BRIDGE MAINTENANCE, SAFETY, MANAGEMENT AND LIFE-CYCLE OPTIMIZATION

PROCEEDINGS OF THE FIFTH INTERNATIONAL CONFERENCE ON BRIDGE MAINTENANCE, SAFETY AND MANAGEMENT, PHILADELPHIA, PENNSYLVANIA, USA, 11–15 JULY 2010

Bridge Maintenance, Safety, Management and Life-Cycle Optimization

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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 and serviceability of existing bridges by making better use of available resources is a major concern for bridge management. Internationally, the bridge engineering profession continues to take positive steps towards developing more comprehensive bridge management systems. Consequently, it continues to be appropriate to bring together all of the very best work that has been done in the field of bridge maintenance, safety, management and life-cycle optimization at the Fifth International Conference on Bridge Maintenance, Safety and Management (IABMAS2010), held in Philadelphia, Pennsylvania, USA, from July 11 through 15, 2010. The First (IABMAS'02), Second (IABMAS'04), Third (IABMAS'06), and Fourth (IABMAS'08) International Conference on Bridge Maintenance, Safety and Management were held in Barcelona, Spain, July 14–17, 2002, Kyoto, Japan, October 18–22, 2004, Porto, Portugal, July 16–19, 2006, and Seoul, Korea, July 13–17, 2008, respectively.

The International Association for Bridge Maintenance and Safety (IABMAS, www.iabmas.org), which serves as the organizing association of IABMAS2010 in conjunction with Lehigh University's Advanced Technology for Large Structural Systems (ATLSS) Engineering Research Center, 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, 835 abstracts were received by the Conference Secretariat. Approximately 70% of them were selected for final publication as full-papers and presentation at the Conference within mini-symposia, special sessions, and general sessions. Compared to IABMAS'08 the total of number of papers scheduled for presentation has increased from 465 to 511.

IABMAS2010 covers all major aspects of bridge maintenance, safety, management and life-cycle optimization including advanced materials, ageing of bridges, assessment and evaluation, bridge codes, bridge diagnostics, bridge management systems, bridge security, composites, design for durability, deterioration modeling, 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, informatics, whole-life costing, and multi-criteria optimization, among others.

Bridge Maintenance, Safety, Management and Life-Cycle Optimization contains the lectures and papers presented at IABMAS2010. It consists of a book of abstracts and a CD-ROM containing the full texts of the lectures and papers presented at IABMAS2010, including the T.Y. Lin Lecture, nine Keynote Lectures and 501 technical papers from 35 countries. This set provides both and 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 and Lehigh University's ATLSS Engineering Research Center, 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 Conference Honorary Chair, Professor John W. Fisher; to the members of the Conference Scientific and Organizing Committees for their dedicated work; and to the members of the Local Organizing Committee for the time and effort they have devoted to making IABMAS2010 a successful event. Finally, we would like to register our sincere thanks to all the sponsors of IABMAS2010.

Dan M. Frangopol and Richard Sause
Chairs, IABMAS2010
Bethlehem, Pennsylvania, USA, April 2010

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MS1: Futuristic bridge maintenance technologies, *organized by C.-B. Yun & B.F. Spencer Jr.*
MS2: Monitoring & assessment of bridges using novel techniques, *organized by A. Strauss & D.M. Frangopol*
MS3: Present & future of bridge inspection & evaluation, *organized by S. Alampalli, A.K. Agrawal & M. Ettouney*
MS4: Research & applications in bridge health monitoring, *organized by F.N. Catbas, J.R. Casas, H. Furuta & D.M. Frangopol*
MS5: New procedures for bridge rehabilitation, *organized by V. Popa*
MS6: Measurement systems for bridge weigh-in-motion (B-WIM), *organized by B. Bakht, A. Znidaric & D.K. McNeill*
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MS13: Management & maintenance of long span bridges, *organized by A. Chen & D.M. Frangopol*
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MS17: COWI Group mini-symposium – Cable supported bridges, *organized by J.S. Jensen*
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SS1: Use of health monitoring for life-cycle cost analysis & optimization, *organized by D.M. Frangopol & A. Orcesi*
SS2: Safety monitoring & maintenance strategy for long span bridges, *organized by A. Chen*
SS3: Bridge condition assessment, *organized by A. Miyamoto & F. Tondolo*
SS4: Bridges for high speed railways, *organized by R. Calcada*
SS5: Industrial smart material applications for civil infrastructure (ISMA), *organized by T.B. Messervey*
SS6: Advances in structural robustness: dependability framework, *organized by F. Bontempi*
SS7: Bridge adaptation to the environmental & climate changes, *organized by V. Popa*
SS8: Nondeterministic schemes for structural safety & reliability of bridges, *organized by S. Arangio*
SS10: Life cycle bridge engineering in Korea, *organized by H.-N. Cho & J.-S. Kong*
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SS18: Optical monitoring techniques for bridge maintenance & safety, *organized by S. Sumitro & H. Matsuda*
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T.Y. Lin Lecture

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Bridge maintenance and safety: A practitioner's view

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ABSTRACT

Maintenance engineering must be practical. Safety is always paramount in the maintenance operations of a bridge. OHSA provides rules for the health and safety of maintenance personnel, and is one of the most important bills ever passed by Congress. Today's bridge code is recognizing the need for designs that use durable materials that are long-lasting and can reduce maintenance demands. Bridge owners and maintenance engineers must ensure that maintenance money is effectively utilized for "good" maintenance. Let the bridge show you where it needs maintenance.

1 INTRODUCTION

If you ask me what I do, I would reply, "I work on bridges. I practice bridge engineering. I am a practitioner". Bridge Engineering covers many activities such as Planning, Designing, Constructing, Maintaining, and, finally, the Demolishing of Bridges.

I've done some designing, some construction, and some maintenance-engineering; I also have done some teaching, researching, serving on committees, and writing papers. I have been very fortunate to have worked in various capacities on many major long-span bridge projects such as the Golden Gate and the San Francisco-Oakland Bay Bridges, the Bridge of the Americas, the Lake Maracaibo Bridge, the new Cooper River Bridge, and, during the past few years, on several large bridge projects in China.

I began my 50 year plus career with the State of California Bridge Department (now called Caltrans) on the design and construction of six major Toll Bridges, that included the 1300-foot cable-stayed Southern Crossing of San Francisco Bay that, unfortunately, was never constructed.

I remember clearly that fortuitous day that I was assigned to the position of Chief Maintenance Engineer for all nine of the State owned Toll Bridges. I thought I was being side-tracked into a do-nothing position with no challenging work.

Was I ever wrong about this "do nothing position! I never experienced a dull moment in the five years that I worked in maintenance! Trucks collided with structural members; vehicles caught on fire, damaging the bridge structure; ships collided with piers and fenders; an over-height barge crane struck and buckled a major compression-member of a cantilever-truss; bearings and pinned joints froze up; expansion joints wore out and needed replacement under traffic! Of course, all these issues were in addition to the required day-to-day cleaning, painting, and inspection of all of the nine structures to keep them in first-class condition, as was required by the toll bond covenants backing the construction of these Toll Bridges.

I had all the maintenance money needed to perform these many functions, and I inherited a wonderfully trained team of maintenance workers and a small staff of registered engineers that were kept busy inspecting the bridges; designing repairs for damaged members and maintenance access-facilities; keeping the electrical systems fully functional; and responding to emergencies that always occur from time to time.

During my tenure in maintenance of these major bridges, Professor T. Y. Lin, who taught several of my classes while I was at Berkeley, offered me a position with his firm, T. Y. Lin International, which was and is a major consulting firm practicing bridge engineering in the Western Hemisphere and in Asia.

In 2004, I retired from T.Y. Lin International and started my own Bridge Consulting Engineering Office. I now consult on major bridges in China with Dr. Man-Chun Tang and on the new Self-Anchored Suspension span of the San Francisco-Oakland Bay Bridge.

Based upon this long career in bridge engineering, I have some thoughts that I would like to share with you about the maintenance and the safety of bridges.

2 MAINTENANCE MUST BE PRACTICAL

Scientists and mathematicians say "it must be beautiful" but maintenance engineers say "it must be practical". Maintenance engineers must be practical because they are responsible for drawing out the last bit of service life that a bridge has to offer. What they do to preserve and extend the life of the bridge must work and work well!

Without regular, professional maintenance engineering, the factor of safety built into the structure could be greatly reduced, the bridge posted for a load limit, and it might look unsafe to drive or walk across.

Most importantly, if the bridge were to be taken out of service, the highway system would lose an important investment of public money.

In emergency situations, maintenance engineers must act quickly, decisively, and wisely to protect the safety of the traveling public. Usually there is no time to look at textbooks, and, if there were time, there are few textbooks on the subject that will solve the immediate problem.

A good example of this occurred during the September 2009 "repair" of a fractured eyebar on the San Francisco-Oakland Bay Bridge. The installation was not actually a repair, but was a partial-strengthening of the cracked eyebar: the strengthening failed seven weeks after its installation. The bridge was closed for six days to all traffic while a second temporary, but improved, strengthening system was again installed. The permanent repair was then installed in December 2009 over eight days by working at night with partial deck closures.

3 MAINTENANCE AND SAFETY

Safety is always paramount in the maintenance operations of a bridge. Good maintenance includes the safe management of facilities such as traffic, roadway, aviation, and navigation lighting; keeping deck drains open; repair of expansion joints that spring loose; as well hundreds of other items. "Safety" also covers the safe management of accesses for maintenance personnel such as ladders, cat walks, and their personal gear such as coveralls, gloves, respirators, body harnesses and lanyards.

Today there are a number of personnel hoists for above-deck and below-deck access that aid in the safe placement of maintenance workers at the point of work. However, these facilities come at the price of one or two lane closures and the redirecting of traffic.

These personnel hoists are also used in the important function of inspecting all elements of the bridge for close-up inspection. These hoists do have their limitations, and sometimes inspectors need to climb steel, install temporary rigging, or use mountain-climbing equipment to get to all the inspection points. Whichever methods are used, "Safety First" is the primary mandate for inspecting a bridge.

Additionally, OHSA compliance for the health and safety of maintenance personnel was one of the most important directives for health and safety ever passed by Congress. OHSA has saved many lives and prevented many injuries over time. Although OHSA rules may be costly to implement and to self-enforce, and thus have drawn some criticism; this is money well spent. All maintenance operations performed on a bridge must conform to the requirements of OSHA, wherever possible. Sometimes, on existing bridges, OHSA rules cannot be fully implemented; in such cases the maintenance engineer must do whatever measures he/she can to increase worker safety, such as padding a header above a sub-standard height opening or posting signs warning of low clearances.

4 THE CHANGING MAINTENANCE SCENE

From time to time new materials are introduced to improve the maintenance of bridges. About 1975, some states and communities started to impose volatility limits on paint and solvents used in bridge maintenance; sandblasting was curtailed, and full containment of removed material was required to avoid sweeping it into bays or rivers. The limit on paint volatility was an opportunity which resulted in improved paints for structural steel. Today's paint systems have three times the life that they had when I started in maintenance with the traditional red lead paint system.

High-Performance Steel and Concrete require less material for construction, should last longer, reducing the cost of maintenance. Protective coatings for steel reinforcing bars in concrete decks and in splash-zone of concrete piers also extend the service lives of these areas.

Today's bridge codes are recognizing the need for designs that use durable materials that are long-lasting and that can reduce maintenance demands. A new buzz word, "Sustainable-Design" is being used to denote the use of these improved materials.

Bridge Diagnostic-Systems are being developed that make detection of bridge elements with structural problems easier to find, record, and maintain. Bridge Management Systems are also being developed that make routine and special maintenance easier to track.

Bridge Security is a new, developing technology for protecting important bridges that will require trial installations on bridges to determine how they may affect bridge-maintenance functions.

These few examples show that Bridge Maintenance can indeed be a changing, exciting profession!

5 THE ROLES OF THE BRIDGE OWNER

All bridges that have been built and all the bridges to be built in the future will have owners. What should be the role of the owners of bridges in regard to maintenance and safety operations? Perhaps, for a new bridge, their role would be to conceive of a beautiful bridge, thoughtfully designed, well constructed, and safely maintained. For existing bridges, the owners are limited to safely maintaining their bridges to obtain the built-in service life.

The design and construction of a bridge requires money up front, whereas good maintenance functions require money after the bridge has been built; sometimes that money, unfortunately, can be slow in coming, or worse yet, be cut-off. The owner must appreciate that good maintenance will prolong the life of the bridge, will provide safe passage for the public, and will require reserves of money that must be appropriated as needed.

6 THE ROLES OF THE BRIDGE DESIGNER

Good maintenance starts with good design. If the bridge designer does a worthy job of designing the structure and a thorough job of selecting proper materials; providing maintenance access when possible; and providing enough clear space for inspecting, cleaning, painting, and replacing – good maintenance should follow.

A bridge designer has many difficult roles to fill: he/she needs to satisfy a large number of requirements, such as those contained in a four-inch thick design code: choosing the proper structural type, span lengths, and foundation types; selecting the materials, bearings, and expansion joints; and writing specifications and estimating costs. Thus a designer may become too busy to think about the future maintenance of the bridge.

However, I believe the appropriate time to think about installing facilities for maintenance operations is during the design phase. These facilities will lower the life-cycle cost of the bridge; but only if the bridge designer is motivated to provide maintenance access and space on the bridge plans, and if the bridge owner will provide a little bit more money now to save more money in the future.

7 ROLES OF THE MAINTENANCE ENGINEER

The roles of the maintenance engineer are too numerous to mention here; suffice to say that the major roles of the maintenance engineer are: maintaining the safety of personnel, the safety of the travelling public, and the safety of the structures. These roles demand, among many other requirements, knowledge of structures and materials; some knowledge of construction practice, repair methods, and of arresting corrosion; familiarity with OHSA, safety devices and access equipment; ability to communicate with and to motivating personnel; and the ability to talk pleasantly to people who are heatedly complaining about pot holes in the bridge deck.

Where can bridge owners find good maintenance engineers? They usually are made the hard way from working on the maintenance of bridges, and if lucky, under the mentorship of a seasoned maintenance engineer. We never see advertising: "Enroll now, get your degree in bridge maintenance engineering, and make a fortune!"

In my case, I was appointed, kicking and screaming, into the maintenance engineering function. Luckily I inherited a good staff that were well trained and knew what they were doing before I came; they educated me very quickly as to what I was to do and how I could best help them to do their job!

However, other maintenance engineers may not be so lucky, and they may have to start almost from scratch in educating themselves and in training their own personnel; or worst, they have small staffs, little or no maintenance money, and must keep their bridges open with bailing wire and sheer determination.

Fortunately, this Association sponsors conferences, such as this one, that promotes and advances the art, practice, and development of Bridge Maintenance and Safety.

8 WHAT IS "GOOD" MAINTENANCE?

I think that politicians do not always understand why money must be spent on Maintenance. They may think: "You built the bridge, it is carrying traffic, and now you want to fix it. Didn't you do your work right the first time?" When money is short, maintenance is usually the first item cut; politicians refer to this process by the euphemism "deferred maintenance".

Although it is fair for the politicians to ask: "Is our money being well spent" or, "can this money be spent more effectively? It is up to bridge owners to provide, and to maintenance engineers to ensure, that maintenance money is effectively utilized for good and necessary maintenance.

Since the advent of the LRFD bridge design specifications, we have seen the probabilistic basis of this new approach to bridge design from publications displaying two bell-shaped curves, superimposed on a diagram, with the curve on the left representing the loads L (dead, truck, wind, others), and with the curve on the right representing the resistances R (of the bridge elements). A simplified interpretation of this diagram is that the distance between the peaks of the two curves is a measure of safety called Reliability Index. The acceptable value of the Reliability Index used for the design of bridges, is set by a specification-writing committee and is based upon their judgment and experience, and by comparison to the performances of existing bridges.

In the practical world, Politicians control L by their votes on legal-load weights for highways, hopefully guided by the advice of bridge engineers. However, politicians are constantly being lobbied to increase legal-load weights, which they often vote to do.

The resistance, R, of each of the myriad of bridge elements that make up a bridge, is controlled by the bridge designer, using the current, thick bridge design specifications and his/her knowledge and experience. After the bridge is constructed and opened to service, the resistance, R, is partially controlled – in one way of looking at it – by the bridge maintenance engineer, ensuring that the service life designed into the structure will be obtained through "good" maintenance.

The measure of "good maintenance" (and good design) is that the bridge will safely serve society, without reduction in load capacity, to the end of its design life.

9 SERVICE LIFE OF A BRIDGE

What is the lifetime of a bridge, or better yet, what is the service life of a bridge? "Service life" is the better description because it implies that the bridge will safely carry the loads, without reduction, for which it was designed, over its specified lifetime. An old bridge can live on after its service life has passed but it may require rehabilitation or extensive reconstruction.

At the beginning of my career, most bridges were designed for a life time of 50 years. We used slide-rules and the now-obsolete "allowable stress design' (ASD); the bridge design specifications were only about ¾ inches thick, and life was much simpler! That 50-year service life was increased, a couple of decades ago, to 75 years when the LFD method of design was developed. I have recently worked on oversight of the design and the construction of the Cooper River cable-stayed bridge which has a specified service life of 100 years. I am presently working in the same capacity on the bridge from Hong Kong to Macau, which has a specified life of 125 years.

There is certainly a trend toward increasing the design service-life of our bridges in the United States: a hundred year life has been suggested. At what point does a longer specified service life of the bridge trigger an increase in the Reliability Index or load factors, or in a reduction in fatigue stress? Do we know enough about the tertiary effects of aging on our materials? More importantly, what effects will longer service lives have on the maintenance and safety functions of our bridges?

Rivets have been around for about two centuries; though not used very often now; reinforced concrete began to be used in bridges a little over a century ago; prestressed concrete and welded steel girders have been used for a little over a half-century. Elastomeric and pot bearings and modular expansion joints have less than a half-century of use in bridges; high performance concrete and steel have an even shorter history of bridge-use – about a decade; yet even shorter is the use of advanced composites, which utilize plastic resins and fiber reinforcement.

If the century, or the century and a quarter of bridge service life is successfully to be achieved, perhaps we need a comprehensive test program for traditional materials being used for longer-life applications as well as for the new materials being used for longer life in traditional applications.

A disastrous example of not performing sufficient development and testing of new materials and new structural forms was the introduction of orthotropic steel decks to the United States, about forty years ago. The first-draft design specifications for orthotropic steel decks were based on strength-design, rather than on serviceability-design. In some of the early installations of this deck-type the steel began to fatigue-crack in high stress areas and had to be repaired in the field.

An even worse example was the choice of wearing surface material placed on the steel deck to provide skid resistance, a smooth ride, and to protect the steel deck from corroding. Asphalt, or modified asphalt was used with very little laboratory testing to prove its durability. All of the original asphalt-based materials failed in just a few years and now need to be replaced every decade or so.

Maintenance engineers had to stand by helplessly because these failures were beyond their control to manage or prevent; all they could do was watch, patch, and repair. However, there are now several installations of orthotropic decks and wearing surfaces that are nearing, or have achieved, a 30 to 40 year service life. These successful installations are typified by the use of engineered and laboratory-tested materials, as all of our materials should be, before being used on bridges.

10 PRIMARY, SECONDARY, AND TERTIARY EFFECTS

One of the jobs of the bridge designer is to determine the effects of the primary stresses generated by loads and structural action, and to use appropriate materials in the proper amount, to provide the necessary resistance to meet the load demands. There are now programs that have the ability to analyze secondary stresses, non-linear and inelastic structural actions, and even dynamic load-time-histories applications.

However, we still lack the ability to analyze for tertiary effects of time, loading, and the environment on material used in our bridges. For example, the breakdown of paint films under the aging effects of weather, oxygen, and moisture; the migration of chloride ions through concrete, that, when reaching the level of the steel reinforcing bars, starts corrosion; and the fatigue-effects of out-of-plane bending of steel plates. To be sure, specialists can do these things, but usually not typical bridge designers. We try to cover these adverse effects whenever they are discovered by code requirements that are based upon experience. However, tertiary effects are the very effects that the bridge maintenance engineer must inspect, monitor, and control, so as to provide a long service life for the bridge.

Although, at the time of this writing, no testing results have been published on the eyebars of the San Francisco-Oakland Bay Bridge, I believe the cause of the crack in the troubled eyebar will be found to be a tertiary effect that could not be calculated nor found by inspecting at the time that the bridge was designed, nor could the cause be found with today's technology.

These tertiary effects showed up very vividly in my experience on the Golden Gate Bridge, back in the 1970s. The Golden Gate Bridge, Highway, and Transportation District (The District) is a completely separate organization from the California Department of Transportation, (Caltrans), for whom I worked at that time.

The District had employed a consultant to evaluate the concrete deck; the consultant reported that the deck reinforcing bars were fatiguing under wheel loads and would begin fatigue-fracturing within a few years. The concrete deck could not be replaced under traffic. Therefore a new lower deck in the plane of the lower truss chords would need to be constructed, traffic diverted to the new lower deck, the upper concrete deck removed, and a new deck cast in place, all under full traffic. Obviously this would be a tremendously expensive and traffic-disrupting plan. The District asked Caltrans for a second opinion and I drew the assignment.

I read the report stating that corrosion had occurred between and on the top flanges of the longitudinal steel stringers, lifting the concrete slab free of its support from the flanges of the stringers. The deck was now spanning over one or two longitudinal stringers, and the extra-long spans were producing higher fatigue stresses under truck wheel loading.

I noted that the report used a beam analogy instead of a plate or arch analogy. I requested that the Caltrans Transportation Laboratory in Sacramento place strain gauges on a few of the reinforcing bars and monitor stresses during the morning commute. The strain gauge showed that the maximum stress recording was about 2000 psi. Even with an impact factor of 100 percent, this low stress would not be significant in terms of fatiguing the rebars.

However, our inspection did find that reinforcing bars near the soffit of the deck appeared to be corroding. We then performed half-cell readings on the deck and took two-inch diameter cores through the 6 1/2 inch thick deck, sliced the cores, and analyzed the slices for Chloride ions. The analysis showed quite clearly that the chloride content in the lower third of concrete in the deck was above the threshold content that sustains steel reinforcing bar corrosion. The deck was corroding, not fatiguing.

The chloride was being deposited on the soffit of the deck from the salt laden fogs that continually roll through the Golden Gate. The Golden Gate Bridge needed a new steel deck that could be constructed under traffic. The new steel orthotropic deck was opened in 1985 and is still performing well – but that is another story.

Both the corrosion of the top flange of the stringers and the intrusion of the Chloride ions were all tertiary effects that could not be calculated or predicted by the designers at that time. We can do this today, but with the exception of the cable-stayed Cooper River Bridge, we just don't do it. The conditions on the Golden Gate Bridge were made worse by the chief engineer stipulating that sandblasting was not to be used to clean the steel for paint application, as each blasting cycle removes some steel, and during the multi-century life of the structure, would remove too much of the steel sections.

The irony of this requirement is that the bridge has lost more steel section from corrosion than it ever would have from sandblasting. I remember many years ago talking to the paint superintendent of the San Francisco-Oakland Bay Bridge, who was very critical of the maintenance of the Golden Gate Bridge. He told me sandblasting was not allowed on that bridge, and as a result many rivets have lost their heads from corrosion, and that the lacing bars are sharp enough to shave by. I thought he was exaggerating, but when I was inspecting the deck, I did see rivets without heads, and lacing bars sharp as a razor.

But this will never happen again on the Golden Gate Bridge. The corroded rivets were replaced with high strength bolts and new lacing bars were installed. About 1970, and over a twenty-year period, the maintenance crews blasted off the old red lead paint, applied an inorganic zinc primer, and protected the primer with an overcoat of durable paint. The Bridge has not lost steel section since. It is now a model bridge for showing what good maintenance should be.

11 THE BRIDGE WILL SHOW YOU

The most important function for the maintenance and safety operations of a bridge is inspection, either by eye or by instruments. If you look, the bridge will show you where it needs maintenance help.

During the most famous bridge collapse of all-time, the Québec Bridge, under construction in 1906, was deflecting abnormally and some of the iron workers walked off the job. The now infamous I 35W truss bridge had bucked gusset plates before it collapsed in 2007. Several of the lifting cables of the old Dumbarton Bridge lift-span in San Francisco Bay were vibrating excessively and had to be replaced in 1975. The cable stays of the Luling Bridge, near New Orleans, were galloping abnormally and developing cracks in the plastic tubing and in the cement grout; the stays are now being replaced. And so on, as there are literally thousands of examples; however the most important observation is "what your bridge is telling you, if you look".

Of course there are distress items that are hidden from view. The flaw in the eyebar of the Silver Bridge across the Ohio River that failed and precipitated the total collapse of the suspension bridge in 1969 was a tertiary effect hidden from view.

But there are now ongoing efforts to develop detection instrumentation, data acquisition recorders, and transmission methods to find hidden distressed areas – the tertiary effects – but these effects are still what the bridge is telling you, but "looking" in a different and more effective way.

We all can look forward to the development of new technologies, and to the improvements of the ongoing technologies, which will assist us with inspection, maintenance, and safety work on our bridges.

12 SUMMING UP

I have had a wonderful career in bridge engineering; but my stint in bridge maintenance stands out as the high-light. I use that experience all the time in my current bridge consulting work, when I ask myself, "How can THAT be maintained?" I mentor younger engineers to acquaint them with, and to think about the maintenance functions of a bridge: Remember, if you can't access THAT for inspection and maintenance, THAT will not last as long as it should!

I have not listed any references, as these are my thoughts alone, based on my own experiences.

Keynote Lectures

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Fundamentals of suspension bridge retrofit

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ABSTRACT: The argument is made that when retrofitting a suspension bridge, the greatest value is obtained when the engineer considers the fundamentals of engineering, rather than simply following the code. By "fundamentals" two aspects are considered. One is a thorough understanding of not only structural behavior, but also how loads, resistances, and safety factors are derived. The other aspect covers the fundamentals of good design (as opposed to good analysis), in preparing designs that can be constructed during short occupancies and are tolerant of problems that might arise during the short construction period between deadlines. The design aspect includes looking for opportunities to reduce maintenance and improve inspection access, and produce such other benefits as may be desired.

1 INTRODUCTION

This paper reflects the author's opinions on what should be the fundamental considerations for the refurbishment or capacity increase of a suspension bridge. The reason for going to the fundamentals is that very often code requirements are inadequate to cover a particular situation, so it is important to understand how code provisions are derived, and therefore how they might be modified in a controlled way if modification is appropriate.

There are also fundamental considerations that the engineer should have in mind while he or she is designing the retrofit of a suspension bridge. These include:

- What am I trying to accomplish?
- How can disruption to traffic be minimized?
- What will be easiest for the constructor to build?
- What other benefits can be achieved, such as reduced and easier maintenance, or better access, or improved seismic resistance?

2 KEY DRIVERS

The key considerations driving the retrofit of a bridge are, as usual, benefits and costs. The purpose of this paper is to see how these are best managed for a suspension bridge.

2.1 *Benefits*

The main benefits are:

1. Improved traffic flow (more lanes, for example) which can also lead to increased tolls;
2. Increased safety, both in terms of how the traffic moves (perhaps wider lanes) and the ability to take increased loads. Sometimes the loads have increased, sometimes the bridge was designed for a loading much less than specified today, often it has simply deteriorated.
3. Improved access for inspection;
4. Easier and less costly maintenance; and
5. Other benefits that may be desired, such as improved seismic performance.

2.2 *Costs*

The four main cost items are:

1. The physical cost of upgrading;
2. The (often hidden) cost to the public caused by traffic disruption. To this can be added lost revenue to the operator where tolls are charged.
3. Over-strengthening by being conservative. Designing a new bridge for an increase in live load usually adds only a small percentage to the cost of the structure, but increasing the capacity of an existing bridge can be extremely costly. This is why unnecessary conservatism can be horrendously expensive. The larger the bridge, the more this is so.
4. Under-strengthening can be the most costly of all, as it can lead to failure of the bridge and even loss of life.

It can be seen that Costs 1 and 2 are related. The cheapest retrofit will probably coincide with the maximum traffic disruption. Since ultimately the user pays the price of retrofit, and also suffers the loss caused by disruption, an economist could in theory find a way to balance the two. In the author's experience, this is not usually attempted, and it may be that good judgment and intuition will provide close to the optimum solution. By the time a bridge is being retrofitted the traffic volume has usually built up to the capacity level of the bridge. Therefore only traffic disruption that occurs at off-peak times will normally be tolerated.

The best balance between Cost Items 1 and 2 can be found by careful and creative design, and this theme will be examined later in the paper.

It can also be seen that Cost Items 3 and 4 are related, in the sense that either cost will be over the optimum unless the narrow band is found between too safe and not safe enough. In other words: how safe is safe enough? That is the starting point for this presentation.

3 SAFETY

One can think of safety in several ways. At its most basic, safety is the margin between the capacity of the bridge to carry its loads, and the loads imposed upon it. We therefore need to know three things:

1. the capacity of the bridge;
2. the loading to be applied; and
3. the required degree of separation between the two (i.e. the safety factor).

It is true that the bridge design codes provide guidance on these matters, but if there is uncertainty, design codes err on the side of conservatism, or should do. And as mentioned earlier, conservatism may be a very expensive luxury, which can make it poor engineering. The above three aspects of safety will be considered in turn.

3.1 *The capacity of the bridge*

Modern analysis software and hardware should be able to provide a good idea of the capacity of a bridge, but experience has shown that this is not always the case.

Inspection of the bridge can detect any weak points, such as corroded steel, broken rivets and the like, but even the best inspection does not provide any information about the stress state of the bridge.

The shape of a suspension bridge is quite sensitive to any change in its condition. This characteristic is useful for what we call the "Analysis-and-Survey" method of determining the stress condition of the bridge. In this method, a computer model is created of the bridge as it was originally built, preferably from as-built drawings or from design drawings if as-builts are not available. The model is then altered to reflect any changes that have occurred since the bridge was constructed. These changes would include any additional dead load, structural changes, foundation settlement, and so on. Once these changes have been made, the geometry of the bridge will also have changed.

The bridge is then surveyed, preferably in the early morning before sunrise, with overcast sky, no wind, and no traffic on the bridge. The survey will include the profile of the deck from end to end, and the verticality in both directions of the towers. Once the computer model has been adjusted to account for the temperature at which the bridge has been surveyed, the shape of the bridge in the model should be the same as that in the computer.

In our company's examination of more than 20 suspension bridges, the two have never fully agreed! This

Figure 1. Traffic loading on the Blue Water International Bridge.

can only be because something has happened to the bridge that was not modeled because it was not known.

For example, when this technique was first applied to the Lions' Gate Bridge in Vancouver, BC, it was found that the centre of the bridge was about one meter lower than expected, and the towers were leaning in towards each other. One of the towers had settled a few centimeters, and some dead load had been added to the bridge, but these effects only accounted for about half of the observed discrepancies. The only explanation of the observed discrepancies was that the main cables, which comprised spiral strands, had stretched over time. Once this was accepted as the only rational explanation, these effects could be added to the others in the computer, and the true stress state was determined.

In this instance, a cable-bent (side tower) was found to be seriously overstressed, and remedial action was taken immediately. This deficiency could not have been discovered by inspection, but it was seriously reducing the capacity of the bridge.

3.2 *Applied loading*

Dead load can be determined from a weight take-off, and this may be modified by the findings of the analysis-and-survey.

The main unknown is the live load imposed by traffic. Various studies have provided some guidance on the subject, such as Buckland et al 1980 which provided the basis for the long span loading in both AASHTO-LRFD and the Canadian Highway Bridge Design Code CAN/CSA-S6-06.

An extreme case of heavy loads is shown in Figure 1.

Recent research by the Author's company has re-examined the multi-lane reduction factors, which are probably the least known part of the live loading for long spans. In one recent study it was confirmed that the multi-lane reduction factors vary with loaded length, being generally greater for long lengths than for short lengths, as shown in Table 1. This could add considerable complexity when trying to apply the design loading.

Table 1 shows that at long loaded lengths of about 500 m (1,600 ft.) the reduction factor for 8 lanes loaded

Table 1. Multi-lane reduction factors, recent study.

	Number of loaded lanes				
Loaded length, m	1	2	3	4	8
6	1.00	0.70	0.60	0.50	0.35
16	1.00	0.65	0.60	0.50	0.35
480	1.00	0.90	0.85	0.65	0.60

Table 2. Multi-lane reduction factors, AASHTO.

	Number of loaded lanes				
AASHTO	1	2	3	4	8
All lengths	1.00	1.00	0.90	0.75	0.75

is nearly double the measured value for 8 lanes of short loaded lengths up to about 20 m (70 ft.).

