain and deflection of the girder were measured by attaching strain gauge and extensometer. Also, to investigate the variation of the bridge structural system before and after cutting external tendons, static and dynamic loading tests were performed. At the time of the loading test, strain and deflection were measured as at the time of cutting external tendons.

In this study, the effects of retrofit were analyzed through the tendons failure test for the PSC beam bridge retrofitted by the external tendon retrofit method during long-term public service. The effects of external tendon retrofit were investigated by cutting the retrofit tendons arbitrarily and measuring the variation of the behavior of the bridge, and the results are as follows.

When one external tendons having the effective tensile force of 110 tonf is cut, the deflection in the central part of the retrofitted girder was 0.8 mm ~ 0.94 mm for the inner girder and 1.04 mm ~ 1.33 mm for the outer girder, and the stress variation in the lower part of the girder was evaluated as about 60 kgf/cm^2. This variation corresponds to about 2.4 times the stress occurred when 32.4 tonf load passes. Also, the stress measured in the upper flange of the girder increased greatly up to the level of the stress variation of the lower flange cutting tendons and then decreased with time.

From the result that structural analysis was performed by introducing the tensile force of the external tendon in the reverse order of the cutting process, the stress variation was similar to that from the measurement, and the maximum stress was 62.2 kgf/cm^2, which was nearly equivalent to the measured value, 60.1 kgf/cm^2. Accordingly, it was found that through appropriate structural analysis, the effects of the retrofit of external tendons can be predicted with enough accuracy.

In the case that the function of the support was lost because of its deterioration, the variation of stress for external forces was not large, but the variation of deflection was large. Accordingly, it was found that to evaluate the state of the bridge, structural analysis should be performed by considering enough the degree of the friction of the support.

It was shown from the test results for this bridge that when retrofitting external tendons, the frequency of the bridge did not vary by the cross section of the tendon, but that the natural frequency varied because of the effects due to the tensile force and the natural frequency increased about 5%.

It could be found that the state of the support should be well considered to evaluate the retrofit effects and stability of the bridge. Also, it is concluded that the natural frequency of the bridge can vary as the effect of structural retrofit by the external tendon retrofit method, and that the study on the frequency variation by the tensile force will be needed.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
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Some efficient solutions for bridge reconstruction

V. Popa
Search Corporation, Bucharest, Romania

1 INTRODUCTION

Sometimes, bridges have to be reconstructed because of the various causes as their age, great deteriorations, new operation conditions so on. In many cases, to reconstruct a bridge is more difficult than to construct a new one. Many solutions have to be analyzed in order to find the best one. That means to solve the reconstruction problem in the best quality conditions with the minimum cost. Such solutions are based sometimes on well inspired ideas that have just to be put in practice. The paper presents three cases of bridge reconstruction where interesting solutions based on excellent ideas were applied, so saving a lot of money.

2 PRESENTATION OF BRIDGE WORKS

2.1 *Case 1. Bridge over the Olt river at Stoienesti*

Olt is the biggest internal river in Romania after the Danube. The highway no. 6 has been crossed the Olt river at Stoenesti on a bridge with a total length of 261 m. The bridge superstructure consisted of a 240 m long deck having a continuous composite structure on six spans of 40.10 m each as well as a 20.60 m long deck with a simple supported composite structure. The hydro-energetical arrangement of the Olt river imposed the Stoenesti bridge reconstruction in order to assure navigation clearance. To reconstruct it, a new 550 m long bridge was proposed on a place located 100 m upstream to the existing one. The existing bridge had to be dismantled and demolished. The new bridge investment had a high cost.

Taking into consideration that the superstructure decks were constructed on the older substructure just ten years before the reconstruction, a new solution was finally applied. The new solution consisted in lifting the existing decks by hydraulic jacks, raising accordingly the existing substructure and extending the bridge with a new continuous composite structure on three spans of 60 m length each. The existing superstructure was lifted 1.08 m at one end and 8.39 m at the other one. The reconstructed bridge has a total length of 443.26 m (including the joint spaces). The new investment cost was half of the initial one.

2.2 *Case 2. Bridge over Arges river at Adunatii Copaceni*

Arges is also a big river in Romania. The navigation arrangement of this river imposed the bridge reconstruction at locality Adunatii Copaceni on the high road no. 5. The superstructure of the existing bridge consisted of a continuous deck on three spans of about 57 m length each. The bridge has also a new and adequate superstructure that was constructed ten years before the reconstruction on an older substructure.

The initial project of the bridge reconstruction proposed a new 230 m long bridge placed 20 m upstream, as well as dismantling and demolition of the existing one.

The idea to reuse the existing bridge deck on a new substructure was finally applied, so saving almost 30% of the previous cost.

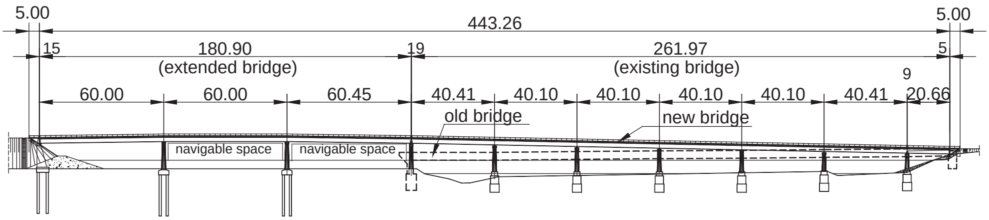

Figure 1. Bridge over the Olt river. General layout.

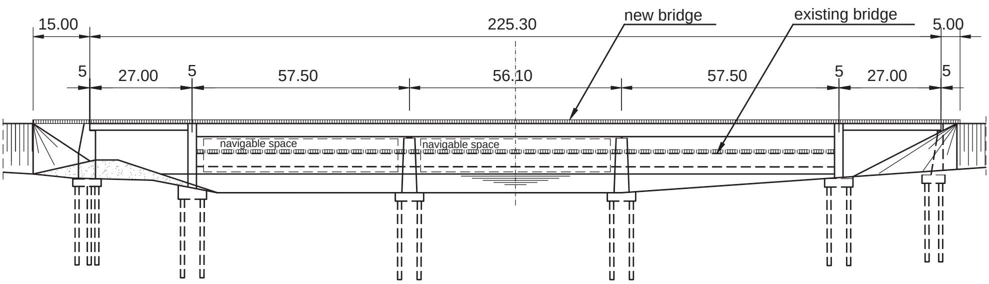

Figure 2. Bridge over the Arges river. General layout.

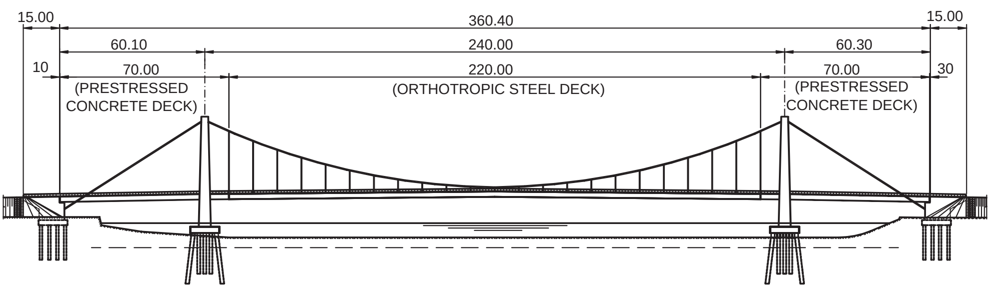

Figure 3. Bridge over the Little Danube. General layout.