For comparison, Table 2 gives the reduction factors from AASHTO. (Interestingly, AASHTO-LRFD uses different reductions even though the traffic is identical.) It can be seen that the AASHTO factors are conservative, as they should be. But it can also be seen that using the AASHTO factors for short loaded lengths is so conservative that it could result in a large amount of very expensive strengthening that is in fact not needed.

The parts of a suspension bridge governed by long loaded lengths are the cables, towers and anchorages. These will be at a maximum when all lanes are loaded, so in the example given in Table 1 the multi-lane loading would be taken as 0.6 for these major components.

For some short loaded lengths, however, such as govern transverse floor beams or trusses spanning between cables, a multi-lane load factor of 0.6 would produce a design loading of nearly double the observed load, and a value of 0.35 would be more appropriate.

This does increase the complexity of the analysis, but if unnecessarily doubling the design load results in strengthening of all the transverse floor beams or trusses, the cost would be enormous. Compounding the problem, the extra strengthening may increase the weight of the suspended structure to the point that the cables, towers and/or anchorages may need upgrading.

This example makes a very compelling case for going back to the fundamentals!

4 INSTRUMENTATION

In a recent case, there was concern that the live load of the A. Murray MacKay Bridge in Halifax, NS, (Fig. 2) was being over-estimated by the design code.

The possibility of measuring traffic axle loads directly and doing a study as in the previous example was considered for the MacKay Bridge, but it was decided instead to instrument directly the members of most concern, which were the diagonals of the stiffening trusses as shown in Figure 3. Recording of data

Figure 2. A. Murray MacKay Bridge, Halifax, NS, Canada.

Figure 3. Stiffening truss, MacKay Bridge.

started at the end of December 2009, and will continue for about a year. This should indicate the combined effects of both single-lane loading and multi-lane presence, without identifying them separately.

5 SAFETY FACTORS

Having looked at the capacity of the bridge and determined the loads, the question arises as to what load (safety) factors to use. LRFD design has a huge advantage for suspension bridge analysis. The principle of LRFD (known as Limit States Design outside the US) is that safety factors are applied more rationally than they are with Working Stress Design. This is because a safety factor is really an ignorance factor: it covers what we do not know about a load or resistance. It therefore follows that if we can lessen our ignorance, we can lessen the safety factor that must be applied. Thus, loads that we know well, such as dead load, command a small load factor, and those loads that we know much less about, such as live loads or wind loads, are given larger load factors.

This concept is particularly useful for suspension bridges, which have different components governed by different types of loads. For example, the main cables are dominated by dead load, and only 10 to 30% of the total load that they carry is live load, whereas the stiffening truss or box girder is essentially unstressed

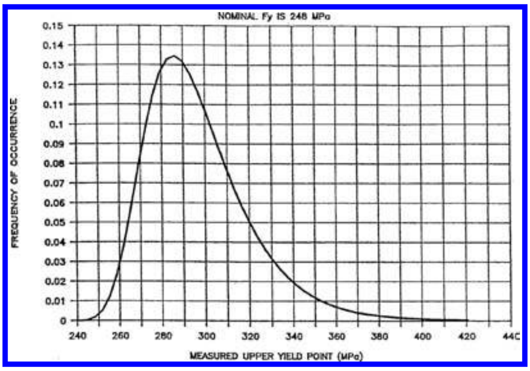

Figure 4. Distribution of measured values for yield values of A36 steel with a nominal yield of 248 MPa (36 ksi).

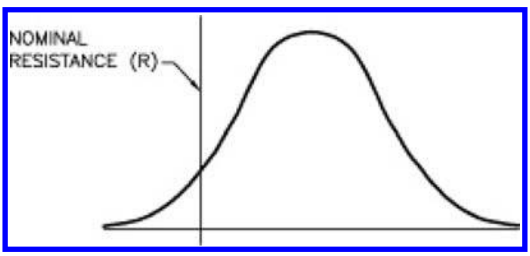

Figure 5. Stylized distribution of yield strengths.

under dead load only, so almost 100% of the applied load is live load.

Towers are a mixture: the axial load in the tower legs, coming from the cables, is almost all dead load, but the displacement of the tower-top is caused by live load. Thus if one thinks of the P-δ effect, P, the axial load, is governed by dead load, and δ, the displacement, is governed by live load.

5.1 *Loads and resistances*

Let us examine the concept of safety in slightly more depth, particularly as it relates to load factors.

Think first about the resistance or strength of a piece of steel or concrete. Figure 4 shows a distribution of measured values for the yield of ASTM-A36 steel, which has a nominal yield of 248 MPa. Clearly the actual yield strength of the material is not what is specified. It is usually greater, but it could, on rare occasions, be less.

Figure 5 shows a more stylized distribution of nominal strengths of steel or concrete.

Now consider the applied live load.

The bridge design code gives us a design live load, such as an HS vehicle or a lane load. The design load is usually not representative of any real vehicle, but it is intended to represent the *effects* of real vehicles. But how accurately do we really know the effects of real vehicles? And even if we do know them well, how confident are we that they will not change in the future? The answer is that usually the design load gives a greater effect than real vehicles, but sometimes

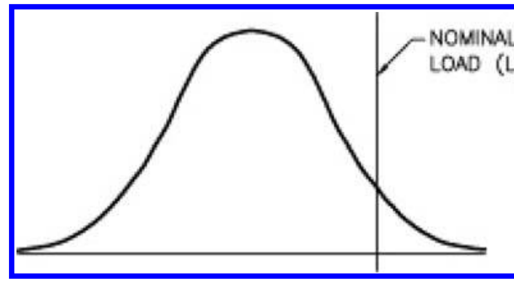

Figure 6. Distribution of load effects.

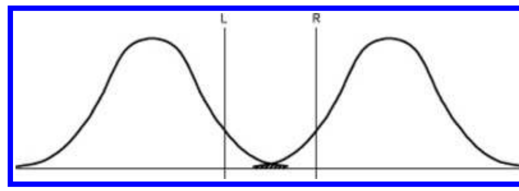

Figure 7. Distributions of load and resistance combined.

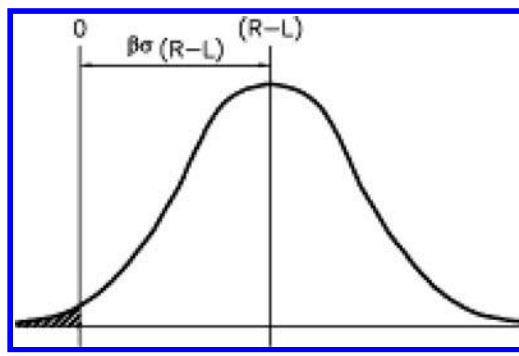

Figure 8. Distribution of (resistance minus load).

overloads occur, so conceptually we have a distribution of load effects as shown in Figure 6.

Now the two distributions of load and resistance can be combined, as shown in Figure 7.

The distance between the nominal load, L, and the nominal resistance, R, in Figure 7 is the safety factor, being the combined effects of load and resistance factors.

As long as the resistance is always greater than the load effect, all will be well, but it can be seen that the two curves overlap, which means that it is possible for the load effect to exceed the resistance, which leads to failure. It can also be seen that if the two curves are further separated, the area of overlap is reduced and the probability of failure is reduced; but it can not be reduced to zero if there are no known limits to the curves, and a quantitative value of the probability of failure is not easy to derive. If, however, we produce a distribution of R-L (resistance minus load), the curve of Figure 8 results.

In this case the measure of safety can be seen more clearly. The probability of failure is the area to the left of zero divided by the total area under the curve.

A measure of safety (as opposed to concentrating on the probability of failure) is often represented by β, the safety index. β is the number of standard deviations that the mean of the distribution is from zero. The greater β is, the greater the safety and the lower the probability of failure.

From this approach, once the target probability of failure has been set (or the β factor established) the required separation between the nominal (design) load and the nominal resistance can be defined. The separation is the combination of the load factor and the resistance factor, in general: the safety factor.

5.2 *Variable safety*

The issue of safety has been discussed in some detail so that we can introduce the concept of varying the load factors for different members.

Risk (perhaps the opposite of safety) can be defined as:

Risk = (Probability of an event occurring) × (Consequences of such an event)

Therefore to have consistent risk (or safety), the greater the consequences of an event, the lower the probability we should accept of the event occurring. Conversely, if the consequences are reduced, the acceptable probability can be greater.

Consider the tower of a suspension bridge. If it fails, the entire bridge collapses. Then consider a steel stringer that supports a steel or concrete deck. It will probably fail in bending, and the load will be shed to adjacent stringers, with the result that there is a temporary dip in the roadway until it is fixed. The two consequences are completely different, and it is clearly desirable to have less probability of tower failure than of stringer failure. In other words, the safety (load factors) for the two members should not be the same.

Properly considering these differing requirements can result in large cost savings during a retrofit. The subject is well covered in Section 14 of the Canadian Highway Bridge Design Code (2006) and its Commentary.

6 PRACTICAL CONSIDERATIONS

The discussion so far has been rather theoretical, although I maintain that a thorough understanding of safety is fundamental to safe and economical retrofit of any major bridge.

There are also, however, a number of practical matters to be considered.

6.1 *Bridge occupancy and traffic disruption*

There have been a few cases of suspension bridges being closed to traffic for months on end while renovations are made. These are rare, however, and it is much more likely that construction operations will be limited to only one or two lanes at a time, or only at night or weekends, or some combination of these.

Figure 9. Shims between new floor beam and existing girder.

The consequences of having short and/or restricted occupancies for construction are:

1. Any retrofit must be designed to be constructed during the short occupancies permitted, and shall be safe for the public between construction periods;
2. The design must be tolerant of problems that may be found during the construction occupancy. Generally these will be unexpected, or they would have been planned for, and they can include finding that the structure is not as was expected, breakdown of the contractor's equipment, errors in design or construction, and non-delivery to site of components or consumables, such as weld rod.
3. The designer's responsibility is to provide a design that is as tolerant of unexpected problems as possible. A simple example is providing shims in a connection so that if the existing steel is misaligned, the new steel can be leveled to the correct elevations, as shown in Figure 9. Here, the new transverse floor beam (framing in from the left) was to sit on the old longitudinal girder. Instead of trying to make the connection directly, a gap was provided, to be filled by steel shims. Thus if the elevations or alignments were not as expected, the shims could be adjusted accordingly.
4. The contractor's responsibility is to provide a higher level of management than for a typical project. If work is only to proceed at night, the contractor has two deadlines per day – one to be ready to start work at a certain time, and the other to have the bridge ready for traffic on time. This requires considerably more attention to scheduling than the conventional project that must simply meet the one deadline of substantial completion.
5. An aspect that is too often overlooked is that most bridge failures occur during construction – by a factor of about a hundred compared to finished bridges. In the case of bridge retrofit, the public is using the bridge during the reconstruction process, which means that the public is placed at a much greater risk during reconstruction than it would normally be. The answer to this is not to increase the safety factors; it is to increase the quality and quantity of independent checking of the proposed construction activities, to a higher level than normal.

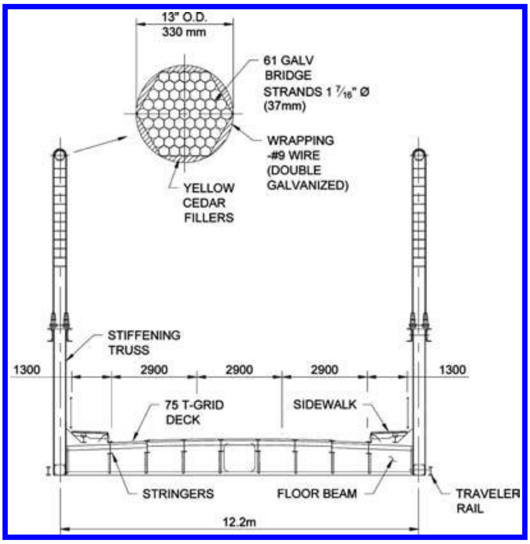

Figure 10. Original cross-section, Lions' Gate Bridge.

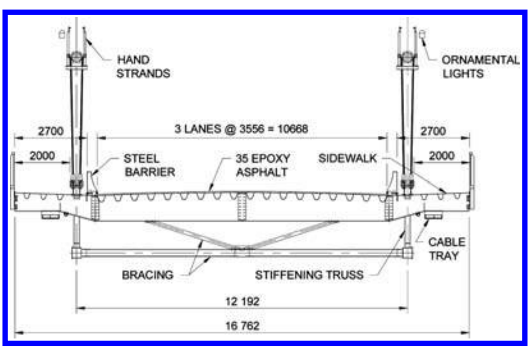

Figure 11. Renovated cross-section, Lions' Gate Bridge.

6.2 *Maintenance and access*

Although the primary purpose of retrofit is usually to restore or increase live load capacity, the opportunity should be taken, wherever possible, of improving access for inspection, and reducing the cost of maintenance. For example, when the Lions' Gate Bridge was widened by 47% (Figs. 10 and 11), the amount of steel surface to be painted, instead of increasing, was actually reduced by half.

When the Port Mann Bridge (Fig. 12) near Vancouver, BC, was widened, the opportunity was taken to add inspection walkways to the outer plate girders of the approach spans as shown in Figure 13. These have been invaluable in easing inspection of this ageing bridge prior to its replacement in the next few years.

6.3 *Other benefits*

In addition to improved maintenance, other benefits may sometimes be achieved with little extra cost, such as enhanced seismic resistance.

Figure 12. Port Mann Bridge, BC, Canada.

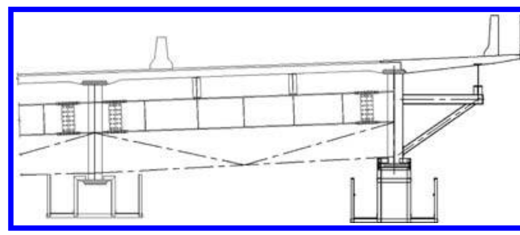

Figure 13. Partial cross-section of Port Mann Bridge approach spans, original material shown grey, new widening and inspection walkways in black.

6.4 *The devil in the details*

Details are typically connections, and field splices have to be capable of fast connection because of the limited time available, and they must be tolerant of misalignment and other potential problems.

Unfortunately, details are also where problems can be generated. Too often, details are designed in such a manner that secondary strains are induced, and fatigue cracking follows a few years later. It is fundamentally important to consider not just forces in a connection, but also the structural behavior. This is particularly true for orthotropic decks, which are becoming increasingly, and in my view, unnecessarily expensive, without a commensurate increase in performance.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Modern bridges in China

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ABSTRACT

In the past 30 years, especially during recent ten years, China has deployed the world's largest scale construction of road and bridges. Bridge construction technology has been improved rapidly and greatly to achieve more and more breakthrough in large spans. Bridge construction has concentrated in the three main regions, i.e. middle and lower reaches of the Yangtze River, middle and lower reaches of the Pearl River as well as the Yangtze River and the Pearl River Delta Region. (Fig. 1).

From 1979 to 2008, 16,000 bridges (with total length of 732 km) were built each year. By the end of 2008, highway bridges built in China had reached 595,000 (with total length of 25247 km).

Bridges in China can be divided into three types, namely girder bridges, arch bridges and cable-supported bridges (including cable-stayed bridges and suspension bridges). Girder bridges and arch bridges make up almost 70% and 30% respectively of all bridges built in China while cable-stayed bridges occupy less than 1% of total amount.

With the focus of bridge construction shifting from upper and middle reaches of rivers and lakes to lower reaches, bay and strait, China will face challenges to apply advanced technology in bridge construction to deal with more complicated meteorological, hydrologic, navigational and geologic conditions and realize a breakthrough in larger spans.

By the end of June, 2008 (statistics excludes Hong Kong, Macao and Taiwan):

– 55 bridges with main span over 400 m had been completed and 20 bridges with main span over 400 m are under construction.

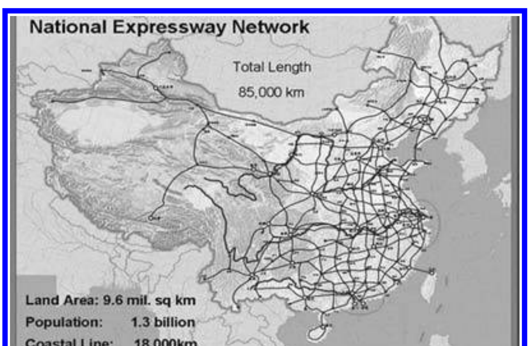

Figure 1. National expressway network.

Figure 2. Xihoumen Bridge with 1650 m span.

– 18 bridges with main span over 600 m had been completed and 15 bridges with main span over 600 m are under construction.
– 10 bridges with main span over 800 m had been completed and 9 bridges with main span over 800 m are under construction.
– 5 bridges with main span over 1000 m had been completed and 5 bridges with main span over 1000 m are under construction.

The maximum main span of completed girder bridges, arch bridges and cable-stayed bridge has reached 330 m, 552 m and 1088 m, ranking the first of same type of bridge in the world while that of the suspension bridge has amounted to 1650 m, ranking the second in the world.

This article introduces briefly the latest technical achievements for the construction of highway bridges, the status quo of bridge maintenance and management as well as the planned construction tasks in the mainland of China.

China has to face new technological challenge to achieve new technological breakthrough of integrated system, combined structure and compound materials in light of bridge construction characterized by "larger span, sea-crossing long bridge, deep-water foundation and extra-high bridge tower".

Heading to the 'golden age' for bridge construction, the Chinese engineers are well prepared and will cooperate with our colleagues from all over the world to meet the challenge of the largest scale bridges construction of the world in the new century with new structure, new material, new technique and new equipment as well as innovative management.

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Cable supported bridges – design, maintenance, rehabilitation and management

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ABSTRACT: This paper presents experience collected by engineers from COWI during involvement in design, maintenance, rehabilitation and management of several major cable supported bridges all over the world. First, the major problems encountered during the exploitation of major existing cable supported bridges are discussed. Then, several aspects of maintenance and management of such bridges are presented. Finally, the main issues related to the proper design and rehabilitation of major cable supported bridges are analyzed. At the end of the paper several case studies related to the maintenance, management, rehabilitation and design of major cable supported bridges are shown.

1 INTRODUCTION

Cable supported bridges are usually crucial elements to the road and railway infrastructure. Very often they constitute a part of critical links between highly habited areas. As a consequence their closure or traffic capacity reduction causes major inconveniences for the users and result in significant losses to the economy. Furthermore, construction, maintenance and rehabilitation of major cable supported bridges are rather costly. Therefore, the design, maintenance, management and rehabilitation of these structures are complex and challenging tasks. Consequently, they have to be performed using state-of-the-art technical solutions and the best practice gained over many years of experience. Also, they should account for the results of Life Cycle Cost analysis and the interferences with the bridge function.

This paper presents the experience collected by engineers from COWI during involvement in design, maintenance, rehabilitation and management of several major cable supported bridges all over the world. First, the major problems encountered during the exploitation of major existing cable supported bridges are discussed. Then, several aspects of maintenance and management of such bridges are presented. Finally, the main issues related to the proper design and rehabilitation of major cable supported bridges are analysed.

At the end of the paper several case studies related to the maintenance, management and rehabilitation of major cable supported bridges are shown. The first case study is the rehabilitation of the Aquitaine suspended bridge, in France, involving replacement of the main cables and widening of the bridge deck from 4 to 6 lanes. The second case study is the installation of the dehumidification system for the main cables at the Höga Kusten Bridge, in Sweden. The third case study is the replacement of bearings and expansion joints in Faroe Bridges, in Denmark. The following is the replacement of expansion joints, surfacing and corrosion protection at New Little Belt Bridge, in Denmark. Afterwards, the case study showing the assessment, monitoring and evaluation of hangers' vibration in the Great Belt Bridge in Denmark and the development of the management system for heavy transport at this Bridge is included.

2 SELECTED MAINTENACE PROBLEMS IN MAJOR CABLE SUPPORTED BRIDGES

2.1 *Deterioration of main cables and hangers*

The main cables and hangers are the essential elements of a cable supported bridge. At the same time they are also the most difficult element to inspect and maintain. Therefore appropriate attention should be paid to the main cables.

Deterioration of main cables on suspension bridges is in practice always associated with corrosion (Sørensen et al. 2006). Deterioration of hanger cables and stay cables are always associated with corrosion or vibrations or a combination of these. The basic cause of corrosion (see Fig. 1) is the presence of water or moisture.

The atmosphere at the bridge site often contains salt spray from the sea or from deicing salt, as well as other pollutants from vehicles or industry. These elements will eventually enter the cable in connection with rain or vapour, which can enter the cable through various openings. It causes corrosion attack on the wires.

Worldwide there are many examples of serious corrosion problems, even on relatively new bridges. For

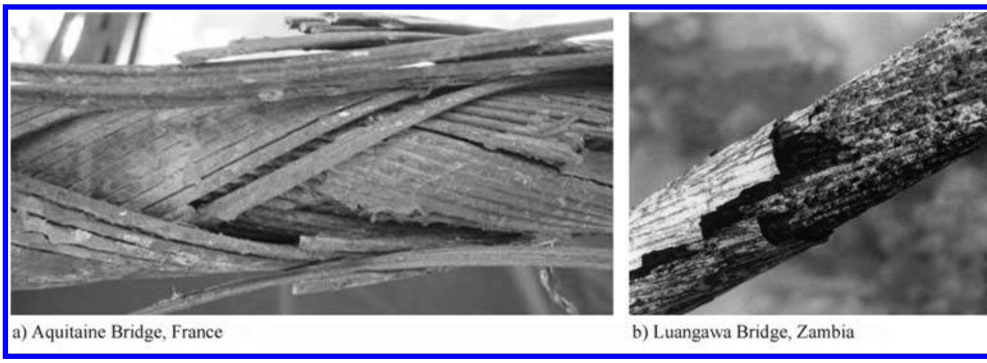

Figure 1. Details of corroded strands.

newer bridges the problem can be even more serious, as the design safety factor for the main cables has been progressively reduced over the years. It has been demonstrated by many examples that a traditional system can slow down corrosion, but it can not prevent it.

All these factors have led to the development of a new corrosion protection system for main cables, which is based on dehumidification (Bloomstine et al. 2006). In contrast to a traditional system, a dehumidification system eliminates the source of the corrosion problem – water/moisture in the cables. Hence, a properly designed dehumidification system provides complete corrosion protection of the main cables, as they are enclosed in an atmosphere with a sufficiently low relative humidity, such that corrosion can not occur.

2.2 *Loss of paint and deterioration of the bridge deck*

All steel elements exposed to the normal environmental conditions suffer from corrosion due to presence of oxygen and humidity. Steel elements of major bridges are usually exposed to much more harsh conditions of marine environments, where the presence of chlorides accelerates the corrosion process. Therefore, they have to be well maintained and repainted every time the coating losses its protective capabilities.

Service life of coating in aggressive environments is often limited to about 25 years. This is a short period compared with the service life of a major bridge, which usually is designed for 120, 150 or even 200 years of service life. Is it possible to improve this in order to avoid the costly repainting works which often also cause some traffic disturbances?

The problem of deterioration of steel elements has been partially solved in modern cable supported bridges with box girders, where dehumidification systems for the interior of the box girder are installed. These mechanical systems keep humidity level inside the girder at such a level that corrosion of the steel does not propagate. Also, in some modern bridges robots for performing repainting works of the outside surfaces of the decks have been installed, which speed up and simplifies the process. However, in many existing bridges loss of coating and corrosion is still a major cost problem.

2.3 *Deterioration of bridge deck surfacing*

Modern cable supported bridges being part of road and railway infrastructure are to a large extent designed using the orthotropic steel decks.

In the case of cable supported railway bridges with orthotropic steel decks the track support details are typically consisting of a slab track solution or a direct fastening system. Both solutions allow a controlled load transfer to take place to the bridge superstructure with minimum fatigue problems. The main challenge is to accommodate secondary loadings (non rail) and to ensure an efficient waterproofing at the interface between the direct fastening/track slab and the remaining part of the orthotropic bridge steel deck.

The roadway surfacing on modern orthotropic bridge steel decks are generally composed of asphaltic layers with a total thickness between 40 and 60 mm. On the Great Belt East Bridge in Denmark the surfacing under the traffic lanes consists of (from bottom to top):

- 4 mm mastic
- 25 mm intermediate layer of mastic asphalt
- 30 mm wearing layer of mastic asphalt

This type of asphalt surfacing has been used on orthotropic steel bridge decks in Denmark since 1970 and has proven to be very durable (see Wegan and Bloomstine 2004).

Road surfacing on movable bridges with orthotropic steel bridge decks with special requirements to weight has been carried out using polymeric resin surfacing. The reduction in weight is a must from a functional point of view and it is expected to be the next step into optimization the design of orthotropic bridge steel decks for long span cable supported bridges. Some of the challenges to be overcome are fabrication tolerances and fatigue of steel.

As regards practical experience of both mastic asphalt and epoxy asphalt roadway surfacing the five U.K. bridges built during the 1960's and 1970's with orthotropic steel bridge decks provide valuable lessons learned. (see McFadyen & Blumensen 2010).

Table 1. Expansion joint movements.

Bridge	Additional mechanical systems	Continuous length	Maximum movement at joint	Yearly accumulated movement at joint
Great Belt Suspension	Hydraulic system (buffers)	2694 m	±1.0 m	~100–200 m
East Bridge	Without hydraulic buffers	2694 m	±1.0 m	~50,000 m
Great Belt Steel Approach spans – Halsskov Bridge	–	2518 m	±0.7 m	~100 m
Standard steel beam bridge	–	50 m	±0.02 m	~5 m

2.4 *Wear and malfunction of bearings, expansion joints and other movement devices*

Many years of experience of design and maintenance of bridges show that bridge elements allowing for movements, such as bearings, expansion joints, etc., are among the more expensive as far as maintenance costs are concerned. These mechanical devices require regular maintenance and partial or full replacement several time during the bridge service life. Movement elements generally represent a minor portion of the total budget for construction of a new bridge. During the service life this ratio is generally quite different and the cost for maintenance, repair and replacement of movement elements is most often a major expense seen in relation to other maintenance costs (Sørensen et al. 2007).

The movements of a major cable supported bridge are naturally larger than on beam bridges with traditional spans. Suspension bridges in particular may experience major longitudinal displacements and specifically a very large accumulated movement which makes specific demands on durability. To illustrate this, the annual accumulated movement of an expansion joint on different bridge types is presented in Table 1.

In Table 1 the maximum movement and yearly accumulated movement at joint, for the case of the Great Belt Suspension Bridge are presented for the structural system with and without hydraulic buffers. The buffers reduce bridge fast movements caused by traffic or wind allowing in the same time for slow movements due to temperature and static loads. Wear of expansion joints is determined not only by accumulated movements but also by other phenomena such as frost, rain, traffic intensity, dynamic interaction, etc. However, the above relative comparison indicates that expansion joints and bearings for major cable supported bridges are more prone to wear and damages than those applied in standard bridges.

During several years of experience in maintenance of major cable supported bridges COWI had to deal several times with rehabilitation and replacement of bearings and expansion joints, sometimes only few years after bridge completion. The lesson learned from it is that bridges should be arranged with as few movement elements as possible. Furthermore, all of them have to be maintained regularly and inspected much more often than other bridge elements.

2.5 *Vibrations of cable stays and hangers*

Vibrations of stay cables and hangers under the combination of rain, wind and frost are observed at many major cable supported bridges (Laursen et al. 2006). This phenomenon is not yet well understood and due to this fact it is quite difficult to control. Although it is rather impossible to eliminate completely the cable vibrations, it is important to minimize their magnitude to such extend that the structural integrity is maintained, and failure due to e.g. fatigue will not occur. Sudden rupture of cable might put in risk safety of the whole structure and will cause disturbances to the traffic when the replacement work will have to be carried out. Consequently, the large vibrations of stay cables and hangers have to be avoided.

Several vibration mitigation alternatives may be used, from the most simple as the cable separators, spiral ropes, wind ropes, to the more elaborated damping systems with hydraulic dampers or tuned liquid dampers. Although previous experience is helpful in choosing the appropriate vibration mitigation devices, in some situations trial and error application supported by long term monitoring or frequent observation will be necessary.

3 OPERATION, MAINTENANCE AND MANAGEMENT OF MAJOR CABLE SUPPORTED BRIDGES

3.1 *General*

Major cable supported bridges are normally planned to be maintained for a very long period of time: 75, 100 or 120 years. Some bridges have been even assumed to be in service for more than 200 years. During all this period, due to limited possibility of traffic diversion (or even lack of any), the traffic on major bridges has to be provided with the minimum possible interruption. Due to all above mentioned considerations and having in mind the fact that the rehabilitation or replacement of such structures is extremely difficult and costly, special attention has to be paid to their operation, maintenance and management.

Generally, the entire service life of a bridge contains of two different subsequent stages (see Fig. 2). The first stage is the normal stage of operation, where just regular inspections are undertaken, some regular testing and monitoring is performed (if necessary) and

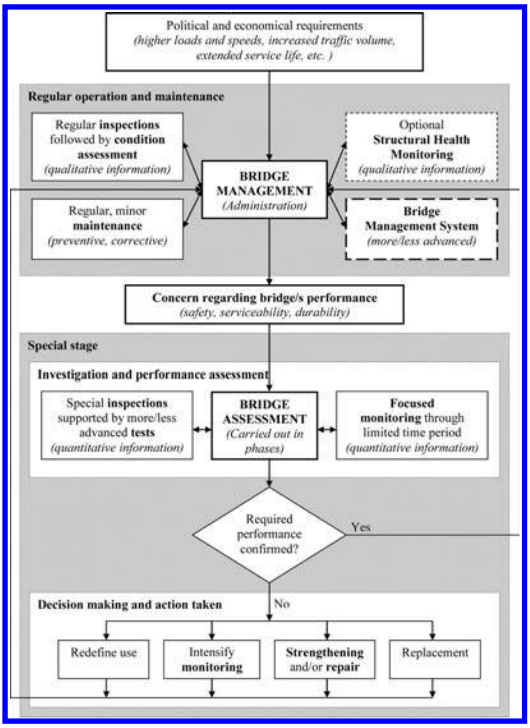

Figure 2. Stages and activities during entire life of bridges.

some minor maintenance actions are undertaken (e.g. painting, lubricating bearings, etc.).

The second stage is a special stage which starts when some concern regarding the safety or the functionality of the bridge arises and the assessment of the bridge have to be performed and/or some more costly maintenance action have to be undertaken, as for example repair, rehabilitation or strengthening. In order to extend the service life of a bridge into the maximum, considering the functional and economical constraints, several maintenance actions have to take place within its lifetime. Depending of the type of maintenance action, they might: slow down the rate of deterioration (routine maintenance), eliminate the source of deterioration and/or restore capacity loss (repair or rehabilitation), upgrade the capacity or the functionality of the bridge (strengthening or retrofitting).

The decision regarding what kind of maintenance action to undertake and when to do it in order to minimize the maintenance costs and maximize the bridge performance should be taken according to the maintenance strategy established for the group of bridges (e.g. along the whole railway line) or for the specific bridge (e.g. for some substandard bridges).

Bridge Management Systems (BMS) support the bridge owners and operators in their choice of optimum maintenance strategy that are consistent with the owners' policies, long-term objectives and financial constraints. Obviously, any BMS, in order to suggest optimal decisions, requires adequate and reliable information regarding bridge condition, safety, functionality, etc. All this information can be obtained due to inspections, condition assessment, performance assessment and testing and monitoring of bridges. Some of the mentioned activities are carried out regularly while others are performed just once or few times along the whole life of the structure.

All the above mentioned activities (i.e. inspections, condition assessment, performance assessment, testing and monitoring) have very important economical and functional impact on the operation, maintenance and management of existing bridges due to the fact that they may postpone the costly repair, retrofitting or replacement of the structure and thus also reduce the traffic interruption. COWI was recently involved in the development of several Guidelines covering all these issues for railway bridges (SB-ICA, 2007; SB-LRA, 2007; SB-MON, 2007; SB-STR, 2007).

3.2 *Inspections*

Effective maintenance management of major bridges has to be based on their actual condition. Appropriate scheme of carefully performed bridge inspections allows to identify damages, malfunctions and wear of several bridge elements and ancillary items (as has been discussed in section 2) before any risk may arise to the bridge users or to the structure itself.

The inspections scheme for major bridges is usually similar to the general scheme used for the inspection of all bridges in the country and most often consist of three (eventually four) levels of inspections:

1. Routine inspections – every half/one year
2. General inspection – every 5–6 years
3. Special inspections – whenever required

During inspections of major bridges, much more attention has to be paid to the ancillary items as bearings, buffers, expansion joints, etc. than it is in the case of ordinary structures. This is due to fact that their malfunction may cause major problems. Furthermore, great attention has to be paid to the corrosion protection and vibration mitigation of main cables and hangers.

3.3 *Testing and monitoring*

Several testing and monitoring techniques, developed in last decades, have found application in broadly meant "operation and maintenance" of some major cable supported bridges. Testing and monitoring are very suitable to supplement visual inspections with some more subjective data. Moreover, continuous monitoring techniques enable, to some extent, reduce scope or the intervals between the visual inspections. Very important feature of the monitoring is also the ability of the immediate warning whenever some minimum required conditions are not fulfilled and the safety of the users or the bridge itself is in hazard. Due to above mentioned, monitoring systems should become the standard "equipment" of modern major cable supported bridges, however selected testing

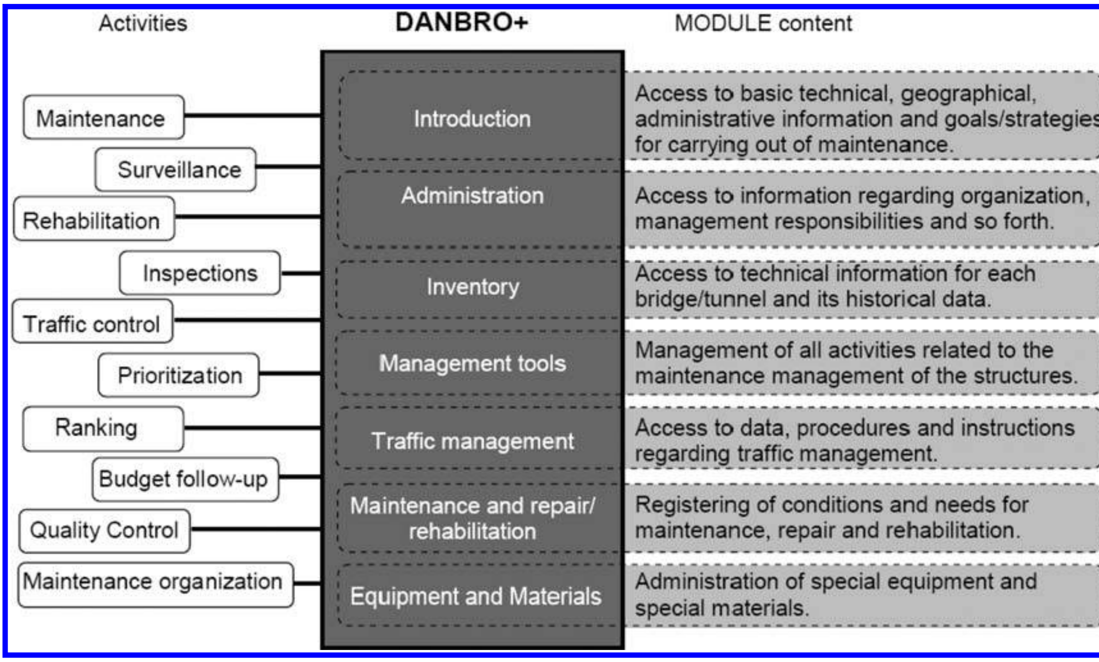

Figure 3. Basic elements of DANBRO + Bridge Management System (BMS) (Bjerrum et al. 2006).

methods should be used for the bridge inspections on a routine basis.

3.4 *Maintenance activities*

Major cable supported bridges require clear maintenance strategy based on the:

1. Actual bridge/element condition identified based on inspections combined with non-destructive testing and optional monitoring
2. Forecast of deterioration based on the previous records, experience and theoretical models
3. Identification of the importance (e.g bridge element or ancillary item) to the global safety and operability

Maintenance and repair work in major bridges in general should be carried out before deterioration develops to a major extent. Such a preventive maintenance policy reduces the risk of large scale repair work and herby minimizes the user inconveniences. From the experience of COWI in the maintenance management of several large bridges it can be concluded that the rehabilitation of such structures is extremely costly and that preventive maintenance policy is the most appropriate. This of course does not have to be the case for small bridges of low importance.

3.5 *Bridge management systems*

An effective management system that allows for an easy access to updated information for bridge operators, consultants and contractors is nowadays, in the age of the computers, a basic "equipment" of major bridges. The systematic maintenance of major bridges involves an extensive list of technical, traffic and administrative activities. Therefore, an IT Bridge Management System (BMS) has to be able to cover a number of different disciplines such as maintenance, organization management, surveillance, inspections, rehabilitation, traffic control, priority ranking, budget and quality control. The purpose of the management system is to optimize and support the daily administration of bridges including systematization and quality assurance of the management of the structures carried out by various engineers and technicians.

A long cooperation of COWI with Danish Road Directorate during the development and application of BMS in the maintenance management of large bridges has lead to the following conclusion. The effective BMS for major bridges has to be user friendly, presenting data in an effective way, updating data and documents in an easy way and having all information located in a single place. This is required due to the number of disciplines, people and organizations involved. The system has to be capable of supporting several activities as presented in Fig. 3.

3.6 *Managing heavy transports*

In recent years heavy (overweight) transports have increased significantly and thereby the need for efficient overweight permit systems is more important than ever to prevent bridges from overloading. In Denmark the increase is first of all a result of the quick-growing wind power industry.

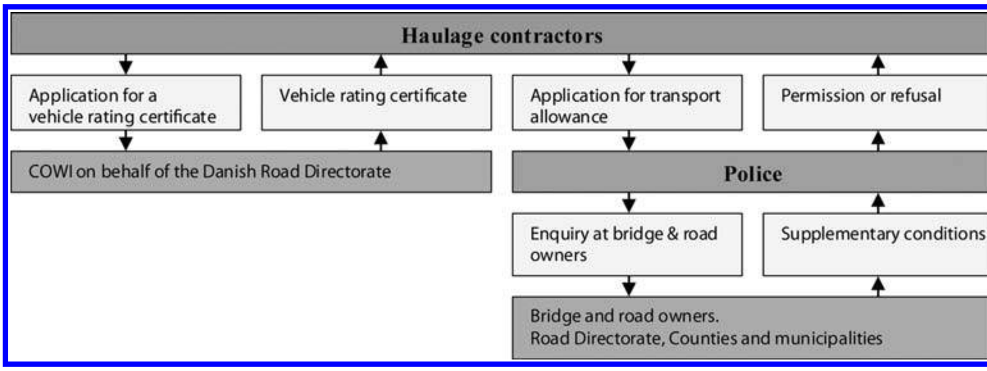

Figure 4. Principles for application of heavy transport permission (Ravn et al. 2006).

The management of heavy transport on standard bridges with spans up to 50 m is made usually by use of simple rating system, where all bridges and vehicles are rated according to the maximum capacity and weight respectively (Ravn et al. 2006). The vehicle rating estimates the load effects which the transport exposes to the bridge while the bridge rating estimates the load capacity of the bridge. In Denmark the standard rating classes are from 10 to 200 tones. The classification of the bridge is determined by the heaviest standard truck that crosses the bridge under specified conditions i.e.:

1. Normal class – Heavy transport and normal traffic, whole bridge area may be used, no restrictions;
2. Normal class: Heavy transport and normal traffic, whole bridge area may be used, no restrictions;
3. Restricted class 1 – As normal class but only the roadway may be used;
4. Restricted class 2 – As restricted class 1, but the speed must not exceed 10 km/h;
5. Restricted class 3 – The heavy transport is the only transport on the bridge, low speed of max. 10 km/h and driving in the least critical position.

The normal classes are visualized on maps on the Internet, called the Heavy Load Grid. The road stretches in the grid are given a classification equal to the lowest Normal Class bridge present on the actual stretch. The maps are for daily use by police and road administrators. Restricted classes are only used by bridge/road administrators, in cases where the vehicle class is higher than the bridge classes shown on the map.

The vehicle class is calculated for simply supported bridge spans from 2–200 m. The vehicle class is determined by reference to predefined standard trucks labeled by their overall weight. The vehicle class equals the standard truck that produce the same load effects as the actual heavy transport on the particular simply supported span.

Application for transport permission shall be forwarded to the police and the police may issue permission if the maximum vehicle class for all spans is lower than or equal to the bridge classes on the current route. If not, bridge/road administrators are involved and restricted classes are taken into consideration as well as vehicle classes on critical span lengths (see Fig. 4).

Major cable supported bridges require refined rating principles but the heavy transport management system can be similar as it is for the standard bridges. The method used in Denmark for major bridges applies the actual overweight vehicle for rating of critical structural elements. As the heavy transport is used directly in the evaluation for the particular bridge, the uncertainties introduced as consequence of the vehicle rating are eliminated and the results are more accurate.

The Danish approach for administrating overweight vehicles was developed keeping in mind that the rating systems are used numerous times every day and hence shall be cost effective and at the same time assure the best utilization of the load bearing capacity of the bridges on the Danish road network. Practical experience has proved that both goals have been achieved.

4 DESIGN, ASSESSMENT AND REHABILITATION

4.1 *Design of new bridges*

The design life requirement for new major bridges is constantly increasing. It is often more than 100 years and in some cases as high as 200 years, e.g. the Messina Bridge. The design traffic load is also increasing with regards to intensity and weight. Therefore it is necessary to especially focus on the durability of the bridge elements that are directly affected by traffic.

Requirements concerning the operation period of the bridge are also becoming more common as bridge owners are starting to realize that a low tender price does not usually give a low lifetime cost. Therefore, Life Cycle Cost (LCC) Analysis must be incorporated in the design to ensure a durable design with the correct lifetime and the lowest possible Life Cycle Cost. The optimal solution with the lowest LCC must be determined for each bridge element, resulting in the lowest LCC for the entire bridge. There is also increasing focus on availability of the passageway,

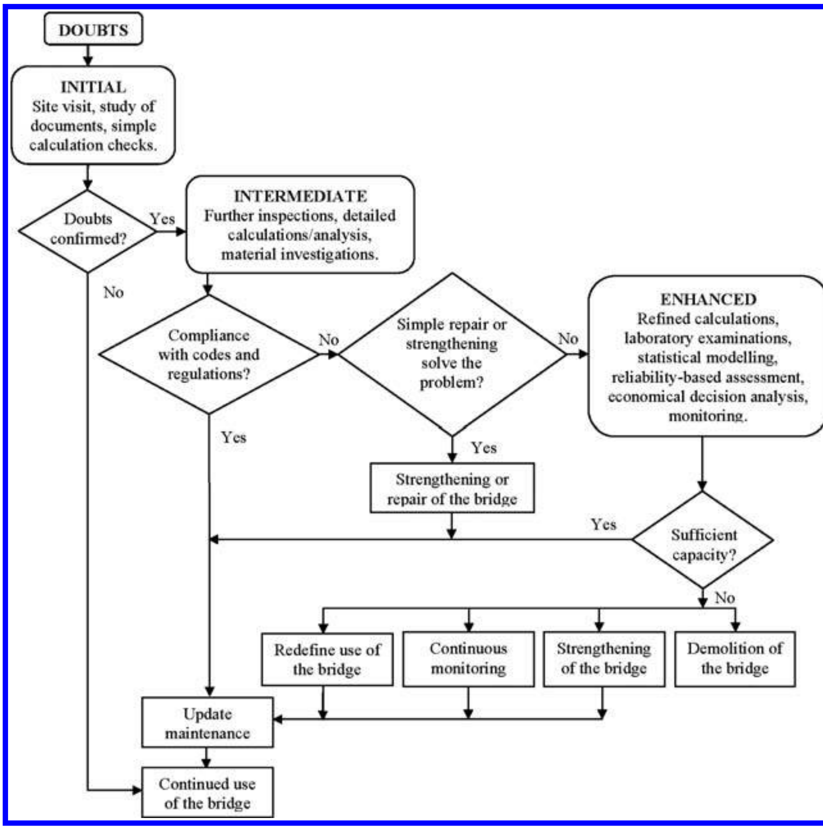

Figure 5. Step-level procedure for reassessment of bridges (SB-LRA, 2007).

meaning that traffic disruptions are not allowed. The design must therefore allow inspection, maintenance and repair without disturbing the traffic. Furthermore access facilities for all structures should be incorporated in the initial design, as this allows fulfillment of the aforementioned requirements with regards to traffic and LCC.

Key areas for future bridge design are therefore durability/low LCC, accessibility and availability.

4.2 *Assessment of existing bridges*

Assessment of an existing bridge with the purpose of re-qualifying the bridge for increased loading and/or for prolonging the service life may be seen as an adaptive, step-level process of refining the state of knowledge regarding the present and the future state of the bridge and its behavior. An assessment may involve a review of project documentation, inspection of the structure, testing of materials, testing of structural performance, refined numerical analysis and planning of future inspections.

The decision on whether or not to collect more information is always based on the existing information (prior information) and the expected reduction of the life cycle cost obtained on the basis of the additional information. Depending on the actually achieved knowledge (posterior information) it may or may not turn out to be feasible to refine further the state of knowledge. Also, in the same manner, the re-qualification actions (strengthening and repairs) may be evaluated, compared and selected. It should, however, be noticed that economical considerations alone, may not be sufficient for re-qualification purposes as explicit requirements to the safety of the bridge are often dictated by legislation.

Fig. 5 shows the step-level procedure recommended in the SB-LRA (2007) Guideline to be used in the process of assessment of existing bridges. Considering the above discussed topics, in the presented procedure, the knowledge about the bridge is established and refined in an adaptive manner according to the actual needs.

As it can be seen in Fig. 5, an assessment of existing bridges in the proposed procedure is divided into three levels, which in terms of refinement and detailing can be characterized as follows:

1. Purely heuristic experience or code based statements (initial assessment);
2. Application of deterministic and semi-probabilistic safety formats (intermediate assessment);
3. Instrumentation, testing and/or probabilistic analyses (enhanced assessment).

Generally, an assessment can be carried out within the framework of these three phases. However, the level of detail within each phase may vary. In this way it is possible to tailor a reassessment for different purposes. The level of detail of the assessment is recommended to be chosen considering the detailed assessment objectives for the particular bridge and its characteristics.

According to the presented step-level procedure, the capacity of the bridge in cause is initially assessed on the basis of simple calculation checks and readily accessible data (drawings, design calculations, earlier assessment calculations, inspection records, etc.). On this basis, the extent to which the bridge fails to comply with the given requirements is evaluated.

In the intermediate level of assessment, the capacity of the bridge (which fails the initial assessment) is evaluated using more advanced analysis (e.g. elastic but giving better idealization, plastic, etc.) and more accurate data (obtained from inspection an simple tests) on the material properties, the loads, the current state and the behavior of the bridge (e.g. material properties obtained from simple measurement, loads defined by measurements, etc.).

Finally, in the enhanced level of assessment, the capacity of the bridge, which fails the intermediate assessment and where repair or strengthening costs are significant, can be evaluated using most advanced assessment methods (e.g. reliability-based assessment methods, system level assessment, etc.) and tools available (e.g. non-linear analysis, probabilistic analysis, testing, monitoring, etc). Testing and monitoring may provide relevant data regarding actual bridge loads, actual properties of material, and actual behavior of the bridge. However, probabilistic analysis and non-linear analysis allow for considering the actual variability in modeling loads and resistance properties, and taking into account bridge redundancy and system behavior.

The sensitivity analysis, performed during the assessment, may help to identify where the refinement of the knowledge about the bridge may be the most beneficial for the assessment of the bridge in cause. As already discussed, such refinements may be based on detailing of the analysis methods and/or further collection of data.

4.3 *Rehabilitation*

Rehabilitation of bridges should generally take into account the same requirements as mentioned above for new bridges. Furthermore, before starting rehabilitation design it is essential to carry out inspections and investigations to determine the cause of the damages/degradation, such that the rehabilitation design can prevent similar problems in the future. New and durable details should be developed to replace the original design.

During rehabilitation it is essential to focus on prevention of traffic disruptions. Many bridge owners have begun requiring this to be a part of the LCC Analysis by setting a price on traffic delays.

Many older bridges do not have safe and/or sufficient access facilities. A rehabilitation project is an ideal opportunity to upgrade these facilities and an upgrade will decrease future maintenance costs. Access facilities should therefore always be considered in connection with rehabilitation.

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Lifetime design of cable-supported super-long-span bridges

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ABSTRACT

The main issue when deciding upon a specific service life is to clarify the event, which will identify the end of the service life. However, the lack of specification provisions related to the length of service life in the codes constitutes an obstacle to the realization of extended service life for bridges. This situation becomes more acute in the case of super-long-span bridges because of their nature. In view of the current practice, service life of 100 to 150 years seems to be achievable and has already been implemented in recent major bridges owing to special specifications by the owners of the bridge projects. This paper reviews current practices for service life design with some recent examples in Korea and addresses issues that will be faced in the lifetime design for cable-supported super-long-span bridges. Limitations of the current practice are highlighted and, additional requirements and research directions that will help designers in achieving extended lifetime of 200 years or more for the super-long-span bridges are suggested. Dedicated research programs currently implemented in Korea are also presented.

1 INTRODUCTION

1.1 *Lifetime design, a definition*

The purpose of lifetime design is to predict and optimize the lifetime quality of the designed structure. Lifetime quality means the capability of the structure to fulfill the requirements of its users, owners and society over its entire planning and design life (Sarja 2010).

This sentence involves two major concepts that are performance and design life of the structure, which are indeed closely related. In the case of bridge structures, the required performances are given in the relevant design codes. These performances are currently specified in terms of allowable strength or limit-states. However, design codes are not specifying explicitly the concept of lifetime design.

The AASHTO LRFD Bridge Design Specifications define service life as the period of time that the bridge is expected to be in operation. The design life is defined as the period of time on which the statistical derivation of transient loads is based. Though the specifications prescribe transient loads based on a design life of 75 years, they are not giving any basis or explanation on the extent of the expected service life. In other words, the 75-year prescribed design life is an indicator of the structural safety and reliability, while the service life is an indicator of the durability. However, the specifications do not prescribe the expected duration for service life nor give direct correlation between design and service life (Bartholomew 2007, 2009).

It is noteworthy that lifetime design is a recent concept of which definition has not been clearly defined to date. The Roman builders did certainly not have imagined that many of their bridges would still remain operational 2000 years after their construction. Roebling, the designer of Brooklyn Bridge, also did not apply the concept of design lifetime for his bridge, which is still one of the most important arterial road linking New York and the district of Brooklyn even after 126 years of service, while most of its contemporaries have collapsed or have been replaced meanwhile.

On the other hand, recent literature sets a target design lifetime for long-span bridges since the end of 1990s. For example, the Great Belt East Bridge (1994) in Denmark and the Confederation Bridge (1997) in Canada have been designed for a 100-year life expectancy, as well as the more recent Incheon Bridge (2009, Figure 1) and the Busan-Geoje Fixed Link actually under construction in Korea (Figure 2). A "useful project life" of the Millau Viaduct (2004) crossing the Tarn Valley in France is 120 years. The Humber Bridge, the world's longest single span suspension bridge has a design life of 120 years. And a record of 150 years design lifetime was set for the

Figure 1. View of Incheon Bridge linking Youngjong Island and the mainland (total length: 12.3 km, central span: 800 m, design service life: 100 years).

Figure 2. View of one of the two cable-stayed bridges (Geoga Bridge) of the Busan-Geoje fixed link under construction (total length: 8.2 km, central span: 470 m, design service life: 100 years).

Akashi Bridge (1999) in Japan and the Oakland Bay Bridge in San Francisco.

The question that arises now is the basis on which such design lifetimes have been selected or computed. A review of the relevant literature shows that the assessment of the design lifetime of these bridges relies essentially on the lifespan of the involved materials. That is, if the durable life of concrete reaches 100 years so does the structure. However, could we simply state that durable lifespan of the materials is guaranteeing equivalent lifespan of the whole structure?

How can designers suggest reasonable service life for a bridge structure considering that the target or expected service life relies on the owner's desires or societal needs whereas the actual service life depends on the quality of materials, design and construction, the environmental conditions to which the structure is exposed, and on the maintenance strategy?

1.2 *Lifetime design, the issues*

In this paper, super long-span bridge refers to a bridge exhibiting span length equivalent or superior to the longest span lengths realized to date. That is span longer than 1,000 m for cable-stayed bridges and longer than 1,990 m for suspension bridges.

Differently from common bridges, long-span bridges represent the unique access to remote places. As a matter of fact, any disruption of such bridge for repair or replacement will have tremendous effects on the local economy and obstruct completely communications for relatively long period. In the case of such super-long-span bridges, these effects will grow exponentially since super-long-span bridges not only by their length but also by their role as inter-continental route will affect a wider area and a larger community. In addition, the costs required for the construction and maintenance of super-long-span bridges will also grow exponentially. Therefore, the lifetime design of super-long-span bridges addresses issues of which feasibility and practicability should be discussed thoroughly.

The ability of the bridge to fulfill its intended functions along its service life can be compromised due to degradation. The ideal bridge any designer is dreaming of is a bridge that will stand for an infinite service life with minimum maintenance or maintenance free. However, such a bridge is not possible in practice as all materials deteriorate gradually when exposed to environment and traffic. Even though bridges are made of different materials, have different structures, are located in different locations and have different ages, their common feature is their degradation with time. Major causes of degradation are high transient loads and severe environmental conditions.

A fatigue life of 200 years may be feasible through careful technical considerations involving high initial quality and adequate maintenance activities. Assuming a fatigue life of the bridge of 200 years, the probable range of the bridge life can be thought of in the order of 100 to 400 years (Frangopol et al. 2000). Improving the design methods of bridge components is the basic step to achieve this target. Proper structural design addresses the effects of transient loads through adequate proportioning of the members and design details. In addition, well-coordinated design, construction and maintenance are essential to sustain bridge structure with an extended service life. As mentioned above, an important issue having determining influence on the service life is the material durability. However, the design lifespan of the material constitutes a necessary but not sufficient condition in the extension of the service life of the structure.

A typical example is the first Jindo Bridge, which is the first cable-stayed bridge in Korea and was also the longest cable-stay span outside Europe at its completion in 1984 (Figure 3). This bridge was designed with respect to a design live load DB-18, equivalent to the design load HS-18 in USA. In 1996, the close inspection of the bridge including loading and vibration tests resulted in the closure of the bridge to heavy trucks since the structure showed signs of fatigue and accelerated degradation due to the increasing crossing of truck loads exceeding the design load. Thereafter, the governmental authorities decided to improve the load carrying capacity of the existing bridge while erecting

Figure 3. The First Jindo Bridge (right) and Second Jindo Bridge (left).

a twin bridge design for vehicle load of DB-24 and DL-24. This example shows clearly that, apart from material durability, the lifetime design of long-span bridges should also consider future change of traffic and loading in order for the bridge to provide extended useful service life.

Another issue is the reliability of the service life of the members predicted on the basis of the material strength. The project performance requirements (PPR) of Incheon Bridge specified a design life of 100 years for the whole structure with a design life of 75 years for the major structural components and less than 75 years for the secondary and replaceable components. However, these values correspond to the target design life which may be different from the actual service life of the members. Any defect in a member of the structure prior to the end of its target life will likely have effect on the neighboring members and endanger the integrity of the whole structural system. Need is thus to assess the eventual satisfaction of the target design life for each of the members in order to achieve the intended service life of the structure.

Issue also is for the current design practice applied for the safety under extreme events. A design life of 200 years implies that the bridge will have larger probability to experience one or multiple extreme events during its service life. For example, seismic design considers a return period of 500 years or 1,000 years. Is the current practice still reasonable for a structure that should stand operational for 200 years? If not, how should we modify our design approach considering the fact that long-span bridges exhibiting longer lifespan should also guarantee higher risk?

The main issue when deciding upon a specific service life is to clarify the event and feature, which will identify the end of the service life (Rostam 2005). For typical structures, national codes and regulations define implicitly service life requirements through the standards and codes. Complying strictly with the performance requirements stated in codes and standards will only provide the minimum quality and performance being acceptable to society. For such structures additional requirements would be required if truly long-term performance and service life of the structures are needed.

Accordingly, this paper reviews current practices for service life design through recent examples in Korea and addresses issues that will be faced in the lifetime design for cable-supported super-long-span bridges. Limitations of the current practice are highlighted and, additional requirements and research directions that are being discussed among Korean experts to help designers in achieving extended lifetime of the structures are presented.

2 SERVICE LIFE AND DESIGN LIFE IN CODES

The bridge stock constitutes one of the most expensive assets in the national infrastructure. Unfortunately, bridge structures start to degrade since their construction. Therefore, reducing at the most efforts and costs to be invested in their future management so as to extend their operational life must be dealt imperatively since the design stage. In such aspect, defining or selecting a target service life is primordial to derive adequate specifications or regulations for durability design, fatigue design or lifecycle costs.

In industrialized countries, the bridge stock has today reached severe degrees of degradation that are requiring immediate repair or replacement. Considering that most of the bridges have been built in the seventies and assuming that these bridges were designed to have service life of 50 years, a simple calculation implies that all these bridges should be replaced within a short period of time, which means tremendous costs induced not only by their reconstruction but also by the indirect losses provoked by the subsequent closure of traffic. Accordingly, extending the service life of bridge structures is a necessity not only in terms of the national economy but also in terms of safety.

Accordingly, national codes are defining design life and categories as shown in Table 1. Since service life involves consideration of many designs, materials, construction and environmental factors, these definitions of design life do not represent a basis for service life. These specifications do not recommend any specific period for service life.

On the other hand, some notable concrete bridges have been constructed recently using criteria developed to ensure service life of 100 to 150 years. Record service life criteria have been indicated by the 300-year target service life of the Second Gateway Bridge in Australia. Special serviceability limit state specifications and concrete specifications were developed for the Confederation Bridge to secure a minimum service life of 100 years. Concrete specifications were also developed to ensure a service life of 100 years for the Great Belt East Bridge between Sweden and Denmark.

This paper identifies three major topics to be dealt in order to achieve the intended service life of the bridge structure. These are material durability, traffic live loads and extreme events. For each of these topics, current practice is reviewed and corresponding limitations for the lifetime design are derived in the case of super-long-span bridges.

Table 1. Design life in national codes.

Code	Definition of design life		
ISO and Eurocode	Design working life (DWL): Assumed period for which a structure is to be used for its intended purpose with anticipated maintenance but without major repair being necessary.		
	DWL category	Indicative DWL (years)	Examples
	1	10	Temporary structures
	2	10 to 30	Replaceable structural parts
	3	15 to 25	Agricultural and similar structures
	4	50	Building structures and other common structures not listed elsewhere in this table
	5	120	Monumental building structures, highway and railway bridges, and other civil engineering structures
BS 5400	Design life: Period of time on which the structure behaves safely without need of repair and within acceptable probability (service life design of 120 years since 1988)		
AASHTO	Design life: Period of time on which the statistical derivation of transient loads is based (75 years for these Specifications) Service life: The period of time that the bridge is expected to be in operation		

3 MATERIAL DURABILITY

As seen above, the current practice in assessing the design lifetime of bridge structures relies essentially on the lifespan of the involved materials.

Concrete and steel have been and are still the most popular construction materials. Sarja (2010) reported several features identifying the end of the service life of bridge structures as follows. Degradation is the dominant reason for maintenance and minor repair of concrete structures and represents 26% (steel) and 27% (concrete) of the causes for bridge demolition; obsolescence load is responsible of 74% of the causes for bridge demolition.

Environmental conditions that cause degradation include carbonation, sulfate attack, alkali-silica reaction, freeze-thaw cycles, and ingress of chlorides and other harmful chemicals. Adverse environmental conditions, if not properly addressed, typically cause chemicals to invade the concrete's pore structure and initiate physical and chemical reactions causing expansive by-products. The service life of the bridge structure in terms of materials depends on the deterioration of concrete, corrosion of steel and interaction between these reactions. The most damaging consequence of these reactions is depassivation and eventual corrosion of reinforcing steel causing cracking and spalling of concrete. The end of the service life of the structure occurs when the accumulated damage in the bridge materials exceeds the tolerance limit. However, the service life is typically extended by performing periodic repairs to restore the serviceability of the structure.

Accordingly, the durability design process for achieving extended lifetime requires the analysis of the environmental conditions in which the structure will be exposed, the adequate selection of the range of materials involved and the understanding and identification of their rate of deterioration. The probabilistic approach adopted by the DuraCrete method can be cited as the best example achieving such durability design process for concrete structures.

3.1 *Durability design, current practice*

Long-span bridges are expected to exhibit longer service life than common bridges, but are exposed to more severe conditions such as chloride ingress or wind load due to their location in marine environment. Therefore improved and specified design methods have been adopted to achieve the expected design life.

During the last several years, models have been developed to predict the service life of concrete bridges exposed to chlorides. Several service life prediction models assume diffusion to be the most dominant mode of transport for chloride ions. The time taken by chlorides to reach reinforcing steel and accumulate to a level exceeding the corrosion threshold is known as time to initiation of corrosion (TIC). TIC depends on many factors like the diffusivity of concrete, concrete cover, temperature, and the degree of exposure. The propagation time, from initiation of corrosion to intolerable accumulation of damage, also depends on many factors including environmental conditions and corrosion protection strategies.

Therefore, design codes provide qualitative method to achieve the expected design life such as concrete quality, type of cement, unit cement content, water-cement ratio, concrete cover depth and curing methods. However all these recommendations are "deem-to-satisfy" requirements, which have no rational relationship to the service life and the service life as such is not defined in an operational manner.

The following present several current practice examples of measures taken to protect materials from environmental ingress.

3.1.1 *Corrosion protections for reinforced concrete structures*

Durability life of concrete constitutes the most essential factor in achieving the design life of long-span bridges in many cases. Deterioration of concrete is usually affected by salt damage, neutralization, freethaw, alkali-aggregate reaction and sulfate attack. In particular, main damage to RC structure is the corrosion of the reinforcements caused by chloride attack and carbonation process which interacts with deterioration of concrete. These degradation mechanisms are currently dealt with by two methods.

Figure 4. View of the 3-pylon cable-stayed bridge construction site of Busan-Geoje Fixed Link.

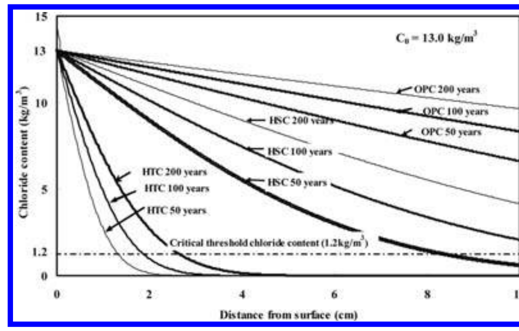

Figure 5. Required cover depth to achieve target design life for HTC (KICT 2009).

The first method controls the material characteristics through the concrete cover depth, strength of concrete, mix ratio and adoption of corrosion inhibitor. Increased concrete strength with lowered W/C ratio delays diffusion of the chloride ions, while increased cover depth slows down penetration of chloride ions and subsequently lessens corrosion of reinforcement. In case of the Busan-Geoje Fixed Link, an 8.2 km motorway between Busan and the Geoje Island (Figure 4), it is expected to last for 100 years on the basis of durability life of concrete and reinforcements. Of this 8.2 km, four-lane fixed link, there will be 4 km of immersed tunnel and two 2 km cable-stayed bridges. Based on the DuraCrete method, speed of chloride penetration and mix ratio were experimentally identified and the cover depths of pylon, caisson foundation and unit of immersed tunnel were accordingly determined to be in a range of 7∼8 cm to achieve sufficient durability from chloride ion attack to reinforcement. It was applied in a similar way in the Incheon Bridge where blast furnace slag was used to increase water density and the corresponding cover depths were required to be 13 cm for the splash zone and 6 cm for the superstructure.

The second method is to protect corrosion through cathodic protection (CP) and prevention. CP of steel reinforcement in concrete aerial structures is obtained by applying a direct current (D.C.) through the concrete from an anode system usually laid on the concrete surface. The anode system is connected with the positive terminal of a low voltage source and the negative terminal is connected with reinforcement acting as cathode. This method was later extended to the protection of bridge slabs and piles, marine constructions, industrial plants, garages and buildings exposed to chloride-induced corrosion (Pedeferri 1995). These methods are usually applied individually or in combination to the pylons or piers of long-span bridges exposed to severe salt attack. In real implementation, this method was adopted for Geogeum Bridge in Korea, which is currently under construction and scheduled to be completed late in 2011. The bridge is a cable-stayed bridge with a central span of 1.1 km and its durability is expected to ensure a design life of 100 years. Cathodic protection method was also included in design of Incheon Bridge project. Steel piles in ship collision protections were required to be protected from corrosion by CP method in CSR (Concessionaire Supplementary Requirements). The application of this method guarantees a 60-year design life for the ship collision protection. The CSR also requires additionally to execute maintenance every 4 years and replacement every 60 years for the cathodic protection systems itself (Incheon Bridge Corporation 2004).