The last bridge solution consisted in movement of the entire existing deck on a new place 20 m upstream. The entire deck had a mass of 3.600 to and a total length of 171 m. The new bridge was extended with two 27.00 m long spans at both ends.

2.3 *Case 3. Bridge over Little Danube (Gogosu Branch)*

Execution of the Hydro-energetical and Navigable System on the Danube at Ostrovul Mare (named also "The Iron Gates II") imposed construction of a suspension bridge over the Little Danube (named Gogosu branch). The initial project proposed a suspension bridge with three spans of 95 m + 170 m + 95 m. The superstructure consisted of an orthotropic steel deck and the substructure of two abutments at the ends and two pylons placed in the riverbed.

The Client decided to postpone the bridge construction because of the very high costs, especially due to the difficulties of the pile execution in the riverbed from the floating platforms.

The idea to extend the middle span to the length of 240 m instead of 170 m and to decrease the side spans to the length of 60 m instead of 95 m was finally applied. The piles were executed from the earth fillings instead of floating platforms. The deck structure was solved by using a hybrid structure:orthotropic steel structure in the middle span and pre-stressed concrete structures in the side spans. The cost decreased substantially (over 25%), so the Client decided to continue this investment.

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Mineral based bonding of CFRP to strengthen concrete structures

T. Blanksvärd & A. Carolin
Luleå university of technology, Luleå, Sweden

B. Täljsten
Luleå university of technology, Luleå, Sweden
Technical university of Denmark, Lyngby, Denmark

E. Rosell
Swedish Road Administration, Borlänge, Sweden

ABSTRACT: Strengthening of concrete structures with epoxy bonded carbon fiber reinforced polymers (CFRP) has been proved to be a good strengthening technique. However, this strengthening technique with epoxy adhesives do contain some disadvantages such as diffusion closeness, thermal incompatibility to the base concrete, working environment and minimum application temperature. Some of these drawbacks can be overcome by substituting the epoxy to a polymer reinforced mortar as the bonding agent. This work presents a pilot study with CFRP strengthened concrete beams. In this case the epoxy bonded CFRP has been replaced with a mineral based composite (MBC). The results from the pilot study indicates that the MBC strengthening system do achieve very good composite action and strengthening effects. These results warrant for further research and improvement of the MBC strengthening system.

1 INTRODUCTION

Large parts of the infrastructure in the world are in need of repair or strengthening. There are numerous reasons for this; higher traffic loads/flows, construction and design errors, the structure has reached its designed lifetime and so on. One way of strengthening a concrete structure is to apply carbon fiber reinforced polymers (CFRP) with an epoxy adhesive as a bonding agent in different ways. There are some disadvantages while using epoxy resins as a bonding agent, i.e. diffusion closeness, thermal compatibility, working environment and the minimum temperature of assemble. The two first aspects includes freeze and thaw problems, the third indicates the allergic reactions which can arise for the labors if not proper protective garment is used. The last aspect refers to the required minimum assembly temperature, which can be a problem in colder climates. It is therefore of interest to replace the epoxy adhesive with a mineral based bonding agent, e.g. polymer modified mortars with similar properties as the base concrete that also is more working environmental friendly. A combination between the polymer modified mortar and fiber reinforced polymers (FRP) can be used for repair and strengthening of civil structures.

Mineral Based Composites (MBC) is such a combination. MBC is a composite material which is made by replacing a part or all of the cement hydrate binder of conventional mortar or concrete with polymers and by strengthening the cement hydrate binder with polymers and with the addition of conventional FRP it becomes a high performance strengthening system.

This paper will investigate shear strengthening of RC beams with mineral based bonding agents and CFRP. Shear strengthening of concrete beams with carbon fiber composite and polymer modified cement mortars is a part of an ongoing research project at Luleå University of technology.

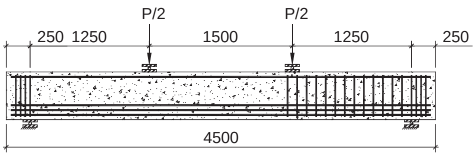

Figure 1. Reinforcement scheme and test set-up for concrete test specimens.

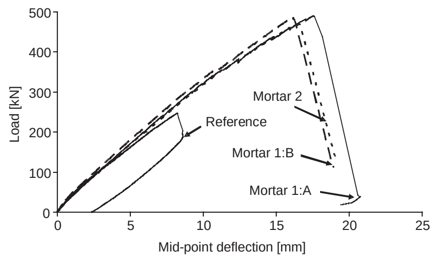

Figure 2. Load and midpoint deflection for strengthened and unstrengthened concrete beams.

2 MINERAL BASED COMPOSITES (MBC)

2.1 *Test set-up and materials*

Three concrete beams were tested and prepared with a mineral based strengthening system consisting of a CFRP grid and two different polymer modified mortars as bonding agents. The test set-up, reinforcement scheme and geometry of the concrete beams are shown in Figure 1. The concrete beams are reinforced in such a way that shear failure will occur at one known shear span. The mortars in this test program are mortars that have shown good bonding and strengthening capabilities based on the results from previous pilot studies. Both mortars are one component, cement based and polymer reinforced.

The CFRP grid is a two dimensional grid with a distance between the carbon fiber tows of 40 mm. Mechanical properties of the carbon fiber are 3800 MPa in tensile strength and a modulus of elasticity of 284 GPa. For the monitoring aspect the mid-point deflection, loading, horizontal and transverse strain in the CFRP grid and photometric strain measurement were recorded.

2.2 *Results*

Figure 2 shows the load and midpoint deflection for the unstrengthened and strengthened concrete beams. There are no large differences in stiffness and failure loads for the two different polymer modified mortars in the strengthening system. The behavior during the load steps was similar for all of the strengthened concrete beams.

All of the strengthened beams failure modes were the same with propagating shear cracks in the polymer modified mortar and finally a brittle failure and rapture of the vertical tows in the carbon fiber grid.

3 SUMMARY AND CONCLUSIONS

A significant shear strengthening effect was achieved by the presented and tested strengthening system. Good bond and composite action between the concrete beam and the strengthening system was obtained. The presented strengthening system was not complicated to install even though the possibility of spraying the mortar to mount the strengthening system was not evaluated in this study. The mineral based bonding agent did provide such good anchorage that fiber rupture in the used carbon fiber grid was achieved. The elongation at failure of the carbon fiber grid was 15‰. The monitoring aspect included both local strain measurement with traditional strain gauges and global measurement by photometric strain measurement. Readings from photometric measurements and from strain gauges were in accordance.

Ongoing tests are focusing at different parameters and theirs influence on the shear strength of strengthened beams. These studies and the results will be published in a near future.

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Repair of a historical stone masonry arch bridge

D.V. Oliveira & P.B. Lourenço
Universidade do Minho, Guimarães, Portugal

ABSTRACT: Portuguese masonry arch bridges were built throughout centuries, spanning from the Roman period to modern times, and thus representing an invaluable architectural and cultural heritage. With time, the deep change of loads for which bridges were initially built, the decay of the materials and the lack of maintenance have led to different states of damage, in many circumstances not compatible with their use or even their safety.

By their own nature, structures belonging to cultural heritage constructions present a set of specific features that effectively limit the application of modern codes. Instead, recommendations regarding adequate approaches to guide the intervention in architectural heritage, within a rational and scientific procedure and within a cultural context are available.

This paper describes the procedure adopted in the analysis and design of repair measures of a historical masonry arch bridge, carried out at Universidade do Minho. The bridge under study crosses over Vizela River, near Guimarães.

1 SURVEY AND DAMAGE PATTERN

The multi-span Negrelos Bridge is located close to Guimarães over the Vizela River. Although considered to be a Roman bridge, there are no available documents to clearly corroborate this hypothesis. The bridge has a flat roadway, supported by three semicircular granite stone masonry arches, with different free spans, as schematically represented in Figure 1. The bridge reaches a total length of approximately 30 m and has a roadway width of about 3.0 m. The central arch is supported by two massive piers, endowed with two triangular cutwaters at upstream and two rectangular cutwaters at downstream.

Fearing for the bridge safety, which was originated and supported by its visual aspect, the local authorities requested a complete survey on the bridge, as well as the definition of a set of remedial measures in order to restore safety, if necessary. The survey carried out has showed that the bridge presented a pronounced damage state, where damage was mostly characterized by:

- Extensive longitudinal cracking exhibited by the central arch, close to the downstream spandrel wall, clearly visible at the intrados.

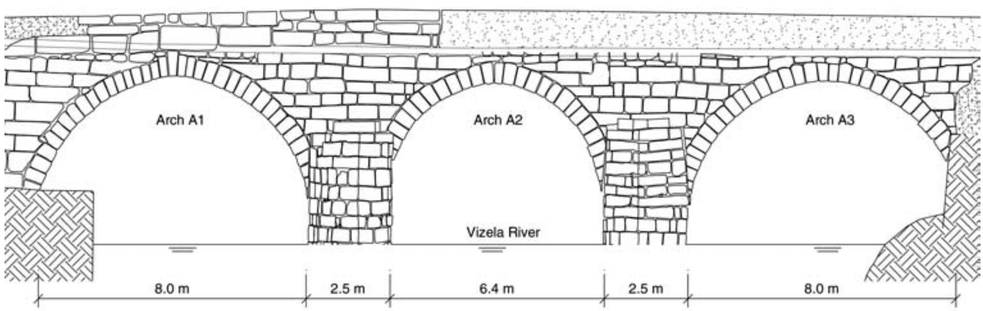

Figure 1. Negrelos Bridge (upstream view).

– Lateral movements of the spandrel walls near the left abutment.
– Generalized damage caused by vegetation, spread all over the bridge.
– Extensive cracking in the left downstream cutwater and minor cracking in the other three cutwaters, mainly due to existing vegetation and the lack of adequate stone imbrication.

2 CARRYING CAPACITY ASSESSMENT

Besides the necessary repair measures to be undertaken, also a numerical assessment in terms of carrying capacity was carried out to have a good estimation of the maximum load that the bridge can sustain prior to failure.

Among the available computational methods proposed in literature to compute the carrying capacity of masonry arch bridges, the rigid block computational limit analysis method is the most generally applicable. Limit analysis is a very practical computational tool since it only requires a reduced number of material parameters and it can provide a good insight into the failure pattern and limit load.

Negrelos Bridge was modeled as an in-plane three-span semicircular arch bridge and a flat pavement. The necessary geometrical data was obtained from topographic surveying and visual inspection. In the absence of in-situ test results, the material properties were considered to assume typical values found in similar structures. Besides the self-weight of the materials, a rolling load composed by the standard Portuguese vehicle was considered.

3 REPAIR MEASURES

The detailed visual inspection carried out showed that a set of repair actions were necessary, namely to stop the progression of the longitudinal cracking along the central arch, to counteract the outward movement of the spandrel walls, to prevent the failure of the cutwaters and to clean all vegetation from masonry. The historical and architectural importance of Negrelos Bridge forced that strengthening measures had to be designed in accordance with the principles that guide structural interventions in historical constructions. The paper describes a set of remedial measures designed and executed aiming at restoring the bridge safety.

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Study on the risk of scaffolding works exposed to strong wind

K. Ohdo
National Institute of Industrial Safety, Kiyose, Tokyo, Japan

1 INTRODUCTION

The weather has a major influence on work performed on construction sites, because construction work usually takes place in the open air. In the construction industry, scaffolds are frequently used, e.g. for bridge construction, or maintenance or demolition works. However, accidents involving workers falling from scaffolds or the scaffolds collapsing often happen when exposed to strong wind, leading to many injuries and fatalities.

Therefore, in this study, to prevent falling accidents from scaffolds, experiments were carried out to assess the risk associated with work assembling and dismantling scaffolds exposed to strong wind. Ten construction workers participated, and work assembling and dismantling scaffolds was carried out in a wind tunnel, exposed to uniform and gust flows respectively.

As for the prevention of scaffold collapse accidents triggered by strong wind, a design code for scaffolds subject to wind loads has been established and continually developed in Japan. The latest design code was instituted more than five years ago to prevent such fatal accidents, but despite the introduction of the new code, several accidents have still occurred. Therefore, a field measurement of wind loads acting on actual scaffolds that were 7 story – 6 bay frame structures was carried out. Based on the results of these experiments and measurements, the risk of scaffolding works exposed to strong wind was examined.

2 RISK ASSOCIATED WITH ASSEMBLING AND DISMANTLING SCAFFOLDS EXPOSED TO STRONG WIND

The risk of assembling and dismantling scaffolds exposed to strong wind were experimentally investigated by using a wind tunnel. The experiments were performed in two cases. One involved a uniform wind flow and another experiment involved gust flow. Ten subjects who were construction workers carried out scaffolding works, while exposed to uniform and gust flow, respectively. For each wind speed, the risk associated with the works were evaluated by the subjects, using 5 grades, ranging from '1: feel no risk' to '5: feel severe risk'. When the subject answered '5: feel severe risk', then their participation in the experiment was ended.

Based on these results obtained, the limits of wind speed for scaffolding works were proposed as shown in Figure 1. The results of previous study on the limits of the other works exposed to uniform flow were also plotted in Figure 1. Based on the latter, it was assumed that such scaffolding works have a higher risk than other works.

3 COLLAPSE RISK OF SCAFFOLDS TO WIND

In previous study, a series of wind tunnel experiments were conducted to measure the wind pressures acting on the scaffolds models.