Increase of concrete durability itself such as high performance concrete (HPC) is another approach to achieve design life by reducing the permeability. HPC with W/C ratio between 0.30 and 0.40 are usually more durable than ordinary concrete, not only because they are less porous, but also because their capillary and pore networks are somewhat disconnected due to the development of self-desiccation (Aitcin 2003). In the case of Millau Viaduct, researches into concrete mix formulation and greater control over its behavioral mechanisms revealed that the lifetime of the bridge can be guaranteed over 100 years in terms of material durability (Lafarge 2007). Increase of material durability of reinforcement can be a solution as well. Epoxy-coated reinforcements have been studied and applied in many cases. Stainless steel reinforcements have been also used for durability of bridge. Application of stainless steel reinforcements to the outer most vertical layer of reinforcing bars and links can be found in the application to the base pylon of Stonecutters Bridge (Mahmoud 2006).

It is interesting here to introduce a recently completed R&D project, called "Bridge 200", performed by Korea Institute of Construction Technology (KICT). It intends to secure bridge technologies extending the lifespan of concrete bridge to 200 years and to develop ultra high-strength concrete exhibiting strength 5 times larger than current concretes through 5 key technologies related to the durability of concrete, development of high-durable structures, and reinforcement techniques of existing bridges.

Ultra-high performance (HTC) featured by a water to binder ratio of 20% was developed and exhibited increased compressive strength of 180 MPa, which is larger than that of existing high performance concretes. Chloride ingress tests showed that HTC secures longer lifespan than other concretes with identical cover thickness. Tests on HTC also revealed that a cover thickness of about 27 mm is sufficient to secure a lifetime of 200 years against the penetration of chlorides (Figure 5). Such properties are achieved owing to the low W/C ratio of the material as well as the homogeneity of the cement matrix.

This project having been successfully completed in 2006, Super Bridge 200 has been launched in 2007 as a succeeding project involved in the top-brand projects of the Ministry of Sciences and Technology. Super Bridge 200 targets the development of bridge sustainable for 200 years by exploiting the so-developed ultra-high strength concrete to cable-stayed bridge structures. The expected outcomes are reduction of the maintenance costs by 20% and extension of the lifespan of the bridges by 200%.

3.1.2 *Corrosion protections for steel bridges*

The previous sections dealt mainly with the corrosion protection of concrete bridges. However, long-span bridges and super-long-span bridges are demanding lightweight materials, which mean that these bridges will make use of steel rather than concrete.

External corrosion protection is a prerequisite for sea side located steel bridges. They are generally required to be galvanized or coated with metal spray and paint. Heavy-duty anti-corrosive painting system is widely applied for steel bridges. Electric reactions between steel of bridge and ionized metal such as zinc, aluminum are used to protect corrosion. The Golden Gate Bridge adopted modern inorganic zinc-rich paint system for the first time in large bridge structures (Kline 2009). Stonecutters Bridge used a high-build epoxy based paint system on zinc rich primer has been specified (Mahmoud 2006). High-build inorganic zinc-rich and epoxy resin painting system was applied to Inoshima Bridge and Akashi Kaikyo Bridge (Chen et al. 1999).

Internal corrosion protections are also important. For Golden Gate Bridge, CP for steel bridge was used. For Incheon Bridge, dehumidification system was applied. This system aims to control the internal environment to provide a relative humidity of less than 60%. This will effectively prevent corrosion of the internal surface of the steel boxes and eliminate the need for the sophisticated corrosion protection scheme used on the outside.

Similarly to the reinforcement, researches and applications on the increase of steel durability itself have been implemented. HPS50W and HPS70W developed in USA show developed performance not only in strength but also in weathering and fatigue characteristics. These types of steel were verified with experiment and included in the AASHTO LRFD Bridge Design Specification.

Similarly in Korea, HIPER-CONMAT (High Performance Construction Material Research Center), a new-born research center established at the Research Institute of Industrial Science and Technology (RIST) in the form of a research institute–industry cooperative program extending for a period of 5 years, has developed HSB500W, HSB600W and HSB 800W with increased weathering and fatigue performances in the HSB (High-performance Steel for Bridge) project. Stabilized rust layer composed with Fe and a small

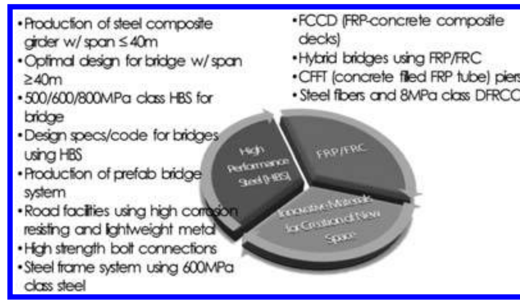

Figure 6. Key technologies of HIPER-CONMAT.

amount of alloying elements such as Cu, Cr, Ni protect steel from excessive rust which result in the increase of durability. It enables the developed steels to acquire weathering indices over 5.8. Those weathering steels also feature high weldability, which mean weld preheat is not required. Hybrid slab systems combining fiber-reinforced polymer (FRP) are also have been developed to improve durability of structure by lightening the weight of slab and prolongation of maintenance period from previous 10∼15 years to over 20 years.

3.1.3 *Cable protection*

The cables of long-span bridges as main structural members are required for long term endurance. The main cables of suspension bridges cannot be replaced usually, which means that their durability must be imperatively verified and secured.

Stay cables of suspension bridge are protected from corrosion conventionally by the following method. Each cable consists of high-strength steel wires coated with zinc. These galvanized wires are covered with an anti-corrosion paste, wrapped with annealed galvanized wires and painted. This system was first used on the Brooklyn Bridge about 100 years ago (Stahl et al. 1996). However zinc-coating is not sufficient in severe environmental condition near sea water that additional corrosion protection is essential.

Youngjong Bridge(2000), the first bridge with three-dimensionally profiled suspension cables and self-anchoring, used the advanced dehumidification process for cable protection. Dry air is produced by a dehumidification device containing silica gel. The air is carried through piping, enters the cable via an inlet cable band passes through the cable, and exits via an outlet cable band. Dry air is continuously injected at a rate of 0.25 m^3/min with the pressure of 300 mmAq. The relative humidity, designed to remain below 40%, has been continuously monitored on the bridge since the system entered service in 2000.

Among the projects implemented by the Super-Long Span Bridge R&BD Center that will be introduced in Chapter 6, the Core Research Project 2 intends to increase durability of cable material itself and cable system for super-long-span bridges. First, zinc-aluminum alloy coating method for high-strength cable has been developed to achieve highly corrosion-resistance coating system. Zinc-aluminum alloy coated cables are expected to have enhanced corrosion resistance more than 2 times compared to former zinc coated cables. Techniques and design standards for high-durability ceramic/heavy duty painting method are also developed to ease maintenance even in harsh conditions like near the sea water area. The painting systems are purposed to have B_{10} life longer than 20 years with 80% reliability. Cable dehumidification system expected to reach the global standards is also going to be developed through this project. Researches and experiments on durability of cables, anchorages and cable systems using FRP to improve corrosion resistance and fatigue performance are also under progress. These researches may contribute not only to the steel durability but also to the guarantee of the 200-year design life.

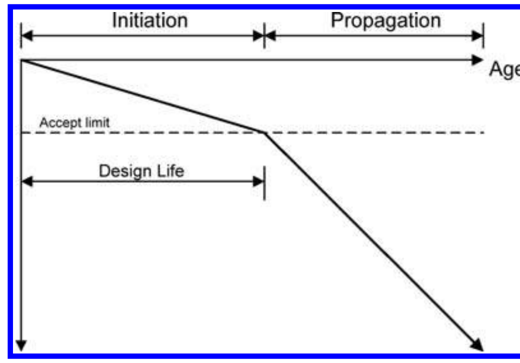

Figure 7. Design life of concrete structures: a two-phase modeling of deterioration (Tuutti's Model 1982).

3.2 *Limitations of the current practice in super-long-span bridges*

The afore-mentioned durability design methods may have some limitations for direct application to super-long-span bridges. One reason is that super-long-span bridges require service life significantly longer than common bridge and another reason is that improved solutions should be applied for the main elements that are not replaceable.

Since super-long-span bridges are usually more important than common bridges in economic and social aspects, higher risk should be guaranteed all along a longer design life. However most of the previous design methods do not explicitly mention the service life.

Since super-long-span bridges are expected to have higher demands in steel not only for the cables but also for the girder, deeper attention should be dedicated on the lifetime design. However, the 2006 National Bridge Inventory data in USA recorded no less than 73,798 bridges as structurally deficient, that is approximately 12% of the bridge stock. These data revealed that the structural deficiency rate is substantially greater for steel bridges than concrete bridges with a value for steel bridges 5.2 times that of prestressed concrete bridges and 2.7 times that of conventional reinforced concrete bridges.

On the other hand, bridges are maintained throughout a series of repairs and replacements of the members and the end of the lifetime occurs when maintenance cost exceeds newly construction cost. For instance, the failure of the I-35W Bridge in Minneapolis on August 2007 may be cited as an example of the user costs associated with the replacement or repair of a major bridge located in urban area. The Minnesota Department of Transportation established a user cost related to this failure of $400,000 per day. This cost reached $165,200,000 for the 413-day period that the bridge was out of service. This cost remained relatively low because of the bridge was replaced within a very short period of 11 months. In the case of super-long-span bridges, the construction costs to be burdened will reach a tremendous level regard to the importance of the bridge as unique access and the time required for its construction. From such a point of view, a service life exceeding 200 years is obviously necessary.

Due to these evident characteristics of super-long-span bridges, several limitations of the current practice can be highlighted as follows.

3.2.1 *Lack of quantitative indicator and assessment methods*

Common design codes suggest implicitly 50-year (Eurocode, ISO) and 75-year (AASHTO) of design life. But these codes fail to specify explicitly a target service life and show no numerical basis (Gehlen et al. 1999; Sarja et al. 1996). The limited consideration of extended service life may partially be the result of the lack of clear specifications on this topic. Different service life levels might be appropriate in future specifications depending on factors like initial cost, importance, and average daily traffic.

Researches on the quantitative prediction of design life for RC bridges have been performed and applied. The ACI Life-365 program (ACI Committee 365 2000) and DuraCrete Project (DuraCrete Report 2000) are representative examples. These approaches use numerical models of the deterioration mechanism of RC member to clarify chloride ion properties near reinforcement steel. If the calculated chloride concentration exceeds the limit concentration for steel corrosion it is assumed that the failure has occurred. And the corresponding time is considered as design life.

The DuraCrete approach has been developed during a European research project (1996–1999) and is internationally the only available probability-based service life approach. This approach computes proper design details such as minimum cover depth and maximum chloride diffusion coefficient guaranteeing the service life of the concrete structure. Chloride ion ingress model based on Fick's 2nd law is adopted to determine limit state function. Selected design details confirm target reliability (i.e. target failure probability) for expected design life. Specified environmental categories are also considered to represent

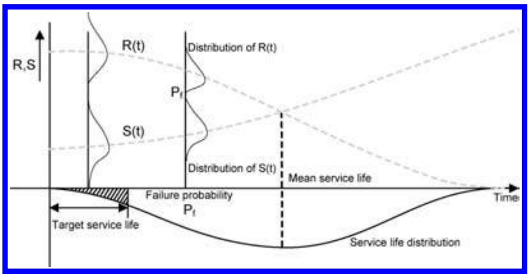

Figure 8. Example of target service life vs. failure probability.

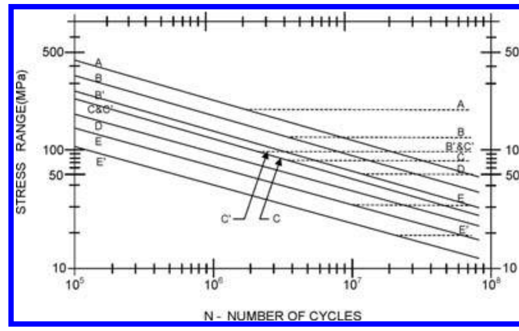

Figure 9. S-N fatigue curve.

severe environment conditions. Coefficients of concrete properties are acquired by experiment and not by theoretical basis to enhance the accuracy.

This approach is used for Busan-Geoje Fixed Link. The environmental conditions of the bridges and immersed tunnel are classified into 3 zones that are splash zone, submerged zone, and atmospheric zone, and limit states are defined as the time when chloride ions reach the reinforcement. Accordingly, the cover depths and chloride diffusion coefficients are determined so that the probability of those limit states to be violated (i.e. failure event) remain less than 10% for 100-year life (Daewoo E&C 2005).

However, these methods present some lack for being applied to super-long-span bridges such as model imperfections of deterioration model due to complexity of deterioration mechanisms. According to Luping (2007), even DuraCrete method which uses simplified deterioration model for RC can overestimate the actual life of structure. This imperfection error can be neglected for bridges with relatively short design life but may become significant for prolonged design life and increased importance of super-long-span bridges. This error can be lessened by using improved deterioration models which are more sophisticated even though the analyzed lifetimes from those models also show some differences with actual experimental results. There are some researches to provide a correct model based on the actual physical or electrochemical processes considering distance from the surface, time, the interactions between chloride and cement hydrate, the influence of moisture transport in parallel with diffusion within the concrete, etc. (Nilsson 2009). Although these types of model would be better than those based on simple Fick's 2nd Law, these methods usually require finite lamina time stepped calculation to solve the sophisticated mathematical equations that are hard to apply field application directly. Accordingly, research should be implemented to provide more reliable and practical models.

On the other hand, the development of assessment methods usually focused on the deterioration of concrete and reinforcement steel and, not on steel element. Even if RC member is important in pylon or pier of super-long-span bridges, the importance of steel member such as the non-replaceable girder is more crucial due to the increased span length. It means that deterioration model for steel members according to time must be established and verified to assess the design life. The long history of concrete allowed the development of model with a relatively high degree of reliability. But steel members need more verifications and experiments. The establishment of a reliable deterioration model for steel would enable to suggest quantitative design life using reliability-based design.

Protection methods such as CP or painting system need also to be quantified. Those methods are not expected to last for 200 years without additional maintenance. Periodic monitoring system and maintenance are essential. Currently, the expected life of these protections is not suggested quantitatively but implicitly. For reasonable maintenance planning, durability and reliability of each protection system should be verified.

3.2.2 *Fatigue load problem*

The Korea Bridge Design Code (KBDC 2008) suggests 2 million cycles for fatigue life of reinforcement assuming a 50-year life. If the fatigue life is extended to 200 years, the number of cycles to be considered will increase which leads to decrease of allowable fatigue stress as shown in the following S-N fatigue curve.

This result can be verified using the following equations. For load-induced fatigue considerations, each detail shall satisfy:

$$\gamma(\Delta f) \le (\Delta F)_n \tag{1}$$

where γ = load factor for fatigue limit state; (Δf) = live load stress range due to the passage of fatigue load; $(\Delta F)_n$ = nominal fatigue resistance (MPa).

Nominal fatigue resistance:

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{1/3} \ge \frac{1}{2}(\Delta F)_{TH} \tag{2}$$

where $N = (365)(t)n(ADTT)_{SL}$; t = design life (year); A = detail category constant taken from Table 2 (MPa^3); n = number of stress range cycles per truck passage; $(ADTT)_{SL}$ = single-lane ADTT (Average Daily Truck Traffic); and $(\Delta F)_{TH}$ = constant-amplitude fatigue threshold in Table 2 (MPa).

In Equation 2, the nominal fatigue resistance decreases as the total number of fatigue load passage N increases. Since N is proportional to design life t,

Table 2. Detail category constant, A, and constant-amplitude fatigue threshold $(\Delta F)_{TH}$.

Detail category	$A \times 10^{11}$ (MPa)	Threshold (MPa)
A	82.0	165.0
B	39.3	110.0
B′	20.0	82.7
C	14.4	69.0
C′	14.4	82.7
D	7.21	48.3
E	3.61	31.0
E′	1.28	–

the fatigue resistance would decrease if the design life changes from 100 years to 200 years.

In general case, fatigue load is not the governing factor. But as allowable fatigue stress decreases due to increased design life, fatigue may become a governing factor and should thus be verified.

In addition, the modified design fatigue truck according to extended fatigue life affects the reliability index. The limit state function based on Miner's rule for fatigue failure is (Nyman 1985):

$$g = D_f - \frac{Vt}{N_D}\frac{1}{S^3}(MGIH)^3 L_0 \tag{3}$$

where D_f = damage to cause failure; V = ADTT; t = design life (day); N_D = design number of cycles; S = stress range ratio; M = moment ratio; G = distribution factor ratio; I = impact factor ratio; H = headway ratio; and L_0 = loadmeter value. These random variables (D_f, V, S, M, G, I, H, L_0) are determined as shown below.

$$S = \frac{S_{rt}}{S_D}, M = \frac{M_k}{M_D}, G = \frac{G_k}{G_D}, I = \frac{I_k}{I_D}, \\ H = \frac{H_k}{H_D}, L_0 = \left[\left(\sum_{i=1}^{n}\frac{\omega_i}{\omega_D}\right)\frac{1}{n}\right]^3 \tag{4}$$

where D = nominal design value; k = k-th bridge site; S_{rt} = true stress range; ω_i = weight of i-th truck passage; ω_D = gross weight of design fatigue truck; and n = total number of trucks.

In Equation 3, the limit state function for the fatigue failure depends on the fatigue life t. As the design fatigue life increases, the function value g decreases, which results in the decrease of the reliability index. Moreover, as the design fatigue life increases, the COV's of the random variables M and L_0 may increase due to further uncertainty and consequently the reliability index may decrease additionally. These results show that the fatigue truck loads may be the governing factor with a 200-year extended fatigue life.

Together with theoretical investigation, the fatigue stress of steel needs also to be experimentally verified. Especially, the fatigue life of improved steels such as HBS and HBSW that will likely be adopted in super-long-span bridges should be imperatively verified to achieve a 200-year design life.

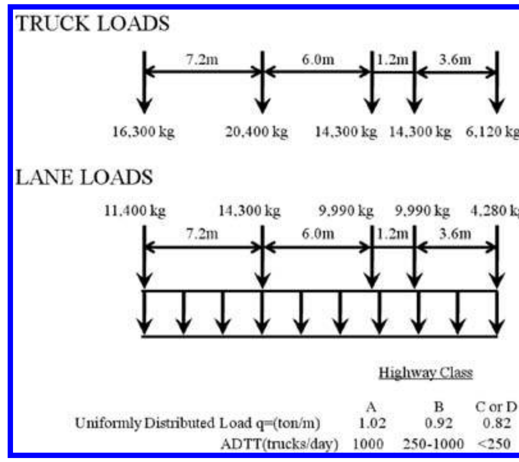

Figure 10. The highway live load model in Canada (OHBDC 1983).

4 VEHICULAR LIVE LOAD

Vehicular live load model used in the design of super-long-span bridges has to be determined according to the truck weight and traffic characteristics in the nation or region where the bridges are located. Because this vehicular live load is highly uncertain factor for the lifetime of the bridge, a probabilistic approach is known to be the most reasonable choice to determine the live load model (Nowak 1993). In the AASHTO LRFD design specification (AASHTO 2004), the HL-93 load model proposed by Nowak (1993) is used. The HL-93 load model is determined based on the maximum load effect that bridges endure for the lifetime of 75 years. The maximum load effect is estimated by truck surveys, weigh-in-motion measurements, etc. It is assumed that extreme vehicular load event is related to the distribution of only heavy trucks. Accordingly, only probability distribution function of upper 20% total truck weight are assumed as normal and the 75-year maximum truck weight is estimated by linear regression on the probability paper. In addition, OHBDC (Ontario Highway Bridge Design Code, 1983) specifies the highway live load as the larger of either a truck load or a lane load as shown in Figure 10. OHBDC truck model is based on commercial vehicle weight surveys conducted in Ontario from 1967 to 1975. OHBDC truck weights 700 KN which is significantly more than AASHTO standard truck.

Meanwhile, DB-24 and DL-24 load models used in KBDC (2008) were developed on the basis of HS20-44 load model of AASHTO in the 1970s. However, because of the growth of the commercial traffic and the increase of truck weight, new vehicular live load model should be developed. Therefore, KBRC (Korea Bridge

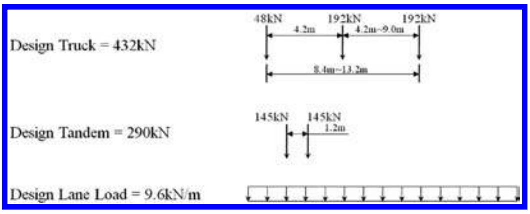

Figure 11. The new live load model in Korea (Hwang 2009).

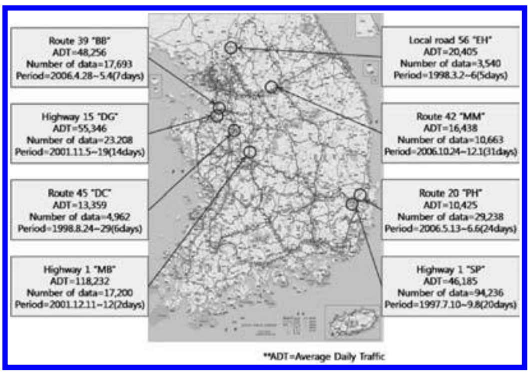

Figure 12. Sites installed with BWIM in Korea.

Design & Engineering Research Center) proposed a new vehicular live load model (TL truck load) based on the traffic data of Korea. The new model is shown in Figure 11 (Hwang 2009).

The procedure adopted for the development of the TL truck load model is similar to that of Nowak's study (Nowak 1993). In order to establish statistical database of traffic load, BWIM (Bridge Weigh-In Motion) system is installed at 8 sites in Korea (Figure 12). From the obtained data, the representative types of vehicles are chosen and the probabilistic distribution function of total vehicle weight is determined. The maximum load effect is estimated and TL truck load model is proposed. Differently from Nowak's study, the Extreme Type I distribution was identified to be more proper than the normal distribution by analyzing the probability distribution of upper data (Hwang 2009). Thereafter, 100-year maximum truck weights are estimated by linear regression on the probability paper.

In the current stage, the two vehicular load modes are compared. Then, we have to consider if the two load models can be applied to super-long-span bridges with a 200-year lifetime. There seems to be three major issues about this problem.

First, the HL-93 load model is made by assuming that lifetime of bridge is 75 years and the TL truck load model of Korea is made by assuming that lifetime of bridge is 100 years. If we design the super-long-span bridge which has lifetime of 200 years, 200-year maximum truck weight should be estimated by linear regression on the probability paper.

However, it is still questionable if this simple extrapolation will be able to predict the actual maximum

Figure 13. Route of the seaside roads and long-span bridges to be constructed in Korea.

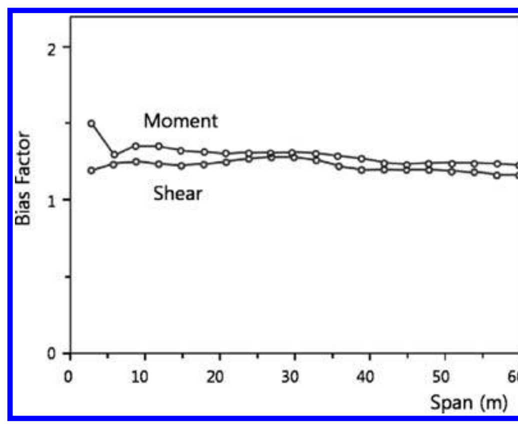

Figure 14. Moment and shear bias factor by span length (HL-93).

load in 200 years or not. In view of the current trend, trucks are becoming heavier but one cannot affirm that this trend will continue. The necessity for CO_2 reduction could lead to the shift from road transportation to rail transportation. The adoption of innovative materials in the vehicle industry may also lead to lighter trucks. Therefore, the future transportation system should be considered with the estimation of current vehicle load.

The second issue lies in the data collection area. The data collection areas (Figure 12) selected in order to make TL load in inland region, but the regions where long-span bridges will be constructed are likely coastal areas (Figure 13).

It is anticipated that there will be substantial difference between the information collected in the current inland region and the vehicle characteristics of naval region, so numerous studies have been conducted in Korea to establish the statistical data for super-long-span bridges.

The third issue is that the current HL-93 load and the TL truck load model are made only for bridges shorter than 100 m. This implies that verification is required if constant bias factor (ratio of the mean maximum moment/shear and HL-93 maximum moment/shear) (Figure 14) is guaranteed for super-long-span bridges.

In the current Korean Design Code for Cable Supported Steel Bridge, legal enforcement specifies to apply reduced factor with span length

Figure 15. Bird view of Seohae Bridge in Korea.

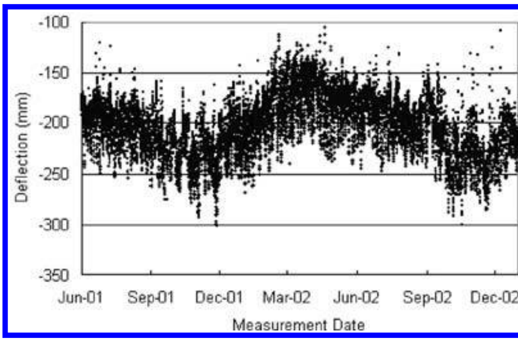

Figure 16. Annual variation of vertical deflection at the center of the main span of Seohae Bridge (Park et al. 2003).

(0.57 + 300/(500 + L)) to uniform load (Korean Design Code for Cable Steel Bridge 2006). ASCE Report (ASCE 1981) states that there are plenty of margins in the live load. Such margins were also been verified in Seohae Bridge (Figure 15) and Youngjong Bridge in Korea (Koh et al. 2008).

Specifically in the case of Seohae Bridge, the structural behavior of the cable-stayed bridge was observed and analyzed during the first 2 years following its completion. Results showed that the annual variation of the vertical deflection in the stiffening girder satisfies the allowable design limit and that the deflection due to live load represents only 25% of 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. Stress margin appears to remain considerable since measured stresses represent only 5 to 12% of the design stress. Accordingly, the actual highway bridge design specifications in Korea are producing excessively conservative structures (Park et al. 2003).

For these purposes, the Honshu-Shikoku Bridge Authority suggests cutting down the designed live load according to span length for designing super-long-span bridges, and the Korean Design Code for Cable Steel Bridge also applies the method reducing the uniform lane load when span length exceeds 200 m. But this cut down does not rely on measured truck data in Korea, therefore a rational live load model should be developed based upon the collected data in the future.

5 EXTREME EVENTS

Extreme events are also important features of concern in the design of bridges. Extreme event could be natural hazard like earthquake, landslide, typhoon, flood and fire, or manmade hazards such as bombing, collision and terror attack. The consequences of extreme events on the integrity of the bridge can be tremendous as we could see during the Kobe earthquake in 1995.

Predicting the effects, loads and eventual damages accompanying the occurrence of extreme event is a major task when designing bridges for extreme events. Therefore, the estimation of the likelihood of the occurrence of a given duration and intensity of the event through its return period is crucial. Risk assessment involves the prediction of the chances of a specific set of events to occur together with their consequences. The exceedance probability curve constitutes a helpful tool for decision-making.

However, long-span bridges are by nature and location more vulnerable to such extreme events. Ensuring a service life of 200 years means also that higher risk should be guaranteed. Accordingly, the current design method which adopts a probabilistic approach only in hazard analysis to determine occurrence rates of the extreme events and uses a deterministic procedure for the structural analysis should shift onto a fully probabilistic approach, i.e. consequence-based design may be needed.

5.1 *Features of extreme events in super-long-span bridges*

The consequences of accidents in super-long-span bridges would likely be much larger in terms of socio-economic losses than normal bridges. Besides, the extended lifetime required for such bridges increases the probability of the occurrence of accidents. Therefore super-long span bridge should be designed to satisfy for much higher risk level. In general a quantitative index of risk is used in extreme event design. The risk R is defined as follows:

$$R = \sum_{i=1}^{n} P_i C_i \tag{5}$$

where P_i = probability of i-th accident within a given time ($i = 1, 2, \ldots, n$); C_i = consequence of i-th accident in the given time; and n = number of possible accidents. Super-long-span bridges have higher risk than that of common bridges because of their extended lifespan, the higher occurrence of possible accidents (n) to happen and the larger consequences such as cost of repair or failure (C_i).

In consideration of the social and economical importance of super-long-span bridges, the societal acceptable risk should be regulated strictly. The societal acceptable risk is based on FN (Fatal Number)

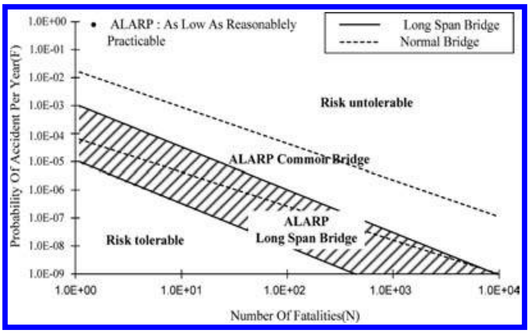

Figure 17. Example of super-long span bridges FN-curve.

curves. FN-curves are a graphical presentation of information about the frequency of fatal accidents in a system and the distribution of the numbers of fatalities in such accidents. They plot the frequency F(N) of accidents with N or more fatalities, where N ranges upward from 1 to the maximum possible number of fatalities in the system. Because the values of both F and N sometimes range across several orders of magnitude, FN-graphs are usually drawn with logarithmic scales. FN-curves themselves are simply a means of presenting descriptive information about fatal accident frequencies and fatality distributions. In this respect they are similar to histograms, and indeed present the same information as histograms in a different way.

These curves show three regions bordered by two curves in logarithmic scale. Values of F for high values of N are often of particular political interest, because these are the frequencies of high-fatality accidents. The curves are acceptable risk higher where risk could be accepted, lower where risk could not be accepted and, model where ALARP (As Low As Reasonably Practicable) where all possible measure should be undertaken to reduce the risk. These tolerable criteria vary case by case. Figure 17 illustrates an example of FN-curve where the criteria of risk level are adjusted for super-long-span bridges which are assumed to have 10 times more risk consequence than common bridges.

5.2 *Lifetime design against extreme event, current practice*

Extreme loads are different from ordinary loads and bear uncertainties, which require a probabilistic approach. Extreme loads are predicted probabilistically based on observed data. Especially, occurrence probability of natural disasters such as earthquake and typhoon is expressed as return periods, and some specifications (e.g. AASHTO LRFD) state return periods as much as 50 to 75 years. The question is how to manage these return periods for super-long-span bridges designed for a service life of 200 years.

5.2.1 *Earthquake loads for lifetime design*

For earthquake loads, the load is determined by means of the average return period exceeding a probabilistic threshold value during the specified period. In the case

Table 3. Return period and risk rate of earthquake.

10% of exceedance within a given lifetime		
Design life (years)	Return period (years)	Risk rate
5	48.0	0.40
10	95.4	0.57
20	190.3	0.73
50	475.1	1.00
100	949.6	1.40
200	1898.7	1.89

Figure 18. Seismically over-designed piers and corresponding arrangement of reinforcing bars.

Table 4. Basic design wind speed on design life increase.

0.6 of Non-exceedance probability		
Design life (years)	Return period (years)	Basic design wind speed (m/s)
5	10.3	25.9
10	20.1	28.6
25	49.4	31.9
50	98.4	34.5
100	196.3	37.0
200	392.0	39.5

where identical risk rate is accepted, the earthquake load is determined by the product of the seismic site coefficient and risk rate.

Table 3 presents the return period of the earthquake with respect to the design life. It can be seen that earthquake load for a 200-year service life is 1.35 times than that of 100 years. Simple application of the load would imply that stiffer foundations and piers should be designed to resist increased earthquake loads, which consequently will lead to increase of the construction costs and loss of constructability due to the larger amount of reinforcing bars to be arranged (Figure 18).

5.2.2 *Wind loads for lifetime design*

In the case of wind loads, the aerodynamic stability should be secured through the stability of the vibrations induced by the dynamic loads. Of equivalent importance, need is also to obtain the material property which resists to static wind loads. At this time, static wind loads increase with the rate of lifetime as shown in Table 4.

Because F_D (Static wind loads per unit length) is linear to the square of the design wind speed, design

Figure 19. Rendering of Gwangyang Bridge with main span of 1545 m in Korea.

wind loads of 100 and 200 years are determined as follows.

$$\frac{F_{D,200}}{F_{D,100}} = \frac{(U_{200})^2}{(U_{100})^2} = \left(\frac{39.5}{37.0}\right)^2 \approx 1.14 \tag{6}$$

Accordingly, a 200-year lifetime design implies that the design wind load should be increased by about 14% compared to that of 100 years. Since design wind load generally governs the design of the superstructure of long span bridges, higher design wind load requires higher resistance strength of the girder. Moreover, higher design wind speed needs also larger flutter onset velocity, which is related to the vibrational characteristics and shape of the cross-section. Therefore, increased wind speed implies strengthening of the girder and improvement of the cross-section.