However, the load transfer from the surface of the scaffolds to each structural member, especially the ties connecting the scaffolds to the building wall, was difficult to evaluate accurately during

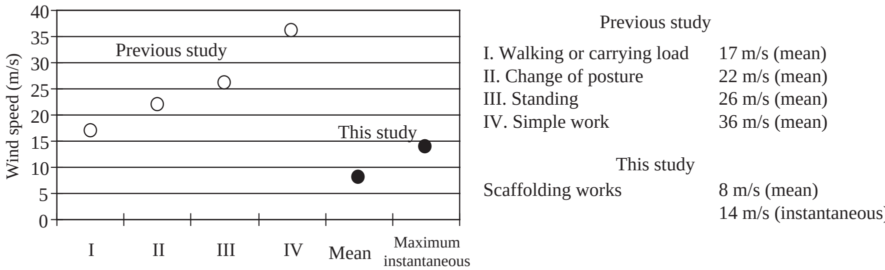
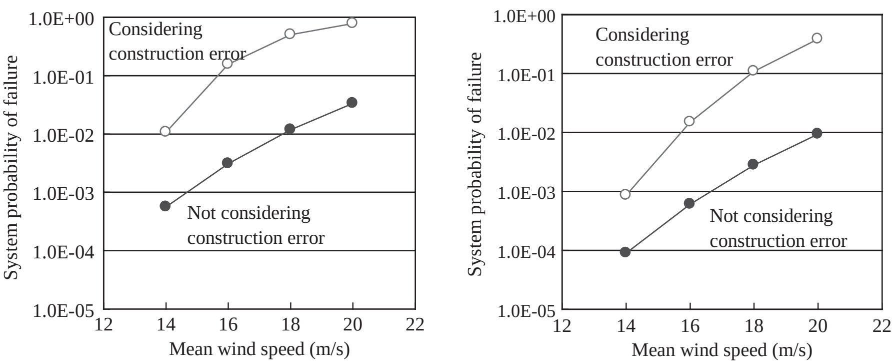


Figure 1. Proposal of wind speed limits for scaffolding works.



a. For the positive maximum.

b. For the negative maximum.

Figure 2. Relation between the mean wind speed and the system probability of failure of the scaffolds.

the wind tunnel experiments. Therefore, in this study, a field measurement of wind loads acting on actual scaffolds was carried out. The measured data of the wind loads acting on the ties were compared with the analyzed data obtained by FEM analysis. In consideration of the construction errors of the scaffolds, the reliability of the scaffolds exposed to strong wind was also analyzed in cases when the surface loads of the scaffolds were at their positive and negative maximums in the measurement. Based on the results of the reliability analysis, the relation between the mean wind speed and the system probability of failure of the scaffolds is shown in Figure 2. In this Figure, the result of the reliability analysis, which was not considered to be related to the construction errors, was also plotted.

From Figure 2, when considering the effect of the construction errors, the system probability of failure is over 10^1 times higher than that in the case of the effect is not considered at the same wind speed. Therefore, it can be concluded that the effect of the construction errors should be considered in the design code for the safety of scaffolds exposed to wind load.

4 CONCLUSION

From the results of this study, appropriate limits for scaffolding works exposed to strong wind were proposed, and the risk of the scaffolds collapsing under the wind load was evaluated.

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Strengthening steel bridges with new high modulus CFRP materials

M. Dawood, E. Sumner & S.H. Rizkalla
North Carolina State University, Raleigh, North Carolina, U.S.A.

D. Schnerch
Wiss, Janney, Elstner Associates, Inc. Cambridge, Massachusetts, U.S.A.

ABSTRACT: Due to corrosion and the continuous demand to increase traffic loads, there is a need for an efficient, cost-effective system which can be used for the repair and strengthening of steel highway bridge girders. Research has been conducted to investigate the use of carbon fiber reinforced polymer (CFRP) materials to address this need and the effectiveness of the technique for repair and strengthening of steel and steel-concrete composite bridge girders has been demonstrated. However, the majority of the previous research has focused on the use of conventional modulus CFRP materials for the repair and strengthening of steel bridge members. While substantial strength increases have been achieved, typically large amounts of strengthening are required to achieve a comparable increase of the elastic stiffness. This is due to the relatively low modulus of elasticity of the CFRP as compared to steel and also possibly due to the presence of shear-lag effects between the steel beam and the CFRP materials.

Recently, high modulus CFRP (HM CFRP) materials have become commercially available which have a modulus of elasticity approximately twice that of conventional steel. This paper presents the details and relevant findings of an experimental investigation which was conducted to investigate the behavior of a new HM CFRP system which can be used for the strengthening and repair of steel bridges. Additional details about the research program are available in Schncerch (2005) and Dawood (2005). Design guidelines are also presented which can be used for the design and analysis of steel-concrete composite girders strengthened with HM CFRP.

1 HM CFRP STRENGTHENING SYSTEM

Two types of unidirectional, pultruded CFRP laminates were used in the experimental program. The first laminate used an intermediate modulus carbon fiber (440 GPa), DIALEAD K63312, which is produced by Mitsubishi Chemical Functional Products, Inc. The second laminate used the higher modulus (640 GPa) DIALEAD K63712 carbon fibers. The CFRP laminates were bonded to the tension flange of the steel beams using a two part epoxy adhesive.

2 EXPERIMENTAL PROGRAM

The experimental program was conducted in three phases to establish the feasibility of various HM CFRP strengthening systems and to investigate the fundamental behavior of steel-concrete composite beams strengthened with HM CFRP materials. The first phase consisted of three beams that were tested to investigate the feasibility of three different configurations of CFRP strengthening systems. Two of the beams were strengthened with high modulus and intermediate modulus (IM) CFRP materials respectively while the third beam was strengthened with prestressed HM CFRP strips. In the second phase, a total of three beams were tested to investigate the behavior under overloading conditions. The third phase was designed to study the fatigue durability of the strengthening system. Two of these beams were strengthened with HM CFRP materials, while the third beam remained unstrengthened as a control beam for the fatigue study. All of the beams were tested in a four point bending configuration.

3 EXPERIMENTAL RESULTS

The results of the experimental program demonstrate that both the intermediate-modulus and the high-modulus strengthening systems resulted in a significant increase of the elastic stiffness and the ultimate capacity of the strengthened beams. Alternatively, the prestressed beam was designed to provide the maximum stiffness increase, without increasing the ultimate strength of the section. The use of the prestressed laminates helped to improve the efficiency of utilization of the strengthening system by reducing the amount of strengthening required to obtain a comparable increase in the elastic stiffness. The results of the overloading study further demonstrated that beams strengthened with HM CFRP materials exhibit lower residual deflections in the event of overloading conditions as compared to an unstrengthened beam. The presence of the HM CFRP also helped to reduce the stress in the tension flange of the steel beam, thereby increasing the yield load of the strengthened beams. The two strengthened beams that were tested in the fatigue study survived three million fatigue loading cycles with an increase of the load range of 20 percent as compared to an unstrengthened control beam. The improved performance of the strengthened beams indicates that it may be possible to increase the allowable live load level of a steel-concrete composite girder strengthened with HM CFRP materials.

4 PROPOSED DESIGN GUIDELINES

Based on the findings of this research program design guidelines have been developed which can be used by practitioners to design HM CFRP strengthening systems for steel-concrete composite beams. The design guidelines are based on a non-linear moment-curvature analysis which satisfies the conditions of equilibrium and compatibility. The guidelines follow the design philosophies presented in ACI 440.2R-02, but have been adopted for application to steel bridges.

Based on the moment-curvature analysis, the allowable increase of live load for a strengthened steel-concrete composite beam should be selected to satisfy three conditions. The three conditions which are proposed were selected to ensure the safety of the strengthened beam under ultimate loading conditions, maintain the fatigue durability of the strengthened bridge girders and prevent catastrophic failure in the event of unexpected loss of the strengthening system.

5 CONCLUSIONS

The findings of this research demonstrate that HM CFRP materials can be effectively used for the repair and strengthening of steel-concrete composite bridge girders. Based on these findings, a simplified design procedure was established to facilitate the safe design and analysis of steel-concrete composite bridge girders strengthened with HM CFRP materials. This program demonstrates that the use of HM CFRP materials represents an efficient, cost-effective alternative for repair and strengthening of steel bridges.

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Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Developments in FRP strengthening of railway bridges in the UK

B. Bell & B. Cox
Network Rail, United Kingdom

S. Luke & L. Canning
Mouchel Parkman, United Kingdom

N. Farmer & I. Smith
Tony Gee and Partners, United Kingdom

ABSTRACT: Network Rail owns over 40,000 bridges as part of the main line railway network in the UK. As part of the national highway bridge assessment programme, a number of these bridges have been identified as under strength and require strengthening or replacement. Network Rail has developed procedures and controls over the past five years for the safe implementation of FRP strengthening on the railway bridge stock, with advice from Mouchel Parkman and Tony Gee and Partners in joint venture. This has resulted in the successful strengthening of sixteen cast iron and reinforced concrete railway over line bridges and viaducts, and one reinforced concrete under line bridge, with FRP composites, using a variety of material forms, fabrication and installation methods. The paper discusses the key issues that have been identified as a result of this work, and presents case studies that highlight the design, certification and installation processes that have been used.

1 INTRODUCTION

Network Rail has actively stayed abreast of, and invested in, the developments in FRP strengthening technology as a possible method for strengthening of the UK railway bridgestock. In the late 1990's a preliminary business case, proposed to Network Rail, provided evidence of the technical and commercial advantages of FRP strengthening methods on the railway infrastructure. Initial strengthening schemes were undertaken on structures where a low level of strengthening was required. The lessons learned, advantages and disadvantages from these strengthening schemes are then discussed in this paper together with future FRP composite strengthening and new-build developments.

2 DISCUSSION OF FEASIBILITY, DESIGN, INSTALLATION AND MAINTENANCE PROCEDURES

The process by which structures were chosen by Network Rail engineers for initial investigation and feasibility by Network Rail's advisors was based upon the current engineering advice note, providing guidance on which structures may be suitable for FRP strengthening. These structures were forwarded to one of the FRP composites advisors for investigation and feasibility into the possibility of FRP strengthening. Typically, 50% of the forwarded structures were found not suitable after consideration of the overall condition of the structure and the economics of FRP strengthening versus other strengthening methods or re-construction.

Network Rail was then advised of the structures that were considered to be suitable for FRP strengthening. Depending on the particular circumstances of the structure, the preliminary design of the FRP strengthening scheme would then be commissioned, or in a small number of cases a further detailed feasibility study was commissioned.

2.1 *Design procedure*

Network Rail typically operates a two-stage design process for the majority of civil engineering schemes. The first stage comprises the preliminary design of the FRP strengthening scheme, providing the design parameters and identifying critical aspects of the design and a cost estimate, for the review, commenting and approval of Network Rail. The second stage comprises the detailed design of the FRP strengthening scheme and independent check of the design including all drawings, specifications and other contractual documents.

2.2 *Procurement and installation*

The FRP strengthening schemes were then procured via competitive tender, using experienced specialist sub-contractors recommended or approved by Network Rail's FRP composite advisors. The experience, performance, and capability of specialist sub-contractors were periodically reviewed to ensure that the standard of workmanship was maximized whilst retaining competitiveness. In particular, the tender returns were required to detail previous experience of the contractor and actual proposed site operatives in FRP strengthening methods. Extensive planning of the FRP strengthening and enabling works were then undertaken by Network Rail, the successful contractor, the designer and other interested parties. Critical enabling works and the FRP strengthening installation were undertaken with site supervision provided by experienced staff from the FRP strengthening designer. After completion of the strengthening scheme, the lessons learned during the process were then reviewed by Network Rail and its FRP composite advisors to enable continuous improvement.

2.3 *Inspection and maintenance*

The FRP strengthening schemes were each provided an inspection regime by the designer. Subsequent inspections were the undertaken and any change in condition of the FRP composite strengthening or structure itself reported (and therefore as-built drawings and other site records were of particular importance). Currently, no significant changes in condition have been observed for FRP strengthening schemes with a service life of up to 7 years.

3 CONCLUSIONS AND FUTURE DEVELOPMENTS

Network Rail has used a cautious incremental approach to the use of FRP strengthening to minimize risk to the railway infrastructure. This approach has been based upon:

- Internal engineering advice notes stating clearly the current limitations of the technique (e.g. level of strengthening, type of strengthening, type of substrate, type of structure), based on continuous review of research, technical literature and field experience. This advice has been updated at intervals to take advantage of advances in FRP strengthening methods and to reflect the experience gained on previous FRP strengthening schemes on Network Rail infrastructure.
- Appointing expert design consultancies to provide a first-stage feasibility of a structure for FRP strengthening (including site visits).
- Ensuring all design is undertaken by consultancies with extensive research, design and supervision experience, and rigorous checking is undertaken by an independent consultant with similar experience.
- Periodic dissemination and discussion of FRP strengthening schemes and developments with Network Rail engineers and contractors.

Recent progress in FRP composite technology has enabled Network Rail to consider the possibility of FRP composite materials for new-build, such as FRP composite bridge decks and FRP composite primary load-bearing members. The implementation of these technologies will be undertaken in a similar manner to that used for FRP strengthening, to maximize the benefit to the railway infrastructure in a controlled manner.

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FRP strengthening of masonry arches towards an enhanced behaviour

D.V. Oliveira, I. Basílio & P.B. Lourenço
Universidade do Minho, Guimarães, Portugal

ABSTRACT: As part of the widespread European cultural heritage, historical masonry constructions deserve particular attention. Being most of them of considerable architectural and cultural historical significance, their study and preservation constitute current issues in scientific research.

Among the materials used to repair or upgrade civil engineering structures, there has been an increasing interest devoted to the use of FRP (fiber-reinforced polymer) composites in the form of bonded surface reinforcements, which are being more and more used.

Following the initial researches concerning the use of FRP in masonry structures, numerous experimental works were carried out showing that this technique is effectively valid as an option to strengthen or repair masonry structures, in particular arched ones. On the other hand, available experimental results show that the strengthening of masonry arches with glass fibers, which exhibit lower mechanical properties than carbon ones, allow a better control of the collapse mechanisms and provide higher strength and better global ductility characteristics.