Gwangyang Bridge provides a good example of the optimization of the cross-section of the girder for the satisfaction of the flutter onset velocity. Gwangyang Bridge will cross Gwangyang Bay and will connect the cities of Gwangyang and Yeosu passing through Myodo Island in the southern coast of Korea. The completion of the bridge is scheduled to happen in 2012 just before Yeosu Expo 2012. The Gwangyang-Yeosu area being located in the southern seashore is regularly swept by typhoons in summer. During the Typhoon Maemi in 2003, a maximum instantaneous wind velocity of 49.2 m/s was recorded at the Yeosu Weather Station far away by about 18 km from the bridge site. Considering that the bridge superstructure will rise at 81 m above the sea level, high wind is likely to occur at the deck. The critical wind velocity of Gwangyang Bridge is determined 81.6 m/s. Therefore, the aerodynamic stability constitutes a main concern in the design of the bridge. The flutter onset velocity has improved from 67 m/s of initial cross section to 120 m/s after optimization of cross-sectional shapes (Kwon 2008).

5.2.3 *Ship collision prevention for lifetime design*

Risk of vessel-bridge collision is decided according to the magnitude and quantity of the vessel passage on the bridge site. The magnitude of design vessel and the passing quantity can be forecasted using linear extrapolation of statistical data. For example, the emergence

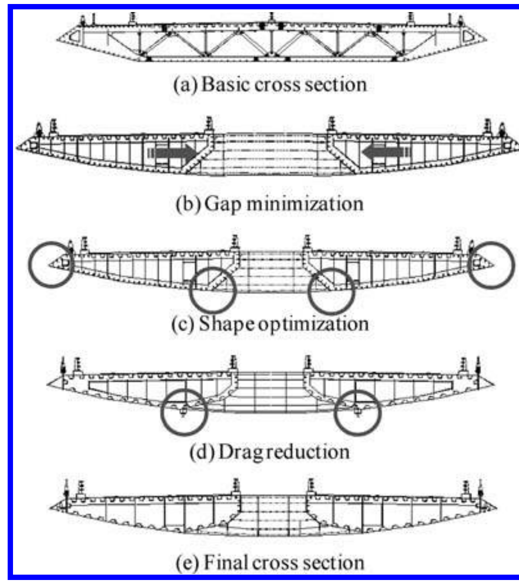

Figure 20. Gradually optimized cross sectional shapes of Gwangyang bridge in Korea.

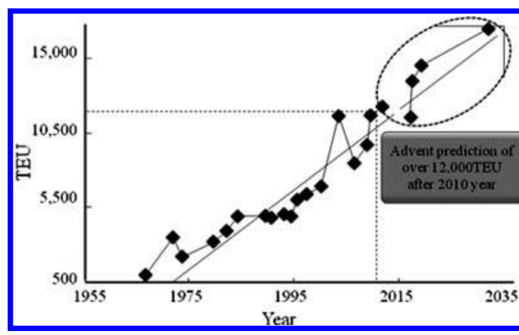

Figure 21. Extrapolation of an extra-large vessel growth.

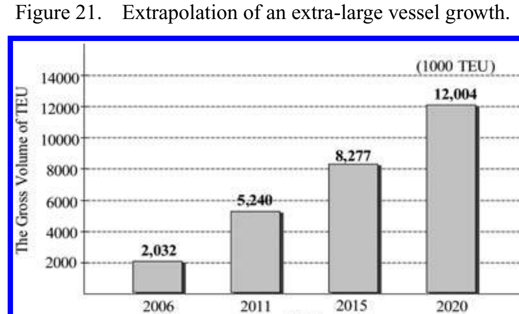

Figure 22. Harbor facility development plan in Korea.

of supersized cargo ship (Figure 21) and the harbor facility development plan (Figure 22) constitute very important data which can influence the design values.

Various risk assessment models have been developed about collision between vessels and bridges. Most of these models rely on formula (7), which calculates the annual frequency of bridge elements' collapse (AF). The inverse of AF ($1/AF$) is equal to the return period (in years). For a specific member, the sum of all the AFs computed for each class of vessels corresponds to the annual frequency of collapse of this specific structural element (PIANIC 2001).

$$AF = N \cdot PA \cdot PG \cdot PC \tag{7}$$

where AF = annual frequency of bridge element collapse due to vessel collision; N = annual number of vessels classified by type, size, and loading conditions which can strike the bridge element: PA = probability of vessel aberrancy; PG = geometric probability of a collision between an aberrant vessel and a bridge pier or span; PC = probability of bridge collapse due to a collision with an aberrant vessel.

According to this formula, the annual frequency of bridge collapse depends on the predictions of the future annual number of vessels (N) and vessel size. Moreover, if design is performed assuming a higher level of risk for extended lifespan of the bridge, a smaller target AF should be adopted in the design. Since the values of PA and PG cannot be controlled or adjusted by the designer, the value of PC must be reduced to strengthen the resistance capacity of the bridge elements to satisfy the current design specifications.

Supplying additional protection facilities like dolphins constitutes another way for reducing the collapse probability. The type and size of the ship collision protection facilities depend on the impact loads of design vessel.

Both options have naturally direct influence on the construction costs. Therefore, for more economical design against vessel collisions the pre-estimation of future vessel type, size and quantity should be carefully considered.

5.3 *Limitations of the current practice in super-long-span bridges*

As mentioned earlier, if the design loads are extrapolated for 200-year lifetime using the current practice, the resulting structures are likely to be excessively overdesigned. Especially, the three types of extreme events considered above have been seen to influence different parts of bridge. That is, earthquake loads govern the design of the substructure, wind loads determine the design of the superstructure and vessel collision influence the design of the piers or require additional protections. Besides, if more stringent criteria are applied in the design of the structural parts, the feasibility of the construction project can be ruined due to the loss of economical efficiency brought by larger sections and stronger structural elements.

Since ship collision assessment requires probabilistic approach because of its rareness, previous design specifications such as AASHTO LRFD suggest methodologies based on a probabilistic approach. However, the lack of statistical data impedes direct application of these methodologies to the design of super-long-span bridges for the following reasons.

The AASHTO LRFD suggests base rate of probability of ship aberrancy (PA) as simple values based on limited historical ship collision data of US waterways. Geometric condition, effects of current and traffic volume are considered in the calculation of PA. However, the "human error" factor, which represents the most important factor in ship aberrancy, is not reflected sufficiently in the current assessment methodology of ship aberrancy probability. Geometric probability (PG) which is concerned with the model of the location of an aberrant vessel and the collision velocity also need to be improved. Based on historical bridge collision data of USA, a normal distribution is assumed to model the location of an aberrant vessel which cannot represent actual aberrant ship's path with accuracy. The same observation can be done for the ship collision velocity. Calculated ship collision risk of super-long-span bridges may differ from actual value because of the lack of available statistics and may result in erroneous designs of the bridges.

Another limitation of the current practice concerning ship collision is the formula adopted to estimate the ship collision force. The impact force formula proposed in AASHTO LRFD is based on studies dedicated to the collisions between ships (AASHTO 1991). In case of ship-ship collision, the absorption of kinetic energy due to large deformations of each ship is the main mechanism. However ship-bridge collision cases are ship-to-rigid-body collision, which results in the underestimation of the impact force by the formula of AASHTO (Petersen et al. 1993; Svensson 2006).

The prediction or estimation of extreme events that may occur in the future is very difficult and involves a wide range of variables. Even though a probabilistic approach is adopted, the corresponding prediction is likely to lose its accuracy as much as the considered design lifetime lengthens. In current practice, simple extrapolation will increase extreme load and make it a governing load in the structural design. Therefore, change of the design paradigm should give priority to the economy and sustainable design against extreme events. Under extreme loads the consequence of damages are more important than other criteria related to the deflection or serviceability. As a result, when extreme events are considered in design, the design paradigm should shift toward risk-based design or consequence-based design to minimize the consequences and damages.

Innovations are needed to transcend the current practice and to devise reasonable approaches in coping with extreme events. In case of natural disasters such as earthquakes and typhoons, the probabilities of extreme events are site-dependent. Careful site investigation should thus be conducted to assess realistically and reasonably the probability of the uncertainties related to these events. In the case of Honshu-Shikoku Bridge Authority, the special windproof design standard for Akashi Kaikyo bridge was established in 1990. Design specifications should prescribe the application of improved structures such as airfoil shaped sections and multi-box girders used in Messina Bridge or Gwangyang Bridge so as to cope efficiently with extreme events. Solutions like seismic control devices against strong earthquakes and gales should also be developed to be applicable for super-long span bridges.

In addition, in case of manmade disasters like ship collisions, cooperation with other industrial fields should also be accounted. As an example, recent navigation technology of ship has experienced tremendous evolution. GPS and laser range finder can reduce drastically the occurrence of ship-bridge collision. In Korea, the volume of vessel has recognized an increase of 800% within two decades. But the annual ship collision frequency has just increased less than two times (KMI 2003). This proves that manmade disasters can be reduced relatively easily by technical tools.

6 SUPER LONG SPAN BRIDGE R&BD PROJECT

The Ministry of Land, Transportation and Maritime Affairs (MLTM) of Korea has set up the VC-10 (Value Creator-10) Program for Construction Technology Innovation. Technology Road Map for each program was established by experts from academia, research institutes and various fields of construction industry. One of the programs is "Super Long Span bridge" R&BD project, launched in 2009 under the supervision of Korea Expressway Corporation, which defines four major programs: (1) development of key technologies for planning and design of long-span bridges, (2) development of high-performance cable systems and high-performance materials for long-span bridges; (3) development of highly efficient and innovative cable erection equipments and methods, and cost effective construction methods for offshore mega foundations; and (4) development of IT-based disaster mitigation and maintenance technologies.

Lifetime design is one of the most important themes of the project. To that goal, research is actively conducted in diversified fields involving materials, live loads and extreme events as discussed in the previous chapters. The detailed researches of each program related to the features discussed above are as follows.

Live load models will be developed to be suitable for super-long-span bridges. For that purpose, measured truck data of domestic bridges have been collected and analyzed. System reliability assessment methodology and reliability based load combination method using the developed model will also be developed through this project. The final outcome of this topic is the development of reliability-based design specifications for super long span bridges.

Related to the extreme events, the extreme loads caused by earthquakes, typhoons or ship collisions will be defined more specifically and reasonable design solutions will be suggested. As mentioned in section 5.3, the current probabilistic approach adopted for the assessment of ship collision presents several limitations due to the lack of statistical data. Accordingly, improved methodology for the assessment of ship aberrancy probability and geometric probability has been studied using ship maneuvering simulation. Cho (2009) suggested a probabilistic analysis methodology using 3D ship maneuvering simulation data to improve the accuracy of the results. The simulation data for the given ship and bridge location reflect more accurately the effects of various weather/current conditions and even human factors on ship collision accidents. However, the time and cost required to perform such simulations constitute also an obstacle in gathering a sufficient number of simulations to establish reliable probabilistic model of aberrant path and collision velocity. For that reason, random vector generation method from multivariate normal distribution is applied to the limited simulation data in order to generate and acquire sufficient and meaningful data. This methodology makes it possible to calculate ship collision risk more accurately over the long service life. Besides more convenient to use and more reliable methodology have been developed using fast-time simulation with advanced auto-pilot function. The collapse mechanism of ship-to-pier according to various types of bridges and ships will also be investigated. In addition, a ship collision risk assessment program including dolphin protections will be developed. An expected outcome of the program will be the optimal design and placement of dolphin protections.

These results from the researches about extreme events will also be applied to prepare for the design specifications for super long span bridges together with the development of aerodynamic cross-section of cable-supported bridges. Disaster management system will be developed for a series of disaster scenarios. Based on those scenarios, disaster factors will be monitored in real time, and the damages will be analyzed accurately by probabilistic risk assessment technique. The so-gathered disaster information will be exploited for bridge users' safety as well as for the implementation of immediate actions.

Other important topic of the R&BD project is the durability improvement of materials and bridge systems. HSB800W steel featured not only high strength and improved fatigue performance but also improved weathering performance is under development. Developments of zinc-aluminum alloy coating method, dehumidification system and high-durability painting method have also been performed to enhance corrosion resistance of high strength steel cable for super-long-span bridges. Increase of cable material durability itself is also supposed to be attained by using FRP cable. Developed FRP cable and anchorage systems will show improved fatigue performance and durability. These researches will contribute to increase material durability and eventually to secure 200-year design life of super-long-span bridges. Details of the assignments can be found in the website (www.longspanbridge.org).

7 CONCLUSIONS

This paper reviewed current practices for service life design and addressed issues that will be faced in the lifetime design for cable-supported super-long-span bridges. Research directions that will help designers in achieving extended lifetime of 200 years or more for the super-long-span bridges have been derived.

The main issue when deciding upon a specific service life is to clarify the event and feature, which will identify the end of the service life. In view of the current practice, service life of 100 to 150 years seems to be achievable and has already been implemented in recent major bridges owing to special specifications by the owners of the bridge projects.

The lack of specification provisions prescribing explicitly the length of the service life is an obvious obstacle to the implementation of extended service life for major bridges. Even though each long-span bridge is unique, the development of relevant specifications providing sufficient margins should be implemented for the designers to guarantee the service life of their structures.

This paper addressed three major topics for which limitations of the current practice are likely to happen for the design of super-long-span bridges in the future. These topics are material durability, live load model and extreme events. Currently, design practice assumes that securing material durability for 200 years will guarantee equivalent or longer lifetime of the bridge. However, one cannot assume the reliability of such approach considering the harsher conditions in which super-long span bridges will be exposed. Accordingly, the application of high performance materials together with appropriate protection alternatives should be seriously conceived and dedicated research should promoted. In the case of load models including live loads and extreme events, the need for probabilistic approach has been highlighted. Should the designer assume higher risks or maximize safety with more conservative design? Although further study should be conducted, the authors estimate that the cost-effectiveness of the structure is a design priority. This means that, from a lifetime perspective, a certain level of risks should be accepted within a probabilistic framework so as to ensure both reliability and cost-effectiveness.

Consequently, extending the service life for bridge structures and especially super-long-span bridges requires a paradigm shift in the current design practice. Rather than focusing unilaterally on material durability, the extension of service life should rely on more systematic lifetime design procedures involving probabilistic approach, consequence-based design and some marginal increase of costs.

ACKNOWLEDGEMENTS

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Managing old bridges

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1 INTRODUCTION

In the design practice, we are used to refer to recurrence intervals of centuries. For instance, in the European countries, 200 years is the return period usually assumed for floods. But, if we consider the real life of a structure, how long do two hundred years last? In fact, two hundred years ago, the material characteristics, the building technologies and the theoretical know-how were very different from the present ones.

As regards to the materials, for instance, the industrial process to produce cast iron was invented at the end of eighteenth century, while a steel with modern characteristics was patented at the middle of 1800. The first experiments on reinforced concrete were carried out in the second half of the nineteenth century. Mörsch published his famous book Der Eisenbetonbau in 1902, (Mörsch 1902). The first applications of prestressed concrete structures were proposed by Freyssinet in 1928 (Freyssinet 1950).

At the same time, the nineteenth century also gave us the basis of the strength of materials and of the theory of structures (Timoshenko 1953). In particular, with the growing use of steel in structures, more complete investigations of truss structures became necessary and the first methods to deal with statically indeterminate structures were proposed.

The great arch and truss steel bridges date back to those times, while the first long span reinforced concrete bridges appeared in the first decades of the twentieth century (Trojanović 1960; Leonhardt 1982).

An epic history of the greatest cable suspended bridges and of their designers was written by Henry Petrowsky (Petrowsky 1996).

Dealing with old structures, we cannot avoid framing them into the context of their time, before assuming any sort of decision and/or choosing any sort of intervention.

The major aspects that must be taken into consideration usually regard:

- Possible changes in the geo-morphology of the territory where the bridge is located, due to settlements and to variations in riverbed geometry and in bathymetry conformation).
- Possible changes in the attitude of the bridge, made evident by vertical irregularities of the road level, by rotations of the piers or by excessive vertical displacements of the deck.
- Conservation or damaging states of the structural elements and of the joints, of the bearing supports and of the auxiliary devices.

All these issues concern environmental influences on the bridge service life. The loads consist of self weight, of the stream flow and of the wind. The distortions may be caused by settlements of the foundations of the piers and of the abutments.

But a bridge is primarily a transportation link. Hence, many bridges modifications are caused by new traffic needs. Again, we remain struck by the strong differences among the traffic loads given by present codes and those assumed at the time of construction: for instance, one hundred years ago, the maximum load for road bridges were 13.5–15 kN/m and the maximum rail bridge loads were 30–60 kN/m. In those times, the speed was also lower and usually two lanes only were sufficient.

This paper gives an account of studies and rehabilitation works carried out on a group of bridges located in the North of Italy and belonging to the main typologies used in the years between 1850 and 1970. Part of these bridges lie on the reach of the Po river which delimits the southern border of Lombardia.

The others bridges cross minor rivers.

Recalls will be made on the surveys and monitoring activities and on the problems addressed during the rehabilitation works.

Synthetic descriptions of the main interventions on the foundations, on the main structure and on the special devices will be given.

2 BRIDGES AND ENVIRONMENT. SURVEY OF A GROUP OF BRIDGES ALONG THE PO RIVER

2.1 *The Po basin*

The Po is the main Italian river. It rises in the western Alps and flows to the Adriatic sea, 652 km away. Its hydrographic basin is 74,970 km^2 wide and receives 43 tributary rivers. The total length of the embankments is 3564 km.

With the exception of the upper reach, until the end of nineteenth century the crossing of the Po was carried out through river ferries and floating bridges. Due to the wide span of its main branch, traditional masonry

Table 1. Characteristics of the main bridges examined (see Figure 1.a).

	River	Place	Year of construction	Total length [m]	Length over the river [m]	Piers in flood-plains	Piers in riverbed	Pier and foundation types
1	Po	Casalmaggiore	1958	1,205.38	580.00	3 + 25	6	Pile/pier type
2	Po	Viadana	1967	1,670.00	734.00	36 + 6	5	Hexagonal columns on piles
3	Po	Borgoforte	1961	1,137.21	471.83	9 + 3 + 0	4	Pile/pier type
4	Po	San Benedetto Po	1964–66	613.00	613.00	5	4	Double blade piers
5	Po	Ostiglia	1929/1947	511.00	511.00	1	5	Masonry piers
6	Po	Piacenza	1908/1947	1,096.00	607.00	12 + 5 + 3	2	Masonry piers
7	Serio	Montodine	~1970	64.00	64.00	0	1	Masonry piers
8	Oglio	Pontevico	~1970	90.00	90.00	0	2	Pile/pier type
9	Oglio	Sarnico	~1970	87.00	87.00	0	5	Pile/pier type
10	Oglio	Montecchio 1	~1970	270.00	270.00	9	2	Circular piers on piles
11	Oglio	Montecchio 2	~1970	90.00	90.00	1	2	Circular piers on piles
12	Oglio	Breno	~1970	403.00	403.00	15	2	Circular piers on piles
13	Po	Pieve Porto Morone	1961	1,250.00	1,250.00	10	5	Rectangular r.c. columns on piles
14	Po	Becca	1912	1,040.00	1,040.00	3	9	Masonry piers

and stone arch bridges involved limited spans resting on a high number of piers, having basements in an insidious and wandering riverbed.

The first long span bridges appeared at the beginning of the twentieth century with the diffusion of the steel-truss girders. Many of these road and rail bridges, although after some reconstruction work, are still operating. After WWII, many new bridges were built, with a wide use of prestressed reinforced concrete. The main challenge during the design and erection phases was not represented by the spans (usually 50–70 m on average), but by the interaction with severe fluvial hydraulics, characterized by cyclic floods, which often overtopped the embankments, invaded the floodplains and sometimes upset countries and villages at the two sides of the river.

We have a fairly complete knowledge of the historical floods of the Po river since a remote age. The main floods of the last hundred years occurred in 1926, 1951, 1994 and 2000.

2.2 *A wide surveying campaign*

After the 2000 flood, the Compartment for the Lombardia region of the Italian Agency for Roads (ANAS) promoted a campaign aimed to survey the state of the piers, of the basements and of the foundations of the main bridges crossing the Po. Such a campaign was meant to provide a first evaluation of the bridges state and to detect possible critical conditions of the piers and foundations, paying particular attention to hydraulics causes of instability, like erosion and scour. The bridge main characteristics are listed in Table 1. Their locations are shown in Fig. 1a.

2.3 *Surveying and monitoring*

Surveying and monitoring consisted in the following activities:

- Preliminary exam of each bridge on the basis of the documents available in the archives.
- General visual inspection.
- Geometrical survey, with dimensional cross-checks with the original drawings. Detailed description of local damage states.
- Survey of the overall attitude of the bridge, with respect to horizontal and vertical references. Verticality checks of the piers. Levelling of the roadway.
- Bathymetric survey, in order to draw the geometry of the riverbed, to detect possible scouring signals near the piers.
- Underwater surveys, aimed to detect cracks or clefts in the submerged parts, as well as traces of erosions or scour holes at the piers basis.
- Vertical boreholes in the piers aimed to measure their buried depth. Echo-soundings, used to check the actual depth of the piers above the piles.
- Geognostic boreholes, with the performance of SPT and CPT tests and with collection of disturbed and undisturbed soil samples.

The relative cost of each survey activity is shown in Figure 1c. The mean cost of the surveys versus the overall length of the bridges is shown in Figure 1d.

2.4 *Laboratory and office activities*

The results of the laboratory tests carried out on the soil samples were used to define the load carrying capacity of the original foundation systems, as well as to design the strengthening works on the insufficient ones.

The office activities concerned in hydraulic engineering and structural assessments.

For each bridge a comprehensive report was compiled. These reports gave a final assessment of the actual state of the bridge and highlighted its possible critical faults. When the safety of the structure was at stake, suitable suggestions for urgent interventions were provided. Each bridge was classified and given a priority level which implied recommendation for ordinary or extraordinary maintenance activities, or, in the worst cases, for radical strengthening works.

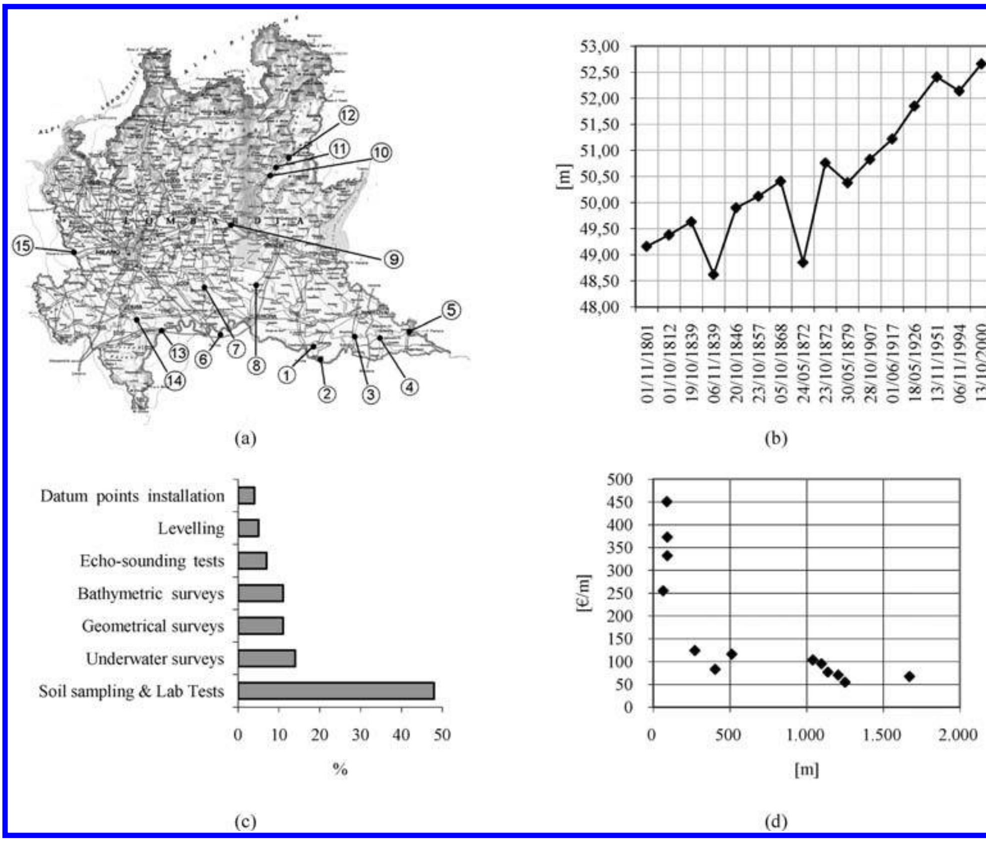

Figure 1. (a) Position of the main bridges listed in Table 1; (b) maximum hydrometric at the Piacenza gauging station in the last two centuries; (c) cost distribution for different types of test; (d) mean cost of the surveys per meter vs overall length of the bridges.

2.5 *Main results drawn from the bridge inspection*

Floods are a common experience for people who live at the Po riversides. Now, comparing the recent surveys with the historical data, it is possible to observe a rising trend of the maximum flood levels and, in recent years, a higher frequency of the flood events.

According to the experts of environmental hydraulics, such a phenomenon is mainly due to anthropic factors, like a progressive waterproofing of the basins, due to urban and infrastructural growing, the removal of expansion zones and the increase of river reaches confined by embankments. Climate changes may also have contributed to these effects.

Figure 1b shows the maximum hydrometric levels recorded by the Piacenza measuring station during last two centuries and confirm these remarks.

Another element which was confirmed is the depth of the scour in the rapid transient phase as determined according to the recent Po Basin Authority specifications, which agrees with the most widely recognized formulations (Hamill 1999). These values of scour depth appeared quite higher than those assumed in the past (Figs 2a, b) and strongly condition the load carrying capacity computation of the foundations and of the piles.

As regards to the body of the piers, it was not found in a bad situation. Some masonry piers presented losses of mortar among the masonry courses. Both masonry and reinforced piles presented traces of collision with the small boats and ships which sail the middle and final reaches of the river.

For the main bridges, the position and the orientation of the piers with respect to the main flow was judged correct. In some minor bridge, spanning over tributary rivers, some cases of wrong foundation basement were found (Figs 2c, d).

Another general consideration regards the soundness of our probabilistic design procedures, tuned to values of return periods (100, 200 years). The surveys of the most recent bridges, for which it was possible to compare the present riverbed profile with that of thirty/forty years ago, showed cases of strong riverbed changes, with the movement of the main current from one alignment to another and also with the growth of temporary islands downstream the old main current.

The underwater surveys reported that, even after a long time from the end of the flood event, a systematic encumbrance of debris remained at the basis and along the body of the piers. This is a problem of ordinary maintenance. But, who is in charge of the

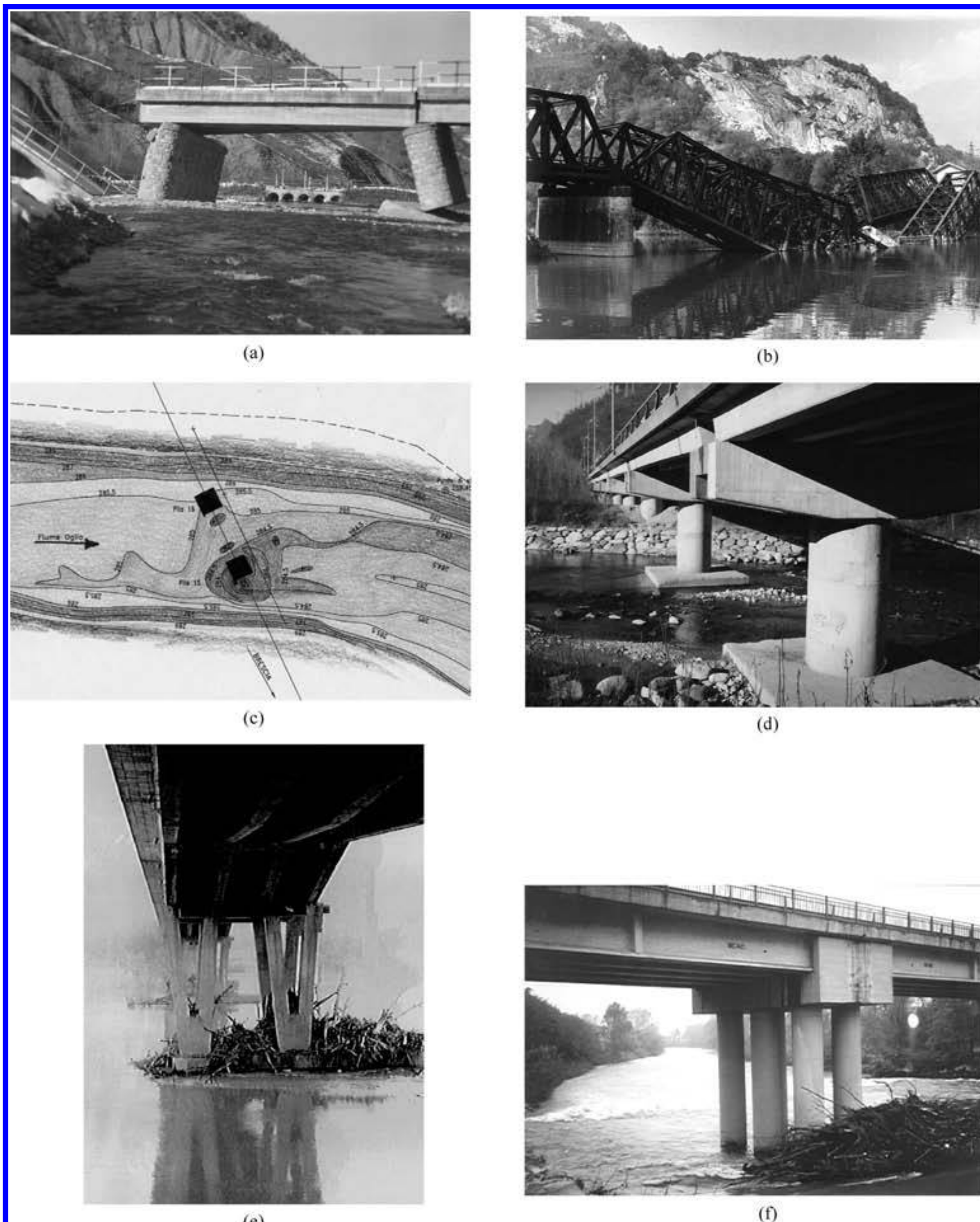

Figure 2. Scour effects and debris action on piers in the riverbed: (a),(b) collapses caused by the scouring action in bridges crossing river Po's tributaries; (c),(d) effects of the position, of the orientation with respect to the water flow, and of the depth of the foundations in a small river; (e),(f) Multi-columns piers acting as grids in the formation of debris rafts.

debris removal? Is it the river Authority or the bridge Authority? More simply: who has to pay?

3 THE NEEDED REPAIR INTERVENTION

Once the data acquired during the inspections and surveys had been analyzed, different repair intervention were set up. These interventions were carried out on:

– the foundation systems;
– the main structures of the bridges;
– their complementary or special devices.

4 INTERVENTIONS ON FOUNDATIONS

Nowadays, the need for an intervention on the foundations arises mainly after their stability has been checked according to new codes and regulations, which lead to higher depths of the estimated scour holes and higher values of the acting forces.

The strengthening interventions are usually based on two contributions. The first one consists in protecting the area surrounding the piers by means of big bags containing massive stones. Such a work stabilizes the riverbed and leads to less severe expected scour depths. The second intervention consists in strengthening the pier basement.

Usually the strengthening is carried out in one of these two ways:

– when the body of the pier is sufficiently compact and massive, like in the case of masonry piers, new piles are driven across the body itself;
– when this is not possible or when previous repairing interventions occurred in the volume of the pier, new piles are driven around the perimeter.

A common problem of both these type of intervention is the accomplishment of a robust connection between new and old structures.

An example of strengthening is presented in the following.

4.1 *An example of strengthening of the foundations of a bridge of the nineteenth century*

The Lodi masonry bridge cross the Adda River and was built in the years 1863-64, by the Milanese Architect Gualini. It is made of nine shallow arches spanning 18.90 m each, for a total length of 175 m (Fig. 3a).

In 1970–71, the deck of the Lodi Bridge was refurbished in order to comply with the new traffic loads (Figure 3b), and a weir was built 200 m downstream in order to protect the basement of the bridge. In the following years, the weir caused some floods involving the town riverside. This brought, after new hydraulic assessments carried out in 2006, to the decision of lowering the weir. This lowering, which is scheduled for the summer of 2010, will increase the flow velocity and therefore expose the foundations to more severe service conditions. After soil samplings and laboratory tests, a subsequent geotechnical and structural assessment showed that the foundation system was not safe even before a worsening of the conditions caused by the lowering of the weir would occur, and needed strengthening works.

This choice was confirmed also by the presence of some foundation settlements already observed in the past. In fact, some works had already been performed in 1947, when each foundation was strengthened and its basement was protected against erosion and scour.