1 BEHAVIOUR AND FAILURE MECHANISMS OF MASONRY ARCHES

Assuming that masonry has zero tensile strength, which can be justified by its relatively low or even zero tensile strength, an arched masonry structure is kept in compression as long as the thrust line is kept inside the central core. When the thrust line moves outside the central core, at a given cross-section, the formation and consequent opening of a crack takes place. Safety is maintained as long as the thrust line is kept inside the thickness of the arch. The crack development leads to the formation of a plastic hinge at the compressed edge of the arch. Then, the formation of successive hinges leads to the formation of a mechanism that causes the arch failure. Typically, unstrengthened masonry arches fail essentially by the occurrence of plastic hinges enough to form a mechanism, see Figure 1a. As expected, the presence of a bonded FRP strengthening changes completely the structural behaviour of a masonry arch. The fibers, which possess a high tensile strength, prevent the afore-mentioned hinge formation and may change significantly the failure mechanism, by preventing the formation of a fourth hinge. This means that new failure mechanisms have to be considered.

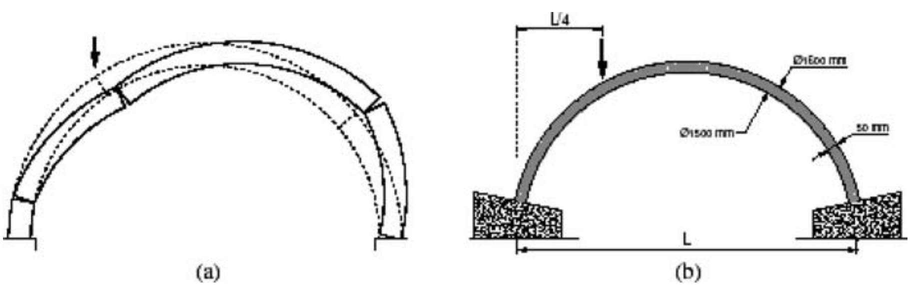

Figure 1. Semi-circular masonry arch: (a) typical four-hinge failure mechanism; (b) adopted arch geometry and load scheme.

This paper presents the first experimental results concerning the behaviour of brick masonry arches strengthened with glass composite materials, carried out within a combined experimental-numerical research project started at Universidade do Minho.

2 EXPERIMENTAL STUDY

All arch specimens were constructed at scale 1:2 in order to optimize expenses related to raw materials and workmanship as well as to achieve a quicker construction process and a feasible testing setup. In order to replicate old masonry constructions, handmade bricks and a suitable mortar were selected. Each semi-circular single-ring arch was composed of 59 brick courses and had a 750 mm radius, 500 mm width and 50 mm thickness, see Figure 1b. The mortar joints were kept with a constant intrados thickness of approximately 10 mm.

Two concrete blocks fixed to the laboratory rigid floor were used as supports, whereas the arches were constructed over a rigid wooden mould. One week after the construction, the mould was removed and the GFRP strips, if any, were bonded to the arch surface.

The first set of specimens was composed by two unstrengthened arches (US1 and US2). However, since both arches did not fall down at the end of the respective test, it was decided to use a localized strengthening arrangement composed of GFRP strips placed over the hinges at either the intrados or the extrados, and test them again (specimens LS1 and LS2). In addition, four undamaged arches were strengthened with continuous GFRP strips. Two arches were strengthened at the intrados (CSI1 and CSI2) and the other two were strengthened at the extrados (CSE1 and CSE2).

3 TEST RESULTS

Both unstrengthened arches US1 and US2 presented a similar structural behaviour, characterized by the formation of a classical four-hinge mechanism. An important feature is the low ductility exhibited by both specimens. Failure occurred suddenly, for small displacements and just after the maximum load has been reached.

The use of a strengthening strategy aiming at repair locally the damaged hinges did not avoid the formation of a four-hinge mechanism. The GFRP strips used were able to prevent the reopening of the existent cracks but new hinges appeared beyond the strip length instead. An average increase of the peak load in both arches in the order of 76% was achieved.

For the arches strengthened with continuous strips, a different collapse mechanism was expected. For specimens CSI, two of the hinges were formed at the supports and the third one appeared on the less-load half of the arch. On average terms, the load capacity was increased in 170% and the maximum load was achieved for a displacement of about 35 times greater than the one corresponding to the unstrengthened specimens. Failure, which occurred for high deformations, was dictated by the successive detachment of the two reinforcement strips, caused by the ripping of a thin layer of brick.

For specimens CSE, the first hinge was formed underneath the load point, whereas the other two hinges appeared at the supports. An important load increase was achieved (about 130% on average terms) comparatively with the unstrengthened specimens, however not as high as the increase enabled by specimens CSI. In this set, the maximum load capacity was achieved for a displacement approximately 20 times greater than the one corresponding to specimens US. A very important feature is the long post-peak branch recorded, which provides the structure with important ductility behaviour. For specimens CSE, failure was characterized by the slipping of one part of the arch with respect to the other along a mortar joint located close to the right springing.

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Fiber Reinforced Cementitious Matrix (FRCM)-advanced composite material and emerging technology for retrofitting concrete and masonry buildings

G. Mantegazza & A. Gatti
Ruredil Spa, San Donato Milanese, Milano, Italy

A. Barbieri
IUAV – University of Venice, Italy

ABSTRACT: Nowadays, the existing buildings need strengthening design due to different loading bearing capacity, serviceability and durability requirements.

These aspects are more evident in historical building constituted of masonry structural elements, which can only bear small tensile stresses.

Strengthening design could be more important for such buildings when the designer has to consider earthquake effects.

In Italy, seismic areas are numerous and the main historical monuments are placed in these areas.

Strengthening and rehabilitation of existing buildings, in particular historical building, is in daily practice, especially when the umbro-marchigiano earthquake provoked the collapse of masonry ribbed vaults of St. Francis Church, in Assisi (Italy).

Starting from this strengthening application the employment of composite materials started to be more widespread between designers.

Usually composite material means the combination of fibres of different type (glass, aramid and carbon) and epoxy resin FRP(fibre reinforced polymers).

This composite material offers a high ratio strength/density and the increment of mass due to strengthening application is insignificant compared to other methodology (steel or concrete elements installation).

Nevertheless, FRP (fibre reinforced polymers) material could lose the efficiency due to fire event and present defects due to moisture movement within the structural members.

A new advanced composite material is described in this paper.

A carbon fibre net is embedded in cementitious matrix; the strengthening system was called FRCM (fibre reinforced cementitious matrix).

The matrix was differed for masonry and for concrete substrate.

The carbon fibers carry the tensile stress whereas the cementitious matrix is more compatible to the substrate in terms of bond, moisture permeability and fire loading.

The paper concerns an experimental and numerical analysis carried on clay brick masonry panels strengthened with innovative materials, carbon fibre composites with cementitious matrix (FRCM). A series of masonry panels were loaded in diagonal compression to verify the effectiveness of proposed technology.

The ultimate shear stress τ_k, the failure mechanisms and the interface between support and reinforcement have been estimated by experimentation. The τ_k value is defined analytically from the experimental data elaboration for each specimen and for each strengthening typology.

The failure mechanism in strengthened panels is represented by a diagonal crack due to maximum tensile stress along the compressed diagonal trust.

The experimental results evidence a considerable increment of the τ_k which is dependent from the reinforcement configuration.

The considerable increments of maximum load, experimentally recorded, and τ_k, analytically obtained, are confirmed by modification of failure mechanism in strengthened specimens.

The strengthening system produces an increment in bearing capacity of masonry panels.

A design simulation is formulated using the mechanical data (τ_k) obtained from the experimental results carried out during testing analysis.

This simulation is formulated for a single masonry panel as for a masonry wall, loaded by vertical loading and horizontal one.

The experimental and design results are used to evaluate the efficiency of FRCM strengthening system.

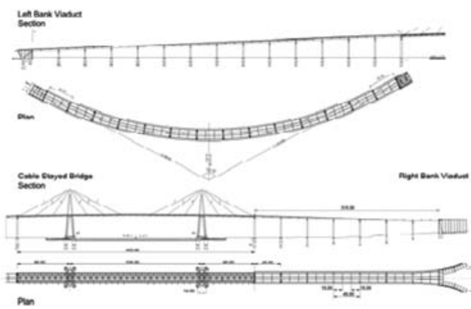
Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost –
Cruz, Frangopol & Neves (eds)

Rehabilitation of the Figueira da Foz Bridge

A. Rito
Proponte, Lda, Lisbon, Portugal

J. Appleton
A2P Consult, Lda, Lisbon, Portugal

The Figueira da Foz Bridge over the River Mondego (Fig. 1) has a total length of 1421 m, including a 405 m long cable stayed bridge and two approach viaducts with 630 m (left bank) and 315 m (right bank). The bridge was designed by Prof. Edgar Cardoso and built in 1982. The bridge was subjected to a general rehabilitation and strengthening.



A detailed inspection and assessment of the safety of the structures according to the new codes showed that the bridge and the viaduct had suffered significant deterioration and that a seismic strengthening was also required.

The rehabilitation of such a construction requires proper planning and the introduction of working platforms which represent a significant cost of the works.

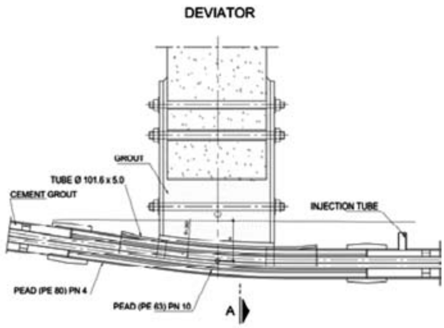
For the cable stayed bridge the main interventions were the following:

a) Strengthening of the transverse top beam of the towers for the seismic action by adding an external prestressing system;

b) Replacing and strengthening the anchorage system of the deck to the transition piers. The initial conceptual design led to the transfer of tension forces from the deck to the transition pier which was guaranteed by prestressing bars;
c) General rehabilitation of the structure, including the saddles, the repairing and addition of a new full protection of the steel elements as well as the local repairing and concrete surface protection of the masts.

For the approach viaducts the main interventions were the following:

a) Strengthening of the main girders by external prestressing. This work was due to the fact that the live load adopted in the design was lower than the recent code values as well as to the damages observed in the beams (cracking, concrete defects).



b) Introduction of new dissipating devices between the deck and the abutments. To introduce the viscous dampers (one in each of the four beams) the structure of the abutment had to be modified to accommodate the dampers and new sliding bearings were introduced between the girders and the abutment.
c) General rehabilitation of the concrete structures, including local repairing and a concrete surface protection.

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Multi-stepwise thermal prestressing method for strengthening of concrete structures

S.-H. Kim
School of Civil and Environmental Engineering, Yonsei University, Seoul, Korea

J.-H. Kim, J.-H. Ahn, M.-S. Jang, K.-M. Kim
Department of Civil Engineering, Yonsei University, Seoul, Korea

S.-S. Han
Wheesun Construction Co. Ltd., Seoul, Korea

ABSTRACT: Various prestressing methods have been proposed for concrete structures to improve their behavior characteristics. Effective as it is for concrete structures however, the applicability of general prestressing methods is generally limited to certain types of structures, namely newly constructed concrete bridges with sufficient cross-section for placing tendons.

Multi-stepwise Thermal Prestressing Method(TPSM) has been developed as a versatile prestressing method, which can be applied to various types of existing or newly constructed concrete structures, with a straightforward concept and a simple construction procedure. The TPSM utilizes thermal expansion and contraction of the steel prestressing plate due to temporary heat sources. The general sequence of construction using multi-stepwise TPSM is as in Figure 1.

The TPSM has some distinctive advantages that differentiate the method from other general prestressing methods. The main advantage of the multi-stepwise TPSM is that it is applicable to

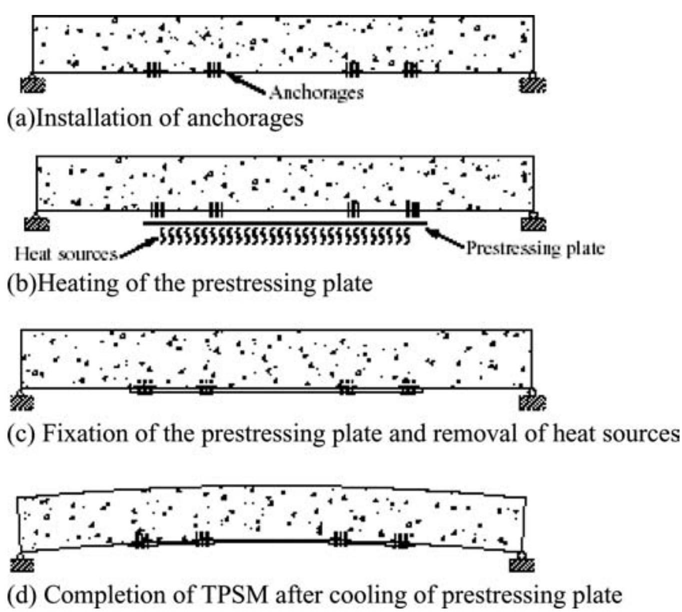

Figure 1. General sequence of construction using multi-stepwise TPSM.

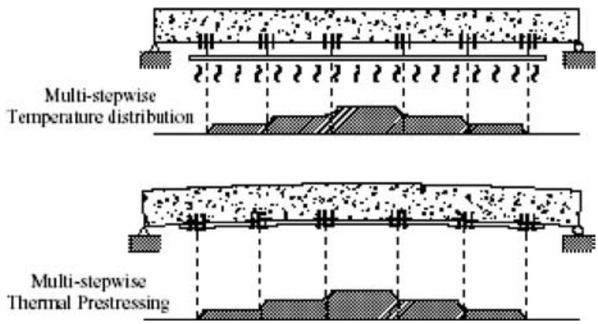

Figure 2. Multi-stepwise TPSM.

various types of structures for various purposes. The field of application range from strengthening of girder bridges to continuation of simply supported girders, and can be applied to existing structures as well as newly constructed structures. The method can also be applied to concrete structures regardless of the sectional shapes. The method is believed to have superior cost effectiveness over the general prestressing methods regarding construction of new concrete bridges. Also, the prestressing force can be dissipated over the anchorages so that there is far less stress concentration as illustrated in Figure 2.