The new intervention substantially recalls the old one. Twenty new micropiles, having a diameter of 0.20 m and 24.70 m long, were drilled alternatively at a $\pm 5°$ angle along the sides of the basement (Figs 3b, c). The micropiles were reinforced by valved steel tubes having a diameter of 127 and a thickness of 10 mm (Fig. 3c).

Their position was defined so as not to interfere with the old piles. The old concrete crown was demolished step by step, caring not to cut the reinforcing bars. The demolition was accompanied by the contemporary reconstruction of reaches of the new, wider crown. Twenty horizontal threaded bars having a diameter of 32 mm, passing across the pile body and placed at two different levels (Figs 3b, d, e), strongly connect the two opposite sides of the crown. Figure 3f shows an intermediate phase of the works. Before and during the works, the attitude of the bridge was topographically surveyed.

5 MAIN STRUCTURES OF THE BRIDGES

The interventions needed on the main structures of a bridge may be summarized as follows:

– Strengthening of the carrying structure due to some lacks in their load bearing capacity, caused, for instance, by settlement effects, or by corrosion of the reinforcing bars or of the prestressing steel in critical sections;
– Refurbishment and adjustment of the structure to new codes prescriptions, involving, for instance, heavier load conditions;
– Changes in geometry, due to the need of widening the road platform.

In the following, the case of refurbishment regarding an old arch bridge is examined.

5.1 *The railway bridge across the Gaggione River*

5.1.1 *The structure*

The railway bridge across the Gaggione River was built in 1885 and connects the city of Milan to Varese, 60 km away.

The bridge, shown in Figure 4a, has a total length of 130 m and is made of a sequence of seven stone barrel arches, which rest on six stone piers. The barrel arches have internal radius of 5.82 m and their centers are 14.00 m apart. The piers are slightly tapered. The longest one is 32.60 m high. The free height of the piers is interrupted by an intermediate service deck made of five shallow arches.

In 1985–86, the bridge was strengthened in order to carry new, heavier train loads. The main interventions regarded the basements of the central piers, the node at the intersection between the piers and the intermediate deck, and the strengthening of the intrados and spandrels of the arches.

The intrados and spandrels were enveloped in a layer of shot concrete, 180 mm thick, and reinforced with 14 mm bars arranged so as to form a mesh with a pitch of 200 mm both ways. After a strong hydrosand blasting in order to improve the chemicalmechanical adherence, the added layers were linked to the stone surfaces through a uniform curtain of pins and confined against the original walls through 22 mm threaded bars, piercing the body of the arches. Other works regarded the river bed stabilization, obtained with the introduction of embankments and weirs.

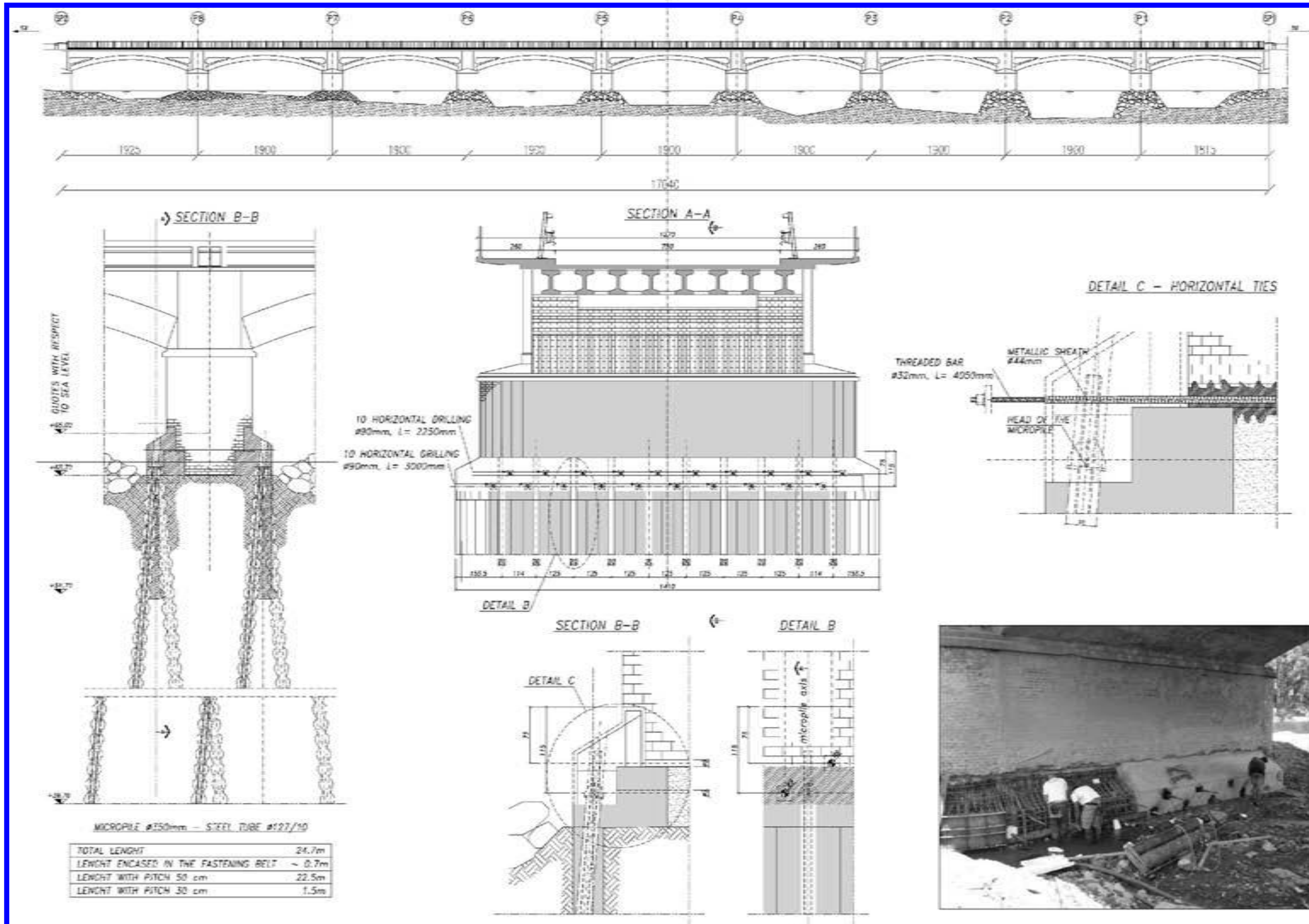

Figure 3. Lodi Bridge (1863–64). (a) front view of the bridge; (b) transversal section showing the new deck, made of precast prestressed beams, the position of the new piles and that of the threaded connecting bars; (c) arrangement of the micropiles at the two sides of a pier; (d), (e) details of the R.C. hooping crown; (g) photograph of an intermediate phase of the works.

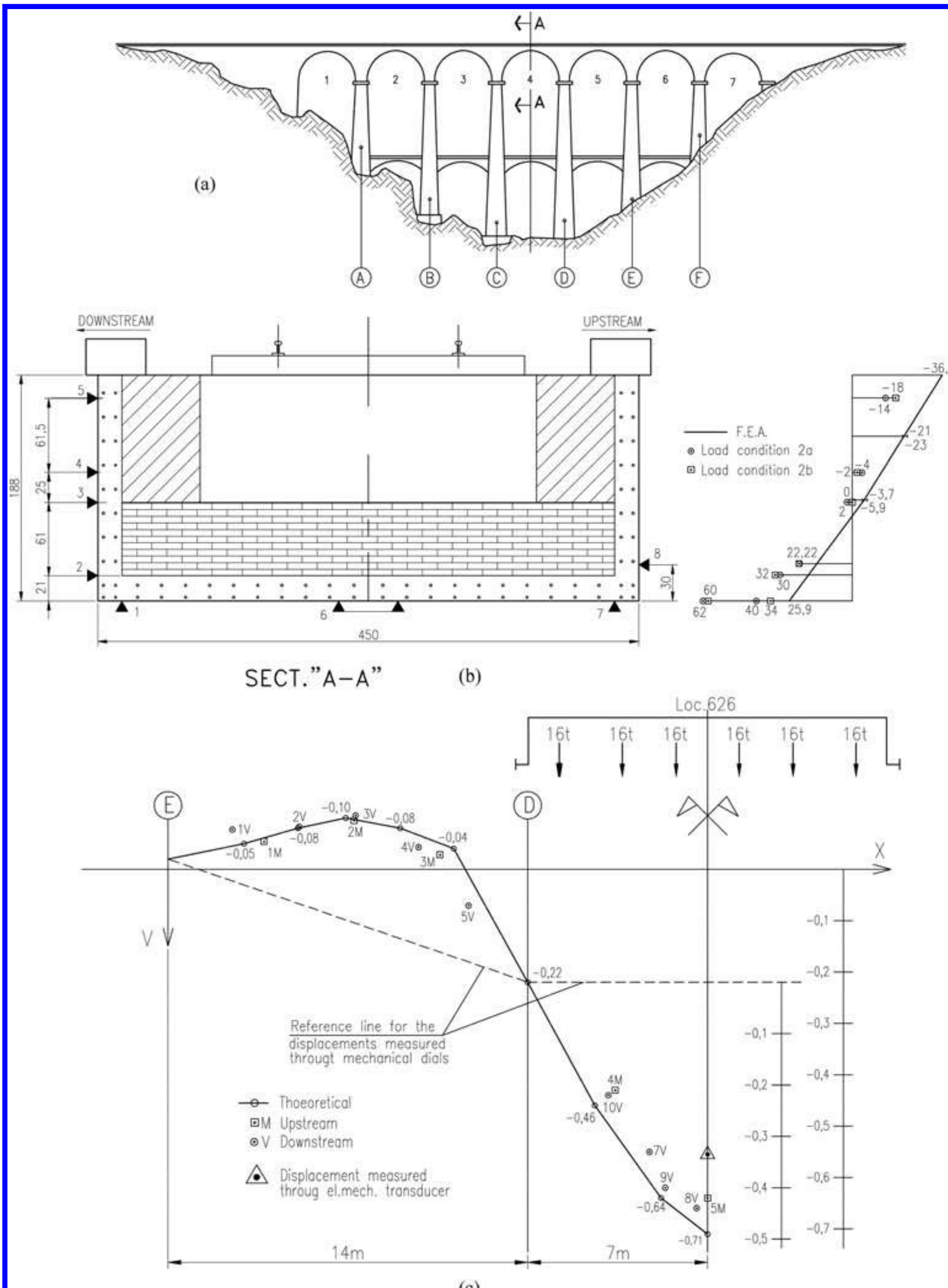

Figure 4. Gaggione Bridge (1885): (a) front view of the bridge; (b) transversal section after strengthening, layout of the relative displacement transducers and distribution of the strains in the depth of section during the loading tests; (c) comparison between theoretical and experimental vertical displacements.

5.1.2 *The problem*

It is quite obvious that, considering this type of massive bridges, any intervention which confines tightly the masonry, and, at the same time, does not add excessive loads to the original structure, leads to an increase in the safety level. It is less immediate to define the actual static scheme and how old and new structures contribute to the overall bridge robustness.

Usually the arch is considered the main bearing structure, while the superstructures (the walls and the filling up, made of light non cohesive material), are considered as dead loads. Then, why it is so common to see clefts around the arches keystones and even, in some cases, the loss of central bricks?

5.1.3 *The bridge behavior*

In December 1986, at the end of the strengthening works, severe loads test were carried out. Several load positions were examined and the vertical displacements were measured. At the intrados and at the lateral walls of the central section, eight electromechanical gauges (relative displacement transducers) were placed as shown in Figure 4b. Figure 4c shows the displacements obtained with the train in the central position. The maximum vertical displacement was $v = 0.71$ mm. Figure 4b shows the vertical distribution of the horizontal strains compared to the stratigraphy of the section: the added reinforcement layer and the arch rib are in tension; the spandrels above the arch top and the infill material are compressed. A fairly good matching between numerical and experimental data was reached.

Hence, during the service life and for the applied loads, the crown of the bearing arch works in tension and the superstructure in compression. Analogous behavior was found through a FEA for the effects of self weight. The theoretical final results of the analysis gave, for the most compressed fiber, $\sigma = -0.28\ \mathrm{N/mm^2}$, a relatively small value which can be sustained also by a moderately compacted soil.

These assessments, which trust to the cooperation with materials that cannot be defined as structural in the strict sense of the word (i.e. the filling of the spandrels), cannot be used for safety evaluations. Safety derives from the certainty that the evolution from the service to the ultimate state would involve the crushing of the filling material, while the line of the thrust lowers until it reaches the extrados of the bearing arch, which finally works according to the usual interpretation of its behavior.

From this experience, many suggestions were drawn for similar interventions carried out in the following years.

6 AUXILIARY AND SPECIAL DEVICES

The service life of a bridge is strongly influenced by the regular functioning of all its different components.

A special mention must be made to the water drainage system. It has a relatively modest cost if compared to other parts of the bridge, but it may cause severe damages. Leakages through any points of weakness in the waterproofing system or in the expansion joints may lead to significant reductions of the structural safety.

Other vulnerable systems are the expansion joints which may exhibit worn rubber covering, torn up anchorage bolts and permanent deformations. A cause of these drawbacks can be found in the dynamic effects due to road platform discontinuities in correspondence of the joints and to poor attention paid to the details during the construction works.

The bearing supports can also be considered as critical parts in a bridge: their substitution in old bridges is frequently not easy.

According to the actual concepts of maintenance, the stays are considered as special devices that can be substituted when necessary. But when the number of stays is very limited, it is difficult not to involve the entire structure.

In the following, the refurbishment of the ties of one of the three A shaped frames which characterize the Polcevera Bridge is presented.

6.1 *The strengthening of the ties of one of the first tied bridges. The case of the Polcevera Bridge*

6.1.1 *The structure*

The Polcevera Bridge was designed by Riccardo Morandi, built in the years 1960-1964 and put in service in September 1967. It flies over a large railway parking lot and connects the A7 Genoa-Serravalle highway to the A10 Genoa-Ventimiglia highway. A general layout of the bridge is shown in Figure 5. The bridge is 1121.4 m long and 18.00 m wide. Its main part is composed by three A shaped frames 90.2 m high, supporting decks 171.9, 171.9 and 145.7 long, connected by 36.0 m long suspended girders. At a distance of ten meters from their ends, the long decks are suspended to a couple of ties, made of prestressed concrete.

This scheme repeats concepts already adopted by Morandi for the Maracaibo Bridge, the Rio Magdalena Bridge and the Wadi Kuff Bridge (Morandi 1969).

6.1.2 *The concept of stay according to Morandi*

This type of ties is probably the most characteristic elements of the Morandi system. The tie sections are shown in Figure 5b. Their construction sequence was no doubt complicated, but had a clear aim: to create ties that behave as an homogenous system made of tendons working in tension and of a prestressed concrete case, working in decompression, but not in tension, under the added and traffic loads. In this way, the fatigue effects in the strands were limited thanks to the reduction of stress variations due to variable loads and, at the same, the strands were protected against corrosion.

6.1.3 *Needed repair interventions*

After about 25 years of service, many parts of the bridge presented severe damage states. On the ties of frame No. 11, at the Genoa side, clear corrosion traces in the strands of the tendons appeared. Minor damages were detected on the tendons at the top of the antenna of the nearby frame (No. 10) and in other parts of the bridge. In 1992–94, a recovery program was carried out under the guide of Francesco Pisani, who was one of Prof. Morandi's aides at the time of the bridge design and planned the repairing intervention phases (Martinez y Cabrera 1994). The main intervention concerned the four ties of frame No. 11. The basic concept of the intervention was to flank each original tie with a set of 12 additional modern cables, in order

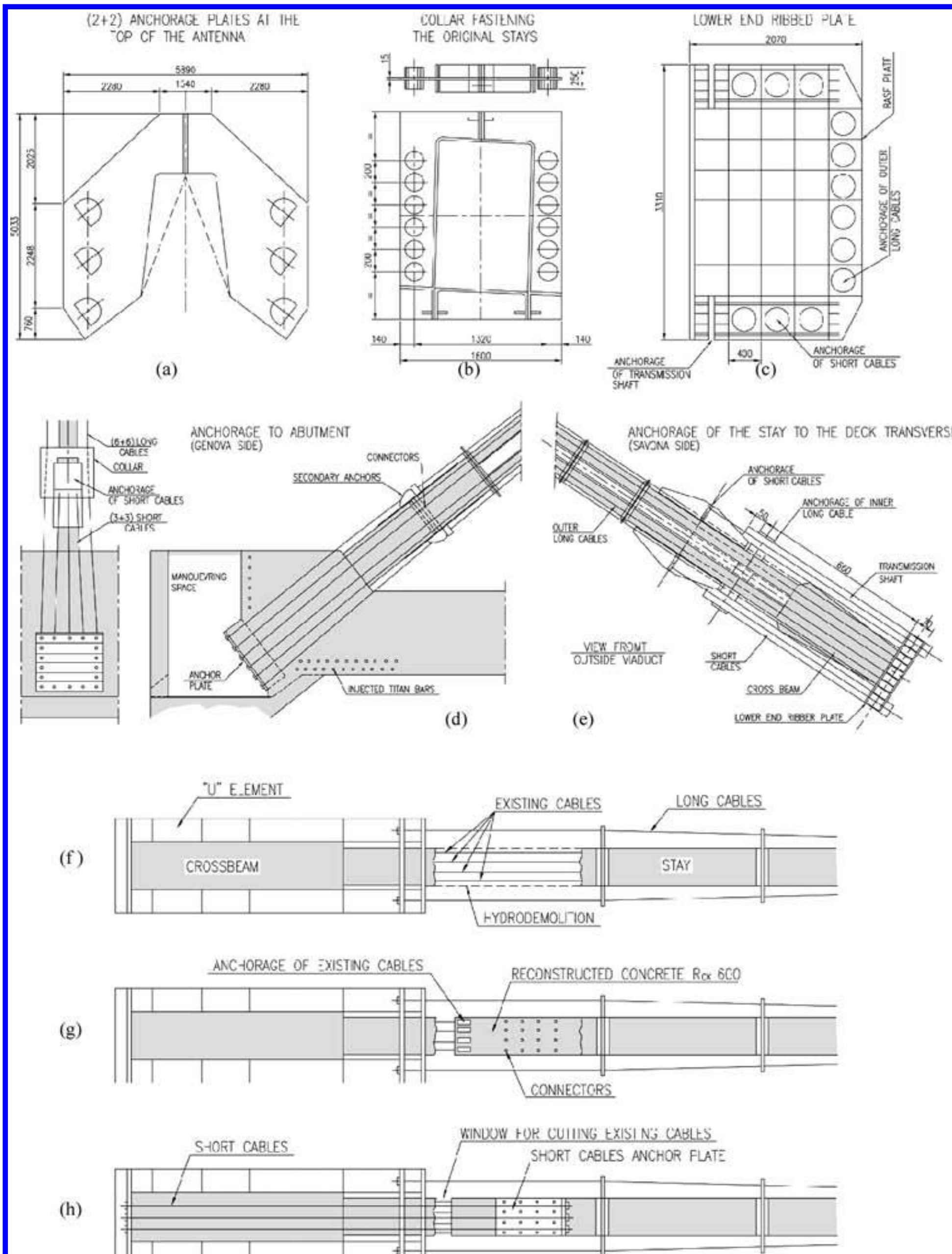

Figure 5. Polcevera Bridge (1960–64). (a) anchorage plates at the top of the antenna; (b) collar fastening the original stays; (c) lower end ribbed plate; (d) anchorage of the tie to the abutment; (e) anchorage of the stay to deck transverse; (f), g), (h) phases of demolition and of reconstruction of the end zones of the ties.

to transfer the suspension action from the ties to the stays.

Through specially designed devices (collars and new anchorage systems) and following the recovery sequence (progressive tensioning phases) shown in Figure 5c, the new "composed" stay system, resulting from the coupling of the old ties to the new cables, maintains its original shape, while the stiffness characteristics remain very close to the original ones. This is important in order to maintain the original design behavior of the bridge and to avoid any change in the deflection and flexural behavior of the deck, with consequences also on the elements of the main frame.

Another aim of the progressive tension transfer from the old ties to the new composed stays, was to reduce the structural risk of excessive compression stresses in the concrete ties, avoiding potential bursting effects when the old cables were being cut.

At the end of this process, the compressive stress in the concrete ties was about 10 N/mm^2, as the original design assumed.

The repair works concerned also the other two frames. On frame No. 10 a local repair, aimed to strengthen the upper end of the ties was carried out. On frame No. 9, whose cables appeared less damaged with respect to the previous ones, no particular interventions were adopted. Surface protection interventions were carried out on all three frames.

It must be pointed out that the interventions were carried out without traffic interruptions. Only some traffic limitations were needed during the demolition and tensioning phases.

6.1.4 *Some final considerations on the Polcevera Bridge*

The Polcevera Bridge and other Morandi tied bridges represent an exceptional reference from the conceptual, aesthetic and technical point of view, which is even more relevant if related to the times in which these structures were built. Nowadays, however, similar static schemes, though brilliant, cannot be proposed.

According to the modern criteria of durability, the "prestressed concrete tie" does not appear as a safe solution for elements in tension. Moreover, the suspension action entrusted to a limited number of elements, makes the whole structure little robust and the maintenance actions quite difficult.

Modern bridge configurations, characterized by a relative great number of stays (a "curtain of stays"), are designed so that, should the failure of one of more stays occur, the subsequent loss of suspension action would be made up for by other suspension elements, making the cables maintenance and/or substitution easier.

7 FINAL CONCLUSIONS

At the end of this presentation, the following conclusions can be drawn.

As regards to the relationship between the bridges and the environment, on the basis of what it was observed, the actual trend to avoid or to limit the number of piers in riverbed, and to prefer an increase in the span of the deck, is confirmed and recommended. Possible piers in the riverbed must have strong foundations, surrounded by suitable riverbed stabilization devices. The piers must be correctly placed with respect to the flow.

As far as existing bridges are concerned, it seems that the old massive and well shaped piers behave better than some types of piers built in 1960s and 1970s, which tend to rake solid debris and are more vulnerable. Moreover, a massive pier makes it easier to carry out integrative works aimed to helping the original foundations.

Speaking about the main structures, the usual remarks on the durability of reinforced and prestressed concrete structures can be recalled. A proper choice of the type of concrete, a correct curing process, adequate cover and detailing of the reinforcing bars may considerably lengthen the service life and reduce maintenance costs.

Particular attention must be paid to the drainage system. The lack of efficiency of the drainage system is one the main causes of damages and corrosion both in steel and concrete structures.

Any effort to eliminate joints or, at least, to reduce their dynamic effects, must be done.

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The evolving science of terrorist threat mitigation for bridges

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ABSTRACT

Since September 11, 2001 (9–11), a new threat has been added to the list of all others that endanger our nation's bridges: Terrorists! Prior to 9–11, bridge engineers would have considered it ridiculous to lump terrorism amongst the more common natural threats such as earthquakes, wind, and flood. Yet, national and international events since that time have proven otherwise.

This paper provides a glimpse of the current state-of-the-art for terrorist threat mitigation for bridges and how it has evolved from almost nothing since 9–11. The science is still very much in its infancy and has far to go before it reaches the level of maturity and definition desired by engineers endeavoring to address the threat in their bridge designs and retrofits. While it is very sad that this science even has to exist, it is exciting for engineers and scientists looking to make a difference in the world that there is still so much room for innovation and development. As shown in the paper, we are still looking for that silver bullet that will effectively and economically mitigate all terrorist threats.

Most importantly, the author hopes to inspire engineers and scientists to find and pursue their niche within this science so that they can make important and innovative contributions, and ultimately achieve what we all desire, to make this world a better- and safer place for all.

So, are bridges really threatened by terrorists? Immediately following 9–11, there were credible threats made against several major bridges within the United States, causing them to close for short periods of time and to deploy military personnel for their protection. Other countries, such as England, have experienced direct terrorist attacks against their bridges over the years (Williamson, in press). And as shown in Figures 1 and 2, terrorists fully recognize bridges as viable targets. Thus, the security of transportation infrastructure against terrorist attacks is now an important issue for engineers.

Figure 1. Bridge destroyed terrorists.

Figure 2. Bridge with piers destroyed by terrorists.

There are unlimited possibilities as to the types of terrorist threats that could be brought against bridge structures (AASHTO 2003). This paper discusses threats in the following basic categories: (1) *Vehicle-Borne Improvised Explosive Devices (VBIEDs)*: These include explosive-packed land-based vehicles that would be deployed against components reachable by land and water-based vehicles that would be deployed against any components reachable by water. (2) *Hand-Emplaced Improvised Explosive Devices (HEIEDs)*: These include improvised explosive devices that while not as large as VBIEDs, can be placed in direct- or near-contact with a structural member and cause severe localized cutting- and breaching type damage due to its close proximity. (3) *Non-Explosive Cutting Devices*: These include any non-explosive devices such as saws, grinders, and torches that can be used to cut/sever structural members. (4) *Intentional Vehicular Impact*: Like VBIEDs, these include both land-borne and water-borne vehicles, depending on the location of the component of concern. (5) *Fire*: Fire of size and duration can cause structural members to lose both their stiffness and strength. Thus, a "pool fire" from such as a ruptured tanker truck on the deck of a bridge, adjacent to key components or in the water adjacent to piers or towers is of concern.

The "science" for mitigation of these threats is discussed in terms of the following sub-areas: mitigation prioritization; vulnerability assessments; and threat mitigation. Detailed discussions of these items are provided in the paper, but a brief overview is provided below.

Mitigation Prioritization: Since 9–11, most bridge owners have completed the prioritization of their infrastructure between individual nodes (i.e. bridges, tunnels, etc.) and have begun or are ready to begin mitigation efforts on their highest priority nodes. At this point, the question once again arises: Where do we start? Based upon the myriad of terrorist threats that could be brought to bare as well as the large number of vulnerable structural components on any given bridge, there are almost unlimited mitigation measures that can be deployed on a given bridge. Yet, there are always limited resources. A rational and consistent means is required to assess and compare individual structural component criticality and the effectiveness of varied mitigation measures throughout an individual bridge. Thus, once again a prioritization is required, this time at the individual structure level.

Instead of prioritizing among a group of structures, the owner must now prioritize among the individual components on a given structure to determine which are at highest risk and most in need of mitigation efforts. The need in this case is to compare individual bridge components based on their specific importance and vulnerabilities. Since most of the high-priority bridges are major structures with potentially massive replacement costs and economic effects if lost, importance should be primarily based on a component's contribution to overall structural stability; i.e. if the component is sufficiently damaged, the bridge will totally collapse. However, other factors such as component replacement or repair costs can also factor in. Component vulnerability will be a function of the specific threat type and size used against the component, the likelihood of such a threat, and the component's resistance to the threat. There are many risk assessment methodologies that can be used for this purpose. This paper provides an overview of a "component level risk assessment" methodology developed by Ray (2007) specifically for this purpose.

Vulnerability Assessment: In order to prioritize mitigation measures on a given structure, the vulnerabilities of important structural components to each threat must be understood. Since damage from terrorist threats is generally very localized (Fig. 3), vulnerability assessments must be accomplished at the individual component level.

Total bridge collapse will only occur if the locally affected structural components (i.e. column, truss member, tower wall, cable, etc.) are sufficiently damaged and structurally important enough to induce a progressive collapse of the entire structural system.

Figure 3. Explosively damaged girders.

The paper provides an overview of the vulnerability assessment process for terrorist threats and discusses various analytical tools that are available for this purpose.

Threat Mitigation: Physical security of any asset must essentially comprise a layered and fully integrated combination of four basic mitigation measures, referred to herein as the "Four D's": Deter, Detect, Delay, and Defend. These measures cannot be applied independently and must be employed as part of an interdependent systematic approach to a layered security perimeter around the protected asset, in this case the bridge component or a specific critical bridge component. The paper provides discussion on each of the Four D's, but emphasis is placed on the Defense aspect (better known as "hardening"), which addresses the scenario where the attacker overcomes the denial methods and carries out the attack before a capable response occurs. Or, with the case of a vehicular-borne device (i.e. vehicle bomb), the attack can be carried out so quickly and with such force that detection and denial methods are essentially of no use. Hardening is the only viable defense for this threat.

The type of structural defense employed will of course be threat dependent. Hand-emplaced explosive threats, non-explosive cutting threats, fire, etc. all require radically different defensive measures. Significant advances have been made since 9–11 for protection of vulnerable components from hand-emplaced explosive- and non-explosive cutting threats. However, most of these technologies are proprietary and/or Classified and thus cannot be discussed in the paper. And while there is always room for improvement, fire and vehicular impact mitigation technologies are well-evolved and require no additional discussion. Thus, the major focus of the paper is on hardening against vehicle bomb blast, which is a predominant threat against which hardening is required.

In sifting through the myriad of potential solutions to structural hardening, the most important thing to remember is that no matter how exotic or high-tech a proposed mitigation scheme may appear, it must ultimately affect at least one of the three variables of Newton's Second Law as given in equation (1).

$$\Sigma F = m \cdot a, \tag{1}$$

where ΣF = the summation of applied force and resisting force; m = mass; and a = acceleration.

The response of any structural component to a blast loading can depend upon many factors, but as shown in equation (1) it will depend primarily upon: structural mass; strain capacity and strength of the component; proximity to the detonation (i.e. standoff); magnitude of the detonation (i.e. explosive type and weight); and support conditions for the responding component. For points of discussion herein, equation (1) is expanded to:

$$F_{applied} = F_{resisting} + (m \cdot a), \tag{2}$$

where $F_{applied}$ = the applied forces (such as blast loading) and $F_{resisting}$ = any resisting forces, such as that due to bending, shear and support reactions of the responding component.

This basic law certainly does not require review, but it is used to make several basic but important points about the role of mass and resistance in the response of a bridge component to blast. First: The greater the mass, the higher its inertial resistance to acceleration; and likewise the greater its strength, the more it will be able to resist the applied explosive force. Conversely, once the mass is moving, the component must have sufficient strain capacity (i.e. resistance) to overcome the momentum of the moving mass and arrest its motion. If not, the component will fail and if sufficient momentum remains, it will ultimately fly away as a "fragment".

In addition to these factors, magnitude and proximity of the explosion affect the applied loading and impulse imparted to a bridge component. Near-contact detonations, such as in Figures 4 and 5, produce extremely severe pressure and impulse loadings. Because the detonation is so close-in and the pressure durations are generally so short in relation to the fundamental response mode of the component, the response is more a function of the total blast impulse and not the peak pressure.

Blast effects can be mitigated by any combination of the following basic categories, all of which affect either the load- or the response side of equation (2):

Increase Bridge Component Resistance to Load: Strengthening of a component against blast is accomplished in conventional ways, such as component thickening, span shortening, etc. It is almost always accompanied by an increase in the component's mass, and as seen in Equation (2), this is always a good thing. Mass is an important part on the resistance side of the equation and it can significantly decrease structural response and damage.

Steel has the significant benefit of high mass and many newer steels also offer high-strength and ductility. While not as high in mass, concrete can offer very high strength and ductility if detailed properly.

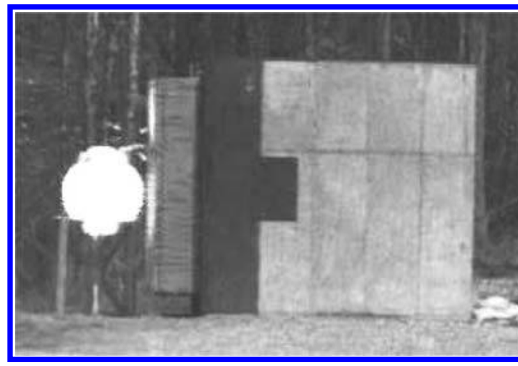

Figure 4. First few microseconds of a detonation near the face of a target.

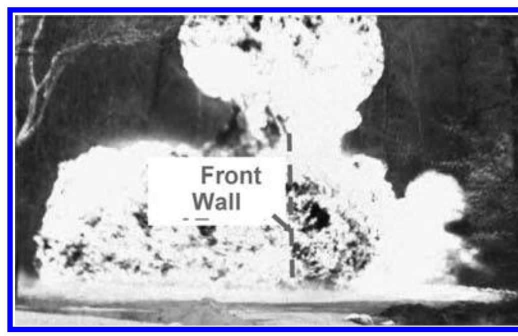

Figure 5. Approximately one millisecond into same detonation as in previous figure.

In addition, many other high-strength, ductile, and lightweight advanced materials have evolved in recent years and gained wide acceptance within the design/construction communities. However, the reader is reminded that while these materials can have application to blast mitigation, the light weight (i.e. low mass) can be detrimental in terms of inertial resistance. Additionally, many of these high-strength materials have a relatively low strain capacity.

Decrease Applied Load: If sufficient resistance cannot be obtained for the component of concern, is it possible to affect the loading side of equation (2) and reduce the blast energy that is applied to the component? Blast loadings decrease very rapidly with distance from the point of detonation (US Army, 2002). Thus, the most effective means to reduce blast loadings is to enforce standoff. This can be accomplished via conventional means such as impact-resistant traffic barriers, etc. However, as previously discussed, this is rarely an option on bridges as this generally requires narrowing- or closure of traffic lanes, and our nation's bridges are so heavily taxed with traffic that this is generally not an option.

Thus, the only remaining option in this category is to place a mitigation measure between the bomb and the target that has the effect of reducing the amount of explosive energy that actually makes it to the protected structural component. Numerous "energy absorbing" concepts have been proposed and explored for this purpose as it is clear that all materials demonstrate various phase changes (an energy absorbing process) as a function of shock pressure. In addition, all materials absorb energy to varying degrees as they undergo gross irrecoverable volumetric strains (i.e. crushing).

Beyond energy absorption, are there ways to completely- or partially shield the component of concern from the explosive energy? This can conceivably be accomplished through blast barriers that can be broken into two basic categories: structural barriers and sacrificial barriers.

A structural barrier is essentially a "wall" in front of the protected component that has sufficient strength to stay in place throughout the blast event, collecting and dissipating all of the explosive energy and completely shielding the protected structure. A sacrificial barrier provides shock wave reflection and inertial resistance just like structural barriers, but has minimal structural resistance and breaks apart under the blast loading, thus minimizing support reaction forces. The experimental testing of various materials for blast energy absorption and shielding is discussed in detail in the paper. Materials considered included: elastomers, dilatants, porous aggregates, concretes, and water.

It is ultimately shown that energy absorption or shielding is not effective for bridge components where bomb standoffs and shield thicknesses are minimal. Certainly the theory is sound, but unfortunately there is just not enough space to place a sufficiently thick shield with enough material to significantly affect the extremely high blast pressures from a near-contact detonation.

Load Path Redundancy: Complete mitigation of damage from very large explosive threats may not be economically or logistically possible. In many cases, it makes more sense to just limit the extent of damage to the most exposed components and ensure that there is enough redundant/residual capacity in other less exposed members to insure that the structural system as a whole can continue to function and not undergo a progressive collapse.

Layered Hardening Approach: A wide variety of hardening concepts are discussed and each has at least some validity and usefulness for specific scenarios. There are also no concepts that just alleviate the extreme loadings without consequences. The explosive energy does not go away and must be defused in some manner. Essentially, any concept only serves to re-distribute the energy; either through inertial resistance, strain energy, or momentum transfer (fragmentation). Mitigation designers are encouraged to consider all of the strengths and weaknesses of each of the concepts and develop a layered hardening approach that capitalizes upon the strengths of each. An example of a layered hardening approach for a cellular steel column is provided.

Multi-Hazard Considerations: Terrorist threat mitigation cannot be considered alone. In addition to terrorism, there are many other hazards that threaten a bridge, including earthquakes, wind, water, fire, weathering, etc. and a risk-based approach must be utilized to determine the relative degree of importance of each threat to a given bridge. And, as funding is always limited, the mitigation efforts must be prioritized according to the level of risk. The paper discusses mitigation measures that can address multiple hazards, such as wrapping of reinforced concrete columns to increase both seismic and blast resistance. In addition to the beneficial multi-hazard overlaps, discussion is also given to detrimental overlaps of mitigation measures. One example is: Local hardening of a structural component to increase its blast resistance may add detrimental mass and stiffness within the structural system, affecting its seismic resistance.

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Preserving the asset for its intended use

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1 INRODUCTION

The problems of maintenance are just as challenging as those of design and construction. This paper draws heavily on concepts written by J.A. Robertson, FICE, Superintending Civil Engineer, Ministry of Public Building and Works, UK circa 1969 and was my guiding philosophy throughout my career at CN. Robertson (1969).

I hope what I have to say will cause you to think hard about the difficult choices that have to be made in funding adequate and appropriate structural maintenance.

What you begin with is critical to long term maintenance requirements. If new structures are adequately designed to shed water and deal with the elements and are sufficient for the load to be carried through the structural life, then maintenance will be less onerous.

A good quality protective coating applied in the shop for structural steel, good waterproofing for concrete and careful treatment of wood products along with careful detailing will pay dividends for many years. Skimping here seems foolish.

We need:

- Good Design
- Good Execution
- Good Inspection
- Good Maintenance

2 MAINTENANCE

Maintenance is: Work undertaken in order to keep or restore every part of a facility to an acceptable standard. Robertson (1969).

An acceptable standard is not necessarily perfection but what is adequate and appropriate to do the job. The inability to distinguish between what is adequate and perfection is often a problem for engineers and a major complaint of professional managers. Remember as Voltaire said:" Perfect is the Enemy of Good". Just make sure the Standard of maintenance is good enough.

Note that often work charged to maintenance is not always really maintenance.

- Some is ineffective
- Some is done inadequately
- Some is unnecessary.

Table 1. Cost Effectiveness of various situations, Robertson (1969).

	Work Required		Work not Required
Work Done adequately	Fair ware and tear, etc. Cost effective	Culpable Neglect, etc. Cost effective Only when Restitution is obtained	Simple waste
Work Done inadequately	Wasteful Rectification costs and possibly indirect costs involved	Wasteful Rectification costs and possibly indirect costs. Restitution of costs may be Prejudiced.	Simple Waste plus Wasteful Rectification and/or indirect Costs may be incurred
Work not done	Indirect costs	Indirect costs. Possibly loss of Opportunity to seek restitution	

These need to be minimized as much as possible through effective maintenance management.

Table 1 shows the main options.

We have all seen diagrams like Figure 1. Note in particular that there is a category of ineffective costs charged to maintenance. These are in effect a total waste.

2.1 *Inspection Sweeney (2010)*

Given the fact that unlimited funds are rarely available, there is a need for high quality inspection and sound engineering assessment. Inspection is the poor man's insurance and the best line of defense to ensure safety and reliability.

With regard to inspections, one size does not fit all. Inspection procedures for a bridge plant consisting of proven robust designs needs to be different than one where there are weaker structures. Clearly, one of a kind structures need special inspections designed for the individual structure, and usage should play a role. Structures that are not used often might justify less frequent inspections.

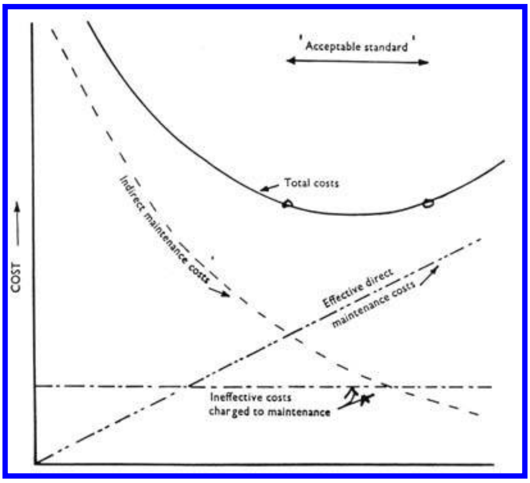

Figure 1. Cost effectiveness of Maintenance.

Roughly 28% of North American rail bridges on the 7 principle railways are timber. The percentage is much higher on the independent short lines. What is not acceptable on a principle main line may be quite tolerable on an infrequently used line.

2.2 *Capital vs. labor*

This brings up the debate as to whether to repair or replace. The railway industry is a capital intensive and labor intensive business. Most businesses are either one or the other. The decision to replace may depend on having an asset that reduces the need for full time staff to perform routine maintenance or if capital is scarce it may be the reverse. Tax and other considerations come into play here.

2.3 *Being complacent*

Watch out for knee jerk reactions. Not all concrete failures are due to lousy concrete nor are all steel failures due to fatigue. Abraham Maslow, a noted American Psychologist quipped: "If the only tool you have is a hammer, you tend to see every problem as a nail."

Also watch out the next major failure type may have already happened but as a profession we may not yet be aware of it. We need to document and be aware of the cause of all failures.

This is the main reason why Engineers in this field must not lose their engineering skills and become preoccupied with management and funding although these are essential concerns.

2.4 *Up-Dating budget allocations*

Reducing maintenance to doing nothing or what we have always done before is not necessarily adequate or responsible. Periodic re-evaluation is necessary.

Budget allocations when structures were new are clearly not adequate when structures are approaching

Table 2. Compare 1964 to today. Sweeney (2006).

1964	Today
50 kg/m (100 lb/yd)	70 kg/m (142 lb/yd)
Wear and corrosion	Fatigue
Design E 60	E 100 (E 80)
Average Steel bridge 58 years old	Average age 95
	Car loads up 180%
	Tonnage up 600%

100 years of service. Observe the changes during my career and note that many of the bridges in service today are over 100 years old and were designed for E 40 load.

2.5 *Funding maintenance*

The first place to look for needed funds is to areas that are wasteful and neither needed nor effective.

Engineers especially those who have to campaign for adequate budgets need to understand engineering economics and how it differs from investment economics.

2.5.1 *Rate of return calculations*

Subjecting maintenance funding to rate of return calculations is not appropriate unless it is to determine if a different standard of service or abandonment of service is appropriate. Rate of return calculations always assume that the do nothing options is acceptable. The do nothing option is never acceptable if work is needed to keep or restore every part of a facility to an acceptable standard.

To emphasize again, rate of return calculations are appropriate in deciding whether an asset should be abandoned or put to other use.

As an example many years ago the state of repair and revenue from traffic on the narrow gage railway on the Island of Newfoundland put its continued existence in doubt. Maintenance was a continuing draw on funds with no return and little or no long term prospects. Simply put the cost of minimum annual maintenance exceeded revenues by a large margin.

Studies to find ways of increasing revenues showed no long term prospects. A relatively simple rate of return calculation that was continuously negative convinced the Government owner to abandon the line and place their investment funds into the nearby highway infrastructure as a more useful investment. Instead of pouring money down the drain this has proved to be a more useful spur to economic activity in that area.

2.5.2 *Present value calculations*

Present value calculations are useful in deciding between two viable options, for example do I paint now or do steel repairs later.

In performing such a calculation the interest rate chosen has to be consistent with the realistic remaining useful life of the structure, should be in constant dollars

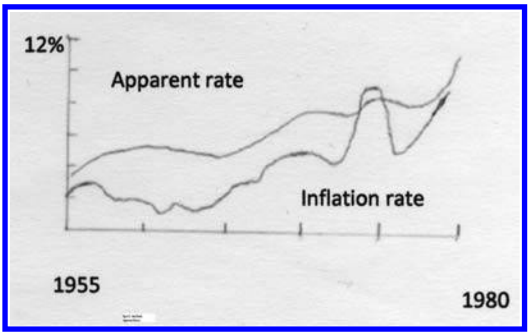

Figure 2. AAA Bonds, Jones (1982).

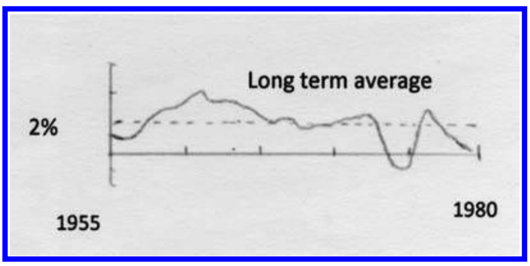

Figure 3. Real rate of AAA bonds, Jones (1982).

and must not include inflation Jones (1982). This is not an investment decision where "not investing" is an option. Using investment interest rates are just wrong in spite of what your investment advisor or venture capitalist advises.

Venture Capitalists tend to want their money back quickly and this is appropriate when deciding whether to invest or not. But once an asset is in place and needed, if its useful life before major upgrading is say 70 years then choosing an interest rate that completely discounts all costs after 20 or 30 years is wrong.

Although inflation has not been a major concern for the past few years, I suspect it may be in the future.

To illustrate the concept of the effect of inflation consider the annual yield rate (%) of AAA corporate bonds (Figure 2) and compare it to the real rate without inflation (Figure 3).

It is seen that the average yield of these bonds over a 25 year period was 2%. "The traditional time value of money has been more of an accounting of inflation and risk rather than a reflection of the true earning power of capital stock in recent years".

2.5.3 *Inappropriate policies*

Various Tax regimes also tend to use distorted interest rates and depreciation rates that are useful as public policy but are total irrelevant to the structural maintenance decision process.

Any policy that does not preserve the asset to an acceptable standard throughout its full useful life is not a good policy.

There was one Railway Chief Engineer who ordered that all maintenance funding decisions be made on the basis of 10 years of remaining life. This for a large bridge plant that is expected to be needed for quite some time to come was totally irresponsible even if he intended to retire in 9 years. Fortunately, his staff politely ignored his order.

Suppose it had been possible to run the structures into the ground where and how would it be possible to earn enough money to completely re-build? As all managers know the first duty is to maintain shareholder value and preserve the enterprise. Substitute the word "the people's" for shareholder if the enterprise is Government owned.

2.6 *Cutting corners*

During times of economic hardship delaying maintenance may be acceptable provided there is an understanding that making up for these delays will be more costly in the future. Painting of structural steel is a good example.

On one railway the decision was made in the 1930s to stop painting steel bridges. The penalty came in the 1960s and 1970s when major steel repairs and replacements became necessary as a result of serious corrosion damage. This increased expenditure became a permanent fixture of maintenance budgets and will remain so for many years to come as corrosion resistant steel was not introduced until the 1970s and a no painting policy remained in effect. On the average we are looking at another 30 to 50 years of steadily increasing expenditures on corrosion damage because of the decisions made in the past.

Another railway maintained a painting policy and their corrosion repair bills remain far lower in comparison.

A present value comparison clearly indicates that the continuous maintenance painting option is the less costly option with two exceptions.

The first concerns those structures that were abandoned long before their structural life was exhausted. In North America many lines were abandoned as Companies consolidated and traffic in some areas declined to levels where service could no longer be economically provided.

The second is that period of time when painting became more costly than structural steel renewal as paint manufacturers struggled to develop environmentally effective acceptable systems that could be applied economically. This struggle is still ongoing particularly for field applied re-coating.

2.7 *Dealing with upper management*

In dealing with upper management or the owners it is important to understand that the man of knowledge is expected to take responsibility for being understood and not the other way around. The maintenance manager must take responsibility for his contribution and ensure that his "product" – that is, his knowledge is effectively understood and used. Drucker (1985)

"Thinking out loud" to senior executives not only wastes their time but can be very counter-productive.

These people need to be given clear actionable effective choices that will accomplish the desired results. They need to be respectfully led to do the right thing.

3 MAINTENANCE MANAGEMENT

Maintenance management can be divided into 5 main possible action levels:

1. A minimum level to comply with legal, statuary and contractual requirements.
2. A slightly better level designed to preserve the asset.
3. A normal level of economical good practice.
4. A higher level (for facilities that are operationally vital, structurally hazardous,
5. An even higher level for assets of prestige value, etc.)

Note that in level 1 it may not be possible to restore the facility to adequate operational condition without considerable expenditure since only the minimum legal and similar requirements are met.

At level 2 it would seem that reintroduction to service in a week would be a reasonable expectation.

At level 3, the asset should be available for service 85% of the time at a minimum.

At levels 4 and 5, the asset should be available 98% of the time under specified overload and abnormal conditions. The difference with level 5 is that the facility should also be in impeccable condition.

In deciding what to do and how much to do the answers to the following questions need to be quantitively assessed as each comes with a rising level of cost:

- Usable with work
- Usable with inspection
- Generally reliable
- No service interruptions

Table 3 summarizes the options.

4 BRIDGE MANAGEMENT SYSTEMS

These are useful and essential tools to determine where resources need to be focused but unless one has an army of persons to input data their usefulness on a particular structure is often overrated. It is very important to understand the difference between decisions that affect a population of bridges and those that affect a particular unique structure. Management Systems at the individual project level are and need to be far more comprehensive in detail. The expense of some management systems would be better allocated to fund maintenance.

If you think you can effectively manage a bridge plant from an air conditioned office then think again. We all should know the science of large numbers. In my country the chances of winning the Lottery are about one in 14 million yet someone wins almost every week. The chances of having a rogue bridge are about the same. Think of the I-35 disaster.

Table 3. Maintenance Levels.

Level	Functional Performance Quality and reliability	Safety of Persons and Property in all aspects	Preservation of the asset and the amenities
Minimum	Nil (Not Applicable)	All legal Requirements To be met	All legal Requirements To be met
Preservation	Nil (Not Applicable)	Fully safe in all respects for reintroduction to service on 7 days notice	Prevention of deterioration of asset and amenities
Normal	Availability Factor at full specified performance not less than 0.85	Fully safe in all respects for regular use	Meets specified condition of facilities and amenities
Operationally Vital	Availability Factor at full specified performance not less than 0.98	Fully safe in all respects for regular use under specified overload and abnormal conditions	Meets specified condition of facilities and amenities
Prestige	Availability Factor at full specified performance not less than 0.98	As above	To be in first class order and impeccable condition

5 SAFETY

Safety comes in two main aspects, the ability of the structure to fulfill its function safely and safety of personnel using, inspecting and maintaining it.

5.1 *Structural safety*

Structural safety means being strong enough and with geometric properties that allow its use in a safe manner.

Make the best of existing conditions. Do not condemn a structure because it is not what it ought to be but ask what it is good for under probable and not under practically impossible conditions of traffic, Waddell (1916).

A Phoenix truss that at first glance looked to be seriously overloaded upon close examination proved to be adequate given the weekly inspection that it received and the very low volume of traffic. The end circular end posts had been filled with concrete greatly increasing their capacity and the bottom chord was pre-stressed. More interesting was the fact that the wrought iron web members of this through truss were strengthened with welded plates. The welds broke regularly but a welder examined all of these welds before each

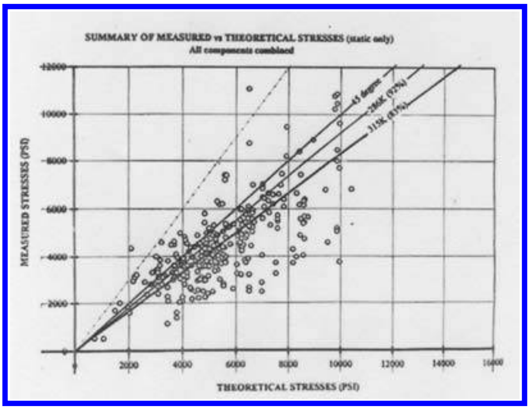

Figure 4. Stress range tests results for 100 steel bridges.

train passage and any failed welds were repaired. The structure proved adequate for its use.

5.1.1 *Real stresses*

Actual stresses or stress ranges often differ from what was assumed in design. Some times for the better and sometimes not.

Load testing and strain gage work often indicates that live load stresses are less than expected but as the figure 4 shows this is not always the case.

5.2 *Worker safety*

There needs to be provision for inspectors to safely do their task and for workers to accomplish maintenance.

One thing that is very important to remember is that ladders, walkways, hooks and railings need to be inspected. Often these are made of much thinner materials than the structures to which they are attached and thus will deteriorate much sooner than the main members unless special provisions are made to ensure adequate maintenance.

Simply putting hand rails for inspectors to tie off to on small simple structures can be adequate.

Major structures need more elaborate devices for inspection.

5.2.1 *Safe work methods*

One of the most frequent causes of serious failures is in the area of instability. Instability comes in various forms of buckling in compression and brittle fracture in tensile situations. During repair and maintenance work it is essential to ensure against these two.

During the retrofit of a large timber trestle, two 6 × 6 inch by 12 foot struts (152 × 152 mm by 3.66 m) that had been removed were inadvertently not replaced before a work train attempted to cross the bridge. The bridge collapsed like a deck of cards killing several people.

Laying out the sequence of a job so workers are working on a safe surface seems so obvious but sadly I have seen a few tragedies that resulted from thoughtless job sequencing.

6 REDUCING THE STRESS STATE OF THE RAILROAD

This has been a slogan for the Research Group of the Association of American Railroads for quite some time. The objective is to reduce the stress state so as to prolong the useful life of all assets including bridge assets.

The movement of a steel wheel on steel rails is ideally supposed to be a very low impact smooth operation so efforts need to be made to make it so or as nearly as possible so. Correcting potential problems early can save bigger problems later

- Removing joints with continuous welded rail reduces impulsive loads.
- Keeping approaches smooth and well tamped and supported reduces the bump and bounce at the end of the bridge.
- Good Car behaviour is important.
- Wheel and rail condition. The smoother these are the less the impulsive loads.
- Car loading/overloading.
- Watch for the effects of corrosion and fatigue on steel spans.
- Although many North American steel bridges were sufficiently over-built that poor bearings can be effectively tolerated this is not always the case.
- Apply Ultrasonic Impact Treatment to details that are sure to cause problems.
- On timber bridges keep the geometry tight and well aligned. If you don't they shake apart. This can also apply to steel structures.
- On timber bridges mechanical wear and environmental decay should be dealt with sooner rather than later.
- My personal view is that a bit of extra ballast on a concrete deck as long as the structure can handle the extra weight is a good idea. It certainly attenuates impulsive loads.
- Watch out for hidden problems in concrete and pre-stressed concrete spans. Reinforcement deterioration can be a critical issue.

7 RESEARCH

The role of test facilities in giving guidance to what and when things need to be done is very important. Most of us are aware of the long term impact of the ASHTO Road tests in the late 1950s on pavement design and maintenance but it influenced bridge work as well.

The future US Federal Highway 20 year Long-Term Bridge Performance Program (LTBPP) will develop a better understanding of how current highway bridges perform under a variety of service conditions. This program will monitor bridges in a number of States and will be of long term benefit to bridge maintenance practice.

Recently in the railway area, research test tracks in Pueblo, Colorado, and the Czech Republic are providing guidance on what is critical for long term maintenance.

At the US Department of Transportation's FAST facility in Colorado that is managed by the Transportation Technology Center Inc. (TTCI) of the Association of American Railroads there are three pre-stressed concrete bridge spans and a steel bridge with two spans. A train of 315 000 lb. (1.4 MN) cars circles a track loop at 40 mph (64 kph), reversing directions daily, and can accumulate the same traffic in a month that a very heavily travelled mainline railroad would see in a year, Sweeney (2007).

7.1 *Open deck fastenings*

For many years open timber decks have been fastened with a number of devices. All were somewhat of a maintenance headache. The FAST facility was used to test a number of devices and within a few years of testing a simple innovative solution was found that greatly increased the time between needed maintenance.

7.2 *Movable rail joints*

These joints are a constant maintenance headache, add impulsive loads into a bridge and are the bane of every track maintainer. Testing at the FAST facility has developed a rail joint for movable bridges that lasts more than twice as long as previous versions.

7.3 *Continuous welded rail on bridges*

Recent work by the TTCI Group of the AAR has clarified the behavior of CWR on bridges for mild winter to hot summer conditions. This has gone a long way to clarify the seemingly contradictory practices of various railroads. This testing better defined those instances where it is possible to eliminate rail expansion joints.

Further work is required at very cold temperatures.

7.4 *Steel bridge tests*

The Steel Bridge at the Fast facility has a serious flange crack that has been in service carrying 315,000 lb. (1.4 MN) cars for over 1400 MGT (Million Gross Tons (US)) or over 8 million cycles of loading. Careful monitoring of this crack with and without rail joints on the span has given information on the threshold stress range for which there is no crack growth and the stress range for which growth occurs.

Given that there are 47 other cracks in this welded bridge, various repair and retrofit schemes have been evaluated as well.

A number of non-destructive testing tools are being evaluated as well.

7.5 *Pre-stressed concrete bridge Tests at FAST*

The pre-stressed concrete bridges have accumulated over 800 MGT so far with few problems. Major studies on ballast depth, ballast mats and different ties (sleepers) as well as approach conditions have been monitored.

Figure 5. Crack into bottom flange.

7.6 *Hybrid-composite bridge Test at FAST*

A hybrid-composite beam span was installed and has accumulated over 120 MGT with no adverse effects noted yet.

7.7 *AAR timber testing*

Research work was performed at an AAR Affiliated lab (Texas A & M University) to evaluate the effect of interior timber rot in circular piles with varying Length to least Dimension (L/D) ratios, Sculley (2004).

Additional work was carried on to determine the shear behavior of timber stringers under large cyclic loading. The result was to increase allowable stresses from 75 to 150 psi, Fry (2004).

7.8 *Preparing for increased loading*

These and other tests are on-going and were initiated to ensure that there would be no serious economic surprises as North American Railroads increased loads by 20%.

The result has been good information on what to retrofit or strengthen and also some good indications of deterioration rates on many components.

7.9 *CN timber tie (sleeeper) tests*

In evaluating timber bridge ties for possible movement to higher axle loads, then current accepted practice indicated that most of that Railway's bridge ties used on open decks would be seriously overstressed.

Given the tremendous cost of replacing these ties, it was decided to embark on a testing program to establish the capacity of a large representative sample of the bridge ties it was purchasing as well as those that had been in service for quite some time.

The result of the study, Madsen (1999), indicated the CN Ties to be much stronger than analytically predicted.

8 FUTURE TO DO LIST

Issues that need further research to increase maintenance effectiveness.

- We need a methodology to use anecdotal data. There are thousands of structures in service that can tell us a lot.
- We need a methodology to use the research test data that did not lead to failure. A lot of tests get stopped before failure occurs, yet we have no agreed way of making use of this information.
- We need a much better understanding of the real load environment.
- The gap between design and prudent limits of use needs further investigation.
- The effect of infrequent loads on fatigue:-does one car in 1000 or 10,000 really effect the life?
- What light vehicles can be ignored in fatigue life evaluations?
- How do we avoid legal or political concerns from preventing the dissemination of valuable research?
- Capacity problems will force higher loads and more frequent loads on our bridges. What can we do to affordably accommodate these?
- Deflection criteria. As capacity problems get worse, the pressure for higher speeds on railways will grow. Current design deflection limits on steel and timber in North America are quite liberal and in fact as speeds increase the deflection of the span will absorb most of the tolerance now allowed for rail., FRA (2005)
- Educators and engineering supervisors need to remember to teach is so far as possible that Engineering is an Art that uses Science and not just applied science.
- Engineers need to develop simple intuitive tools to ensure their conclusions are in the right ballpark. Particularly in this computer age.
- It is very important that older engineers tell their stories so that mistakes are not continually repeated.
- Continuous relevant learning and improvement are a must.

9 CONCLUSIONS

Preserving the asset for its intended use involves work undertaken in order to keep or restore every part of a facility to an acceptable standard. This paper expanded on that concept and how to achieve it.

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On longevity and monitoring technologies of bridges – a survey study by Japanese Society of Steel Construction

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ABSTRACT

JSSC, Japanese Society of Steel Construction, has organized a special task committee and conducted a study on the recent development of maintenance, renewal and retrofitting for buildings and bridges of Japan. This paper describes the outline of the study by a working group of bridges and consists of two parts. The first part is concerned with the survey on the technologies for the longevity of bridges whereas the second part is specifically concerned with the structural health monitoring technologies reflecting recent developments. In view of the fact that important terminologies such as lifetime and service life have been used differently throughout the world, some fundamental terminologies are reviewed, discussed and redefined. Furthermore, recent developments on the structural health monitoring technology are reviewed and the results are summarized in a matrix form. Lastly, voices of some local governments, owners of expressways and railroads are taken into account in the form of questionnaires and replies to propose future structural health monitoring system for traffic infrastructures.

1 INTRODUCTION

In recent years, with some exceptions the number of children is significantly decreasing and young people tend to seek easy jobs which do not necessarily require them working hard with perspirations in the developed countries in particular and in some developing countries. In this view point, the education of engineers and important traditional technologies that have been acquired through generations are going to be forlorn and these facts are in fact becoming deplorable social problems.

In the world of construction industry, the era of the new construction seems to be ending in most of the developed countries and how to maintain and manage the existing facilities are considered to be one of the most important paradigms. On the other hand, as the socio-economic activities continue in a large extent, the scale of the production, consumption and scrap have been significantly increasing and the exhaustion of resources and the destruction of environment are being rapidly accelerated throughout the world. It is needless to say that the construction environment should be drastically changed from the era of scrap to the ecological use of existing stock of infrastructure. Therefore, the structural longevity and elongation of the lifetime of infrastructure should be regarded as increasingly important subjects at the present time and in the future.

According to the white paper of the Government of Japan, the lives of typical infrastructure such as roads and bridges are thought to be approximately 60 years and that of harbors and coasts is 50 years, respectively (MLIT 2002). Furthermore, according to Nikkei Newspaper, the annual cost of maintenance, management, replacement of public infrastructure and restoration from natural disasters of Japan would be doubled from that of 2004 to 2030, namely would become from 5 trillion JPY to 10 trillion JPY. According to the detail of the white paper of Japanese Government, the following facts on the infrastructure are worth mentioning. Assuming the total investment remains as it is, the figures from those in the year of 2000 will increase or decrease respectively annually to those in the year of 2025 in Japan:

(1) The investment for the maintenance is predicted to increase, specifically, from 3.8 trillion JPY to 6.2 trillion JPY.

(2) The investment for the renewal is predicted to increase, specifically, from 0.3 trillion JPY to 3.7 trillion JPY.
(3) The investment for the newly-built is predicted to decrease, specifically, from 15.9 trillion JPY to 9 trillion JPY.
(4) The stock of the infrastructure will be vastly accumulated.

2 LIFETIME OF BRIDGES

2.1 *Definition of lifetime*

It is quite confusing to know that the definition of the lifetime varies considerably from places to places. In this paper, the "lifetime" may be defined as the period of time since structures have started to be in service until they cease to be used for some reasons or the final stage when they are possibly in service any further (JSSC 1991).

2.2 *Definition of expected lifetime*

The expected lifetime may be defined as the period of time in which structures are expected to satisfy the demand performance, to possess the physical load-carrying capacity and to fulfill the serviceability.

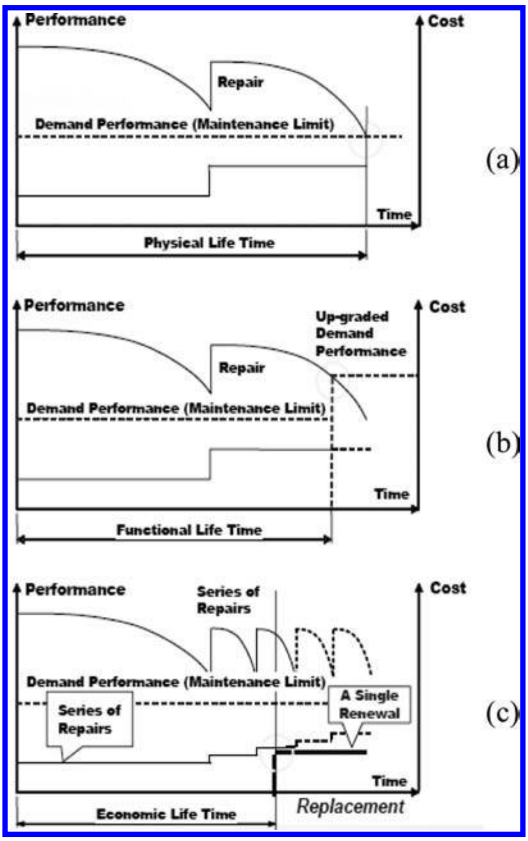

Figure 1. Chronological change of lifetime.

2.3 *Function and performance*

Performance refers to the structural capacity based on the field inspection data and the structural health assessment taking into account the deterioration of infrastructure. On the other hand, the function implies updated capacities meeting the current standard of the wheel loadings, natural forces such as wind and earthquake excitations, river flow, design traffic flow and so forth.

It is a general practice to decide the maintenance plan based on the field inspection data and the structural health assessment of bridges taking into account their deterioration rate. However, the old bridges built in the past were based on different old codes from now with respect to loads, earthquake-resistant design and river conditions and so forth, thus their function does not correspond to the present design codes. Thus, the judgment on the bridge replacement solely on the basis of the bridge health assessment alone may lead to wrong decisions. From this standpoint, the final decision requires the functionality in addition to the health assessment as explained in the following. In this paper, the countermeasure such as earthquake-resistance retrofitting is not considered for mere convenience. However, base-isolation will be regarded as a method to improve the performance in this paper.

2.4 *Classification and concepts of lifetime*

Lifetime may be classified into physical, functional and economic lifetimes (Kato 1983) as shown in Figure 1 and Table 1.

(1) Physical lifetime

It refers to the lifetime of an existing bridge which may have to be renewed by a new bridge upon significant reduction of the load-carrying capacity or by the troubles encountered during the service but deemed difficult to meet these demands by just reforming the present bridge. As shown in Figure 1 (a), it represents the period from the initial state of a bridge to the time when its performance decreases and coincides with the performance corresponding to the serviceability limit. It frequently happens to restore the performance by intermediate repair, retrofitting or replacement works.

Table 1. Comparison of Lifetimes.

Classification of Lifetime	Characteristics	Judgment of Lifetime	Application: Whole Bridge System	Application: Structural Elements: (Girders, Slabs, etc)	Judgment of Economy/Comparison of Cost
Physical Lifetime	Limit of Usefulness	Inspection, Structural Health Assessment	No	Yes	No
Functional Lifetime	Difficult to define the index appropriately and point out when	When budget is insufficient, the lifetime will become longer	Yes	No	No
Economic Lifetime	Replacement is deemed cheaper	Comparison of cost and decision-making	Yes	Yes	Yes

(2) Functional lifetime
It refers to the lifetime of an existing bridge which may have to be renewed by a new bridge when the widening of roadway width or the up-grading of the traffic loads are legally demanded but deemed difficult to meet these demands by only reforming the present bridge. As shown in Figure 1(b), it represents the period from the initial state of a bridge to the time when its demand performance is changed beyond the present performance.
(3) Economic lifetime
It refers to the lifetime of an existing bridge which may have to be renewed by a new bridge when the up-grading the existing bridge is judged much more expensive than building a new bridge. In Figure 1(c), it represents the period from the initial state of a bridge to the time when its cost for the repair and retrofitting become greater than that for the replacement.

Table 2 shows several examples of physical, functional and economic lifetimes.

2.5 *Chronological change of reasons for replacement of bridges*

Figure 2 shows the chronological change of reasons for bridge replacement in Japan (PWRI 1997, BMSG 2004). Figure 2(a) describes the reasons for bridge replacement during the period of 1977–1986. These reasons may be listed in the order of larger percentage:

(1) Improvement of road alignment
(2) Improve the functional obsoleteness
(3) Structural defects such as corrosion
(4) Insufficient load-carrying capacity
(5) Insufficient earthquake-resistance

After 10 years, some of the reasons for replacement of bridges have changed as shown in Figure 2. For example, the replacement due to damage corresponds to only 1/5 of the total replacements. Most of the replacements are due to the improvement of road alignment, refit of river flow and widening of road width. The percentage of the improvement of the functionality changed from 57% to 76%.

2.6 *Definition of technology toward longevity*

The longevity technology refers to the technology and all methods of management including those for inspection, system, timing, judgment, measure and budget to prolong the lifetime beyond the expected lifetime.

The keywords for the longevity may be summarized as:

(1) Increase of bridge stocks
(2) In view of the limitation for budgetary appropriation, the scenario of "renewal of every bridge does not exist any more.
(3) The national and local governmental principle for the preventive management is supported by the authorized experts.
(4) The concept of longevity for those newly-built and already existing are different.
(5) The performance-based specification is encouraged for the longevity by adopting innovative and development technologies.

Figure 3 shows the ratio of the bridges with the lifetime over the age of 50 to those under 50 (MLIT

Table 2. Examples of physical, functional and economic lifetimes.

	Whole Bridge System	Structural Components (main girders, deck plate, etc)
Physical Lifetime	Traffic suspension, left or scrapped due to insufficiency to carry external load. Lifetime is the same as that of components excluding replaceable ones	Only partial repair is judged to insufficient to meet the bearing capacity and only the replacement is the solution
Functional Lifetime	Narrow road width, road alignment is not good, refitting river flow is planned or aesthetically poor	Road width, road alignment depend on that of the whole bridge system. Partial aesthetic retrofitting may be possible
Economic Lifetime	Replacement is judged better than continual repairs	Replacement is judged better than continual repairs

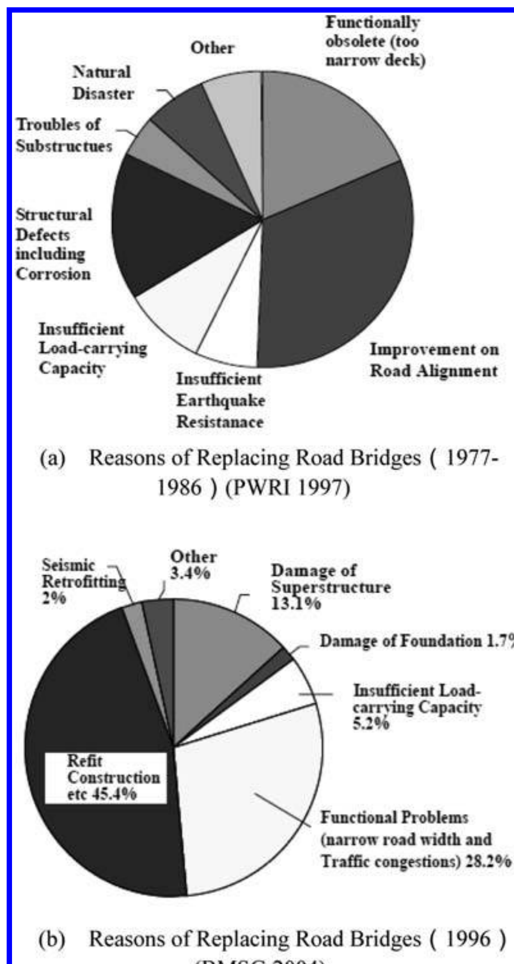

Figure 2. Chronological change of lifetime of Japanese roadway bridges.

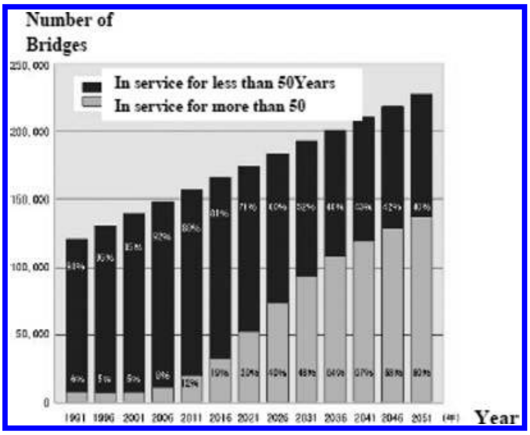

Figure 3. Chronological change of ratio of the bridges with the age over 50 to those under 50.

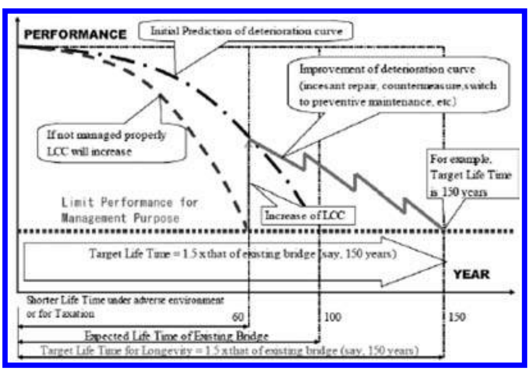

Figure 4. Performance and lifetime of bridges (Abe 2008).

2008). It may be apparent that after the year of 2030, the ratio of bridges over the age of 50 becomes more significant.

2.7 *Target lifetime*

The target lifetime is not defined by the structures but by the manager.

(1) It is the matter of consciousness of the managers and the public opinion.
(2) The limit performance for management varies depending on the demand performance of the time.
(3) Theoretically, steel structures are considered to be ever-lasting structures.
(4) The target lifetime may be considered to be 1.5 time of the lifetime of existing structures as shown in Figure 4 (Abe 2008).
(5) Even at the end of the target lifetime, the structures may not have to be renewed if they posses the sufficient structural health.

Table 3. Chronological characteristics of replacement of steel bridges (NILIM 2004).

Completed Year	Average Life-time in Years	Standard Devia-tion in Years	Remarks
1920-1930	50	10	
1931-1940	40	10	
1941-1950	30	10	2nd World War
1951-1960	60	20	
1961-1970	70	20	
1971-1980	70	20	
1981-1990	100	30	Scarce data
1991-2000	100	30	Scarce data

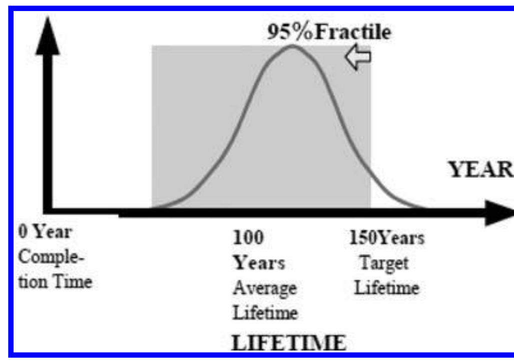

Figure 5. Target lifetime for bridges.

2.8 *Management for longevity*

The lifetime can be made longer or shorter than the initially expected one depending on the management method.

2.9 *Setting of basic parameters for determination of lifetime for steel bridges*

The National Institute for Land and Infrastructure Management, NILIM, showed the statistics of the number of replaced bridge plotted against the year of replacement.

The statistical distribution is assumed to follow the Normal distribution and the parameters are so determined as to minimize the errors between the achieved record and the prediction. Table 3 shows the average lifetime and standard deviation of bridges built from 1920 to 2000 (NILIM 2004).

For those bridges built during the period of so-called "rapid economic progress" from the year 1961 to 1980 the statistics show that the average lifetime is 70 years with the standard deviation of 20 years. Thus, if the normal distribution is assumed and 95% fractile of non-exceedance is assumed, this fractile corresponds to 1.65σ, the longevity of prolonged lifetime of 35 more years can be expected ($20 \times 1.65 = 35$) as shown in Figure 5. While those bridges built in recent years from 1981 to 2000, the average lifetime is 100 years with the standard deviation of 30 years. By the similar token, the longevity of 50 more years may be expected ($30 \times 1.65 = 50$).

Table 4. Expected average lifetime of bridges for different probabilities of non-exceedance.

Era	Average Life Time	Expected average lifetime of bridges for Probabilities of non-exceedance		
		90%	95%	99%
From 1960's to 1970's	70	96 Yeas	103 Years	117 Years
From 1980's	100	139 Years	150 Years	170 Years

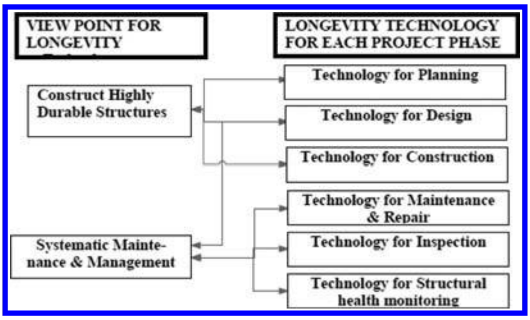

Figure 6. Vision of longevity technology and expected developments of technologies for each step of construction projects.

If different fractile other than 95% is assumed, the expected average lifetime for steel bridges may be as listed in Table 4. Thus, for recent bridges built after 1980, the expected lifetime may be regarded to be 150 years.

3 PRESENT STATUS OF TECHNOLOGIES FOR LONGEVITY OF BRIDGES

3.1 *Change of social demand*

The philosophy of essential maintenance, namely, "rebuilding bridges when they become older than the lifetime" in the past is nowadays shifting to the philosophy of preventive maintenance and building durable bridges to prolong their service life.

3.2 *Type of damages for steel bridges*

Most damages are caused by the defective structural characteristics and environmental effects. The most popular damages of steel bridges are considered to be fatigue and corrosion.

3.3 *System for longevity technology*

Figure 6 shows the vision of longevity technology and expected developments of technologies for each step of construction projects.

3.4 *Cause of damages for bridge parts and structural members*

Figure 7 shows the causes of damage of bridge parts and elements of steel bridges. Described are the

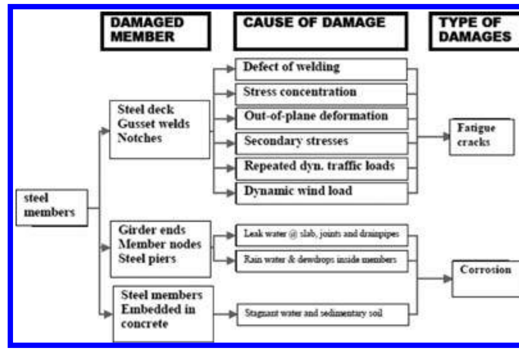

Figure 7. Causes of damage of bridge parts and elements.

locations of damaged steel parts or members, causes of damage and the type of damages.

It will be seen that the fatigue and corrosion are the most popular damages for steel bridges.

3.5 *Anti-fatigue technologies*

For road bridges, the fatigue design has been introduced in Japan only quite recently. The countermeasure consist of reduction of stress concentration, reduction of residual stresses. Not only in the maintenance but also they should be taken into account at the planning and design steps. Recently, the following two countermeasures are to be noted.

(1) *anti-fatigue steel* The steel itself has a desirable characteristics of fatigue-resistance through the adjustment of chemical ingredients and metal texture.
(2) *UTI, Ultrasonic Impact Treatment* Impact of ultrasonic wave is applied to the weld in order to have the following effects:

   (a) Tensile residual stresses can be converted to compressive.
   (b) Reduce the stress concentration by smoothing out the surface of weld.
   (c) Increase the surface toughness.

3.6 *Anti-corrosion technologies*

As has been described above, the corrosion is regarded as one of the most serious problems for steel structures. Not only to those in the area near the sea zone susceptible to salt water but to those cold area where anti-freeze is used inevitably, bridges tend to be corroded much faster than in the other locations. Furthermore, attention must be paid to some specific locations of bridge parts of elements where water tends to be stagnant and pooled or the humidity is easily kept high. Sometimes dewdrops become also harmful unless they dry out. Figure 8 shows an example of dehumidifier used to keep inside a girder dry (Kaneko 1999). Figure 9 also shows an example of dehumidifier

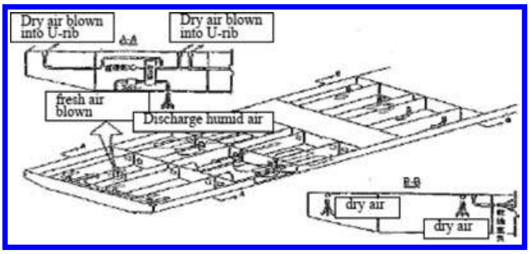

Figure 8. Dry air dehumidifier inside a bridge deck.

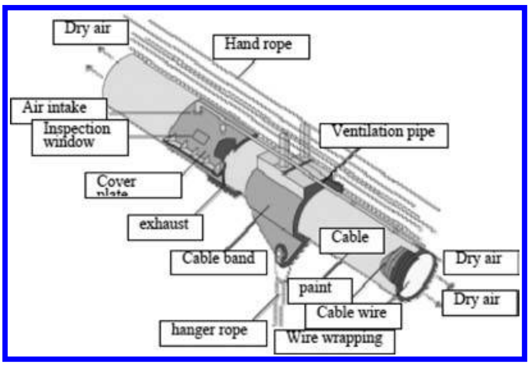

Figure 9. Dry air infusion system for cables.

used for main cables of a suspension bridge (Kitagawa 2001).

(1) Improvement of corrosive environment

(a) This is to dry up inside girder by removing humidity to prevent dewdrops using either dehumidifiers or ventilators.
(b) Clean-up technologies: This is to remove the dirt and salt from the surface

(2) Cathodic protection methods
Because of the severe corrosive environment, corrosion protection should be made, especially for the parts just below the M.L.W.L. when severe local corrosion occurs. For such parts, cathodic protection is generally applied.

(a) Cathodic protection without external electric power
(b) Cathodic protection with external electric power
(c) Cathodic protection in the air

(3) Other anti-corrosion methods

The other coating methods include (a) painting, (b) organic lining, (c) petrolatum lining and (d) inorganic lining. The inorganic linings include metal linings such as titanium-clad lining (JTS 2000, Nippon Steel 2000), stainless steel lining, thermal spraying using zinc, aluminum and aluminum alloys (JAPH 1999). Figure 10 shows an example of cathodic

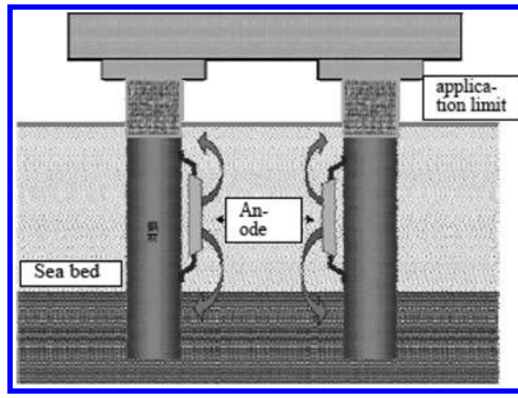

Figure 10. Cathodic protection (without external source power).

protection without external power source (Ishida 2006).

3.7 *Other technologies*

(1) Technologies for planning and design stages
The assessment is required taking into account LCC considering the construction site and selection of structural type.
(2) Inspection technology

(a) Anti-fatigue inspection: In the past, the crack inspection was first done by flesh eyes. After confirmation of problems such as peeling-off of painting some detection tests were carried out. However, this method has many shortcomings such as cost, labor, and impossible when eye observation is impossible. ◇A wireless detection technology may be applied by placing IC tags at the place where cracks are expected to initiate.
(b) Anti-corrosion inspection: It may be very useful to find out corrosion at the place where eye observation is impossible such as reinforcements in concrete.

4 EXPECTED TECHNOLOGIES FOR LONGEVITY

4.1 *Current attitude of road owners*

Depending on the demand performance, road owners are interested in the following structural maintenance matters:

1) Essential maintenance to repair a great damage.
2) Preventive maintenance to repair a light damage
3) Up-grading of wheel loads and earthquake retrofitting
4) Daily maintenance such as cleaning up road surface, drainage pipes.

4.2 *Several problems to be solved*

There are many problems remain yet to be solved. These may be summarized below:

(1) Difference of management level (demand performance)
Not only the natural environment but also socio-economic environment are to be considered for the determination of lifetime and maintenance management.

(2) Limitation of budget (evaluation of LCC)
Balancing of cost for maintenance management by essential management and that by preventive maintenance must be established.

(3) Effective use of data (data sharing)
This is effective when combined with management system and good for accountability.

(4) Succession of technology
The education of young engineers and out sourcing are important.

(5) Effective inspection method (frequency and methodology)
Priority in inspection, remote inspection and structural health monitoring technology are expected to be established.

5 NEEDS FOR STRUCTURAL HEALTH MONITORING AND REQUIRED PERFORMANCE

5.1 *Background for survey by sending out questionnaires to owners of structures*

Attempts were made to communicate with several owners of bridges to find the needs of structural health monitoring by sending questionnaires to them. H However, these are omitted again.

6 SURVEY OF STRUCTURAL HEALTH MONITORING TECHNOLOGIES AT THE PRESENT TIME

6.1 *Summary of investigation*

An extensive literature survey were conducted to know the technologies available at the present time and future applications.

6.1.1 *Purpose and method of investigation*

The purpose of investigation is as follows:

(1) to understand the present state of development of structural health monitoring technology
(2) to construct DB for references on structural health monitoring
(3) to extract possibilities and problems of application of structural health monitoring toward longevity of steel structures

The methodology of investigation is as follows:

(1) to collect the information of related books, reports including committee reports, and seminars that are published in Japan
(2) to collect references on typical international conferences on the structural health monitoring
(3) to understand the present trend of technologies for structural health monitoring and evaluation of performance
(4) to classify the existing technologies by summarizing the target of application and methodology in the form of matrix
(5) to summarize the prospect of application of structural health monitoring technologies after reviewing references

6.1.2 *Target references of investigation*

The references reviewed are shown in Table 5.

6.2 *Reference survey*

6.2.1 *Summary of references*

Totally 136 references were reviewed and summarized by Tables 10 and 11. Although a great number of figures and tables were contained. They are not shown in this paper because of the restriction of page numbers.

Table 5. List of references surveyed.

	classification	name	authors / publication	year
domestic	books	Health structural health monitoring	S.Yamamoto et al	1999
		Maintenance engineering of infrastructure	JSCE committee on maintenance engineering	2004
	Symposium and seminars	Report "basic study on advanced structural health monitoring on ships"	JSNA (Japan Ship Technology Research Association)	1998
		Present status and prospect for maintenance of railroads	Railway Technical Research Institute seminar	2007
		Workshop on health diagnosis for buildings	Committee of structures, AIJ (Architectural Institute of Japan)	2005
		Prospect of structural health monitoring of performance implemented by ubiquitous technology	Committee of Information system, AIJ	2007
		Journal of steel structures and bridges, JSCE	Committee of steel structures, JSCE	2008
		Seminar on "New inspection and structural health monitoring technologies for steel structures"	Committee of steel structures, JSCE	2007
		Guideline for structural health monitoring of bridge vibrations	Committee of structural engineering, JSCE	2000
		Structural health monitoring technologies for concrete structures	Committee of concrete, JSCE	2007
		Assessment of residual performance and recovery technologies for corroded steel structures	Committee of steel structures, JSCE	2007
	Periodicals	Structural health monitoring Series	Journal of The Society of Naval Architects of Japan (SNAJ)	1992
		Special issue "technologies supporting life lines and others	Journal of JSNDI (The Japanese Society for Non-destructive Inspection	2006
Oversea	Journal	Structural Control: Past, present, and future	Engineering Mechanics, ASCE	1997
		New sensors, Instrumentation and Signal Interpretation	Infrastructure Systems, ASCE	2008
	Conference Proceeding	Structural Control and Structural health monitoring	IASCM	2006
		Structural health monitoring and Intelligent Infrastructure	ISHMII	2007
		Application of Statistics and Probability in Civil Engineering	ICASP	2007
		Smart Structures and Materials	SPIE	2006

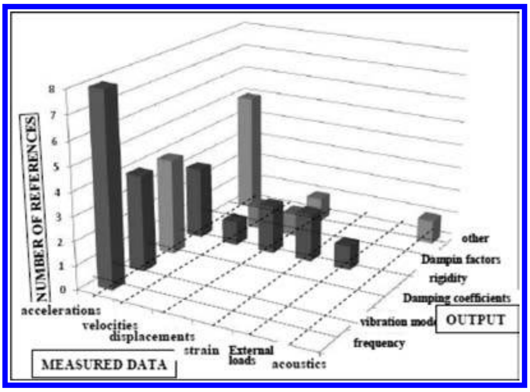

Figure 11. Number of references with respect to measured physical quantities and output.

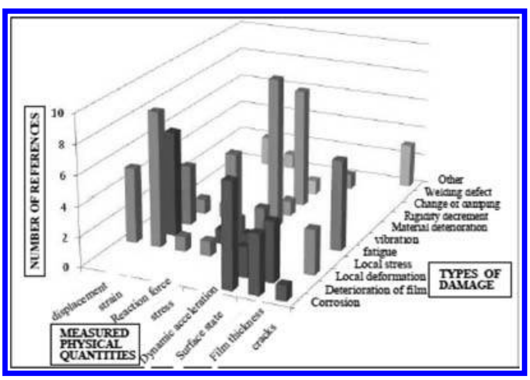

Figure 12. Number of references in the matrix form with respect to measured physical quantities and types of damage.

6.2.2 *Representation of matrix*

The result of the survey on sensing technologies and assessment methods is conveniently represented by a matrix method as shown in Figures 11 and 12.

The following observations may be made:

(1) Status of developments of sensing technologies

a. Majority of measured physical quantities are strains and accelerations. However, the number of instances of stresses, reaction forces, ph and temperature is quite few.

b. Local deformations and vibrations constitute the majority of the diagnosed damages. The number of instances of direct measurements is quite few for material deterioration and decrement of rigidity target damage.

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 ISBN 978-0-415-87786-2

Wireless sensing technologies for bridge monitoring and assessment

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ABSTRACT

Recently, there has been increasing need and interest in adopting wireless sensors to structural health monitoring (SHM) applications. Smart wireless sensors have been considered as an alternative tool for convenient and economical realization of SHM system. The wireless sensors can provide rich information which SHM algorithms can utilize to detect, locate, and assess structural damage caused by progressive deterioration under operational loadings and severe loading events. Many applications of wireless technologies to real structures have been made for on-line wireless SHM of in-service civil infrastructures. This paper presents several hardware/software issues and example applications of smart wireless sensing technologies. The first example is smart wireless sensor network, which is applied to a cable stayed bridge. The second one is a multifunctional wireless impedance sensor node, which is fabricated and applied to a real steel bridge. The last one is an innovative wireless system for guided wave excitation and sensing based on laser and optoelectronic technologies, which has been experimentally demonstrated.

1 INTRODUCTION

The modern civil engineering structures are becoming more complex and are expected to be fully functional under severer environmental conditions. Hence, there has been a demand to monitor structures in order to ensure safety and serviceability to the community. Structural health monitoring (SHM) is a methodology to monitor the performance of a structure and identify incipient failure conditions, which helps in improving the safety and life span of the structure. The SHM system often offers an opportunity to reduce the cost for the maintenance, repair, and retrofit throughout the life-cycle of the structure.

However, in the conventional SHM system, the expensive cost for purchase and installation of the SHM system components, such as sensors, data loggers, computers, and connecting cables, is a big obstruction. SHM systems generally require coaxial wires for communication between sensors and the data repository but the installation of coaxial wires in real structures can be very expensive and labor-intensive. For example, it was reported that a SHM system installed in a tall buildings generally cost in excess of US$5000 per sensing channel (Celebi 2002).

To overcome these problems, there have been ongoing efforts to adopt smart wireless sensors to SHM applications (Straser et al. 1998, Lynch et al. 2006a). Smart wireless sensor is an emerging sensor with the following essential features: sensing, onboard processing, wireless data transmission, and power supply (Nagayama 2007). When many sensors are implemented on a SHM system for a sizable civil structure, wireless communication between sensors and data repository seems to be attractive in the aspects of the cost effectiveness. Dense arrays of low-cost smart wireless sensors have the potential to improve the quality of the SHM dramatically using their on-board computational and wireless communication capabilities.

This paper presents three wireless sensor and sensing technologies with their hardware/software issues and applications to monitoring of bridge structures as (1) wireless vibration sensor network and application to a large scale SHM system for a cable stayed bridge which consists of 70 wireless sensor nodes with 420 channels, (2) multifunctional wireless impedance sensor node and application to local monitoring and damage detection of a steel bridge, and (3) wireless actuation and sensing system using laser and optoelectronic technologies for guided waves based structural health monitoring.

2 WIRELESS VIBRATION SENSOR NETWORK FOR BRIDGE MONITORING

To address critical issues on wireless smart vibration sensors and to promote related R&D activities, such as (1) data synchronization and recovery of missing

Figure 1. The 1st (right) and 2nd (left) Jindo Bridges.

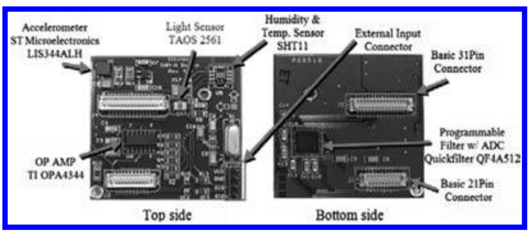


Figure 2. SHM-A sensor board (Rice et al. 2010).

data, (2) on-board and decentralized processing, (3) autonomous operation, (4) power management and energy harvesting, and (5) environmental hardening, an international test bed was developed on a cable-stayed bridge (the 2nd Jindo Bridge shown in Figure 1) in Korea through a trilateral research collaboration among Korea (KAIST), the US (University of Illinois at Urbana-Champaign), and Japan (University of Tokyo).

2.1 *Hardware of wireless smart sensor network*

The key hardware components of a wireless smart sensor node are an Imote2 (Crossbow 2007), a sensor board, and a battery board. An Imote2 can be interfaced with a SHM-A multi-scale sensor board to measure 3-axis acceleration as well as temperature, humidity, and illuminance, or a SHM-W sensor board to measure wind speed and direction interfaced with 3-axis anemometer. The SHM-A sensor board contains a 3-axis accelerometer, thermometer, hygrometer, and luxmeter, whose signal is digitized by the embedded signal conditioning chip with user-selectable sampling rates and programmable digital filters as shown in Figure 2. This board allows to measure data from other types of sensors such as anemometers and strain gages. Four sampling frequencies (10, 25, 50, 100 Hz) have been pre-programmed on the SHM-A board for bridge monitoring applications. The SHM-W board was developed by modifying the SHM-A board to have three external 0–5 V input channels to be interfaced with a 3-D ultrasonic anemometer. The wind speed in horizontal and vertical wind directions are measured through analog input interface connectors on the SHM-W board. The wind data can be acquired precisely synchronized with the bridge acceleration data from the SHM-A board. The IBB2400CA battery board powers the Imote2 using three 1.5-volt batteries. To investigate the feasibility of sustainable energy harvesting, solar panels and rechargeable batteries have been adapted on several sensor nodes installed at the locations hardly reachable.

2.2 *Software of wireless sensor network*

An open-source middleware services toolsuite, which interacts between the target SHM application and operating system of hardware, were developed to help the civil engineers easily code into the wireless smart sensor nodes. The toolsuite contains basic middleware to provide high-quality sensor data and to transfer the data reliably to the base station via wireless communication as well as a library of numerical algorithms. The toolsuite components are categorized into foundation services, tools and utilities, application services, and continuous and autonomous monitoring services. (Mechitov et al. 2004, Nagayama & Spencer 2007, Rice et al. 2008).

To operate large-scale wireless smart sensor network in this study, 3 problems are seriously considered: 1) energy saving software architecture, 2) data inundation, and 3) continuous and autonomous operation. To save batteries of large number of nodes, the network is allowed to sleep default and wakes up periodically for a short time to listen to broadcasted commands. To prevent data inundation from large number of sensor nodes, preset sentry nodes wake up at a preset interval and measure a short period of data to make a decision on the measurement based on preset threshold. Autonomous SHM network management software is utilized for the base station to enable the automatic, continuous monitoring with reduced power consumption.

2.3 *Deployment of SHM system on Jindo Bridge*

The developed hardware and software framework has been deployed on the 2nd Jindo Bridge to realize SHM system using wireless sensor network. The network was divided into two sub-networks with different radio channels: one on the Jindo side and the other on the Haenam side as shown in Figure 3. A total of 70 nodes are deployed on the bridge. They contain SHM-A sensor boards to measure 3-axis acceleration mainly, except one node which contains SHM-W board interfaced with an anemometer to measure wind speed and direction. The Jindo subnetwork consists of 33 nodes with 22 nodes on the deck, 3 nodes on the pylon, and 8 nodes on the cables. The Haenam sub-network consists of 37 nodes with 26 nodes on the deck, 3 nodes on the pylon, and 7 nodes on the cables. The sensor nodes were enclosed in water-tight plastic enclosures for

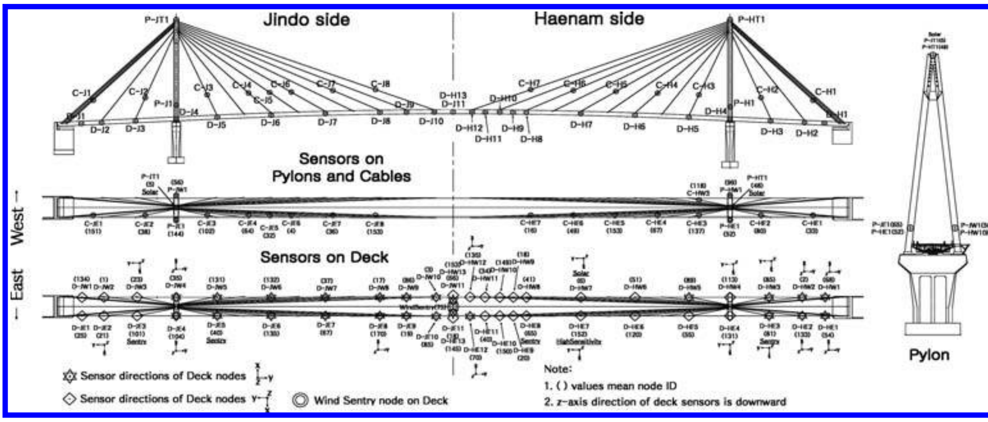


Figure 3. Locations of sensor nodes deployed on the 2nd Jindo Bridge.

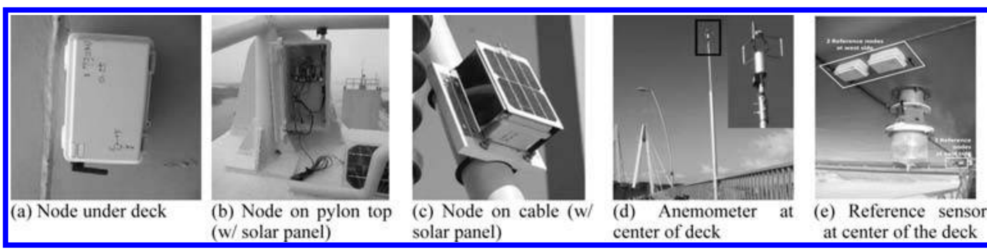


Figure 4. Various types of wireless sensor nodes