Another advantage of the thermal prestressing method is that the prestressing plate, which is a prestressing member equivalent to the tendons, acts as a structural member and contributes to the structural stiffness. TPSM can be applied to continuous bridges without restriction to the number of spans, whereas prestressing methods using tendons are generally limited to number of spans no more than three due to the friction loss. Thermal prestressing method can also be applied to continuation of concrete bridges, by inducing initial stresses at the joined sections.

This paper describes the basic concepts of multi-stepwise TPSM and its field of application with some illustrative experimental results. The feasibility of the multi-stepwise TPSM is verified by applying the method to a RC girder specimen. Results from the experiments show that the TPSM is an effective prestressing method for improving the behavior characteristics of concrete structures and may serve as a feasible strengthening method.

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Reinforcement and protection of the Tâmega Railway Bridge

F. Martins
REFER – Rede Ferroviária Nacional, EP, Lisbon, Portugal

ABSTRACT: The Tâmega Railway Bridge is an extraordinary engineering feat in stone masonry, which was built last century, more precisely in 1934, being one of the last type of these bridges to be built in Portugal. It is places over the Tâmega River at PK 57.384 of the Douro Line, between the Livração and Marco de Canaveses stations.

Recently this bridge suffered and intervention aimed at protecting the foundations of its piers and its enhancement under depthness, where, according to the hydraulic studies that were carried out, the maximum excavation point of the river bed will have taken place.

This present article intends to describe the various stages of this project.

1 INTRODUCTION/GENERAL DESCRIPTION

The Tâmega River represents a right line configuration of the bridge. The place is influenced by the regulfing curve created by the Torrão dam's lagoon, sited about 17 Km downstream of the Bridge.

At the area of this work of art, the Tâmega valley has a V shape, showing the watercourse a width of about 100 m.

The Tâmega Bridge is a 60 m high stone masonry structure, 313.22 m in total length, having its plant a rectilineous shape. It consists of 11 arches, with 3 central arches 15 m in radius, and the remaining 8 arches 9 m in radius. Two abutments, also in stone masonry, limit the work of art on both riversides.

The bridge's 11 arches are set on 10 stone masonry piers, which have all efflux towards the base (variable section), with variable inclinations of 1/30 to 1/40.

Piers P3, P4, P5 and P6 have their foundations under water and have breakwater that extend all the way to the foundation. They have a variable height, of about 37 m in pier P5, up to the foundation – understood as the beginning of the poorest masonry (stone masonry massifs and "stakes" and other wooden elements) "fit" into the "earthly" massif at the bottom of the riverbed – and almost 47 m up to the base of this foundation – the aforementioned rocky horizon; the heights are shorter in the other piers, which are the object of this intervention (about 30 to 35 m for P3, 36 to 40 m for P4 and 26 to 33 m for P6). These foundations are encased by ripraps of significant dimensions.

2 SCOPE OF THE STUDY AND OF THE PROJECT

Before the construction of the Torrão Dam, the Tâmega Railway Bridge may have suffered erosion mechanisms on its riverbed, being its present development totally conditioned by the downstream presence of this Dam. The lagoon creates water levels with greater draining levels than the ones that existed before the construction of the Dam, thus causing the decrease of the draining speeds and the consequent deposit of sediment, a situation that might be "masking" eventual deficiencies in the foundations – hollows and cavities.

Parallel to this situation, the Bridge manifested signs of structural damages, singularly observed on pier P3, namely a vertical fissure in its downstream mantle, with a 23.5 m development, which created concerns about its safety.

In this way, REFER being aware of this problem, undertook a detailed study of the foundations, and later carried out the respective reinforcement and protection work.

The carried out studies, of both inspection and diagnosis, about the constructive methods to be applied, consisted, not only on foundation consolidation and protection work on piers P3, P4, P5 and P6, but also in the structural reinforcement of pier P3. This specific project on Pier P3 consisted on a reinforcement solution via trussing it with 6 pre-enforced stacks of reinforced concrete, being the consolidation of the foundations preceded by this reinforcement work.

Though the project developed rehabilitation and structural measures, it cannot be seen as a global rehabilitation project of the art work itself, as the study did not bear in mind the superstructure.

3 STUDIES CARRIED OUT/DIAGNOSIS

As a starting point for the studies to be carried out, a very carefully well-drawn design from 1934 was laid out, where the dimensions of the structural elements, as well as the conditions of the structural conditions was indicated – granitic rock, which allowed to fundated directly via shoes of a great dimension (10 × 15 m).

Next, and to undertake these studies, the following field activities were laid out: a) inspection work; b) initial survey of the damages and their causes; c) batimetrical survey of the riverbed immediately Downstream and Upstream of the Bridge and along its axis; and d) hydrological and Hydraulic study.

It was confirmed that Pier P3 had serious problems, namely a vertical fissure 20,0 m above the breakwater on the downstream side, which was moving towards the interior of the underwater buttress, which was found to be degraded and detached from the pier.

Pier P4 showed a sign of deshoeing on the downstream (Southern) side, which was only possible to confirm after the strong local sedimentation was removed. Another sign converged with the yielding mechanism on that side, as the breakwater was partially disintegrated on the Northern-downstream end, with a broken vertex.

Infra-excavations were also detected on piers P3, P4 and P6, which resulted from the hydrological regime of the Tâmega River – very erosive before the construction of the Torrão Dam and a lot less now.

4 DESCRIPTION OF THE TECHNICAL SOLUTION

All the reinforcement and protection work were conditioned and penalized by the levels of the Tâmega River, which depended on the exploration of the lagoon of the Torrão dam by the REN – Rede Eléctrica Nacional (National Electrical Network) company. Thus, the work plan had to bear in mind the water levels between the 56.50 and 59.80 m heights.

In this way, the work consisted of carrying out filling and consolidation injections to the piers, to the respective foundation massifs and the foundation terrains, ending not only any possible gaps or cavities, but also reinforcing the resistance characteristics of the foundation grounds.

The constructive methods that were used on the Tâmega bridge derived from the requirement found in the development of the project, to previously inspect and consolidate the foundation massifs of the piers, before carrying out the placement of the riprap prisms.

It was thus a well defined sequence of activities adjusted to the need to know, step-by-step, the full development of the task, in order to guarantee it was properly done.

To do this, the reinforcement and protection work was started on piers P3 and P4. On the former it was begun by sealing and re-closing the fissure and taking care of the passive sole bars, and only after this was done, the trussing beams were put in place. Pier P4 was started by having its foundation massif treated.

5 CONCLUSIONS

This intervention was always followed and monitored by a monitoring program, consisting of systematically carrying out topographical and surveys and precision levelling, through markers placed on the Bridge's crown blocks, two in the area of each abutment and another two in the section of each pier. These serves to control possible vertical and horizontal slides that might occur.

Bridge Maintenance, Safety, Management, Life-Cycle Performance and Cost contains the lectures and papers presented at IABMAS'06, the Third International Conference of the International Association for Bridge Maintenance and Safety (IABMAS), held in Porto, Portugal from July 16 to 19, 2006.

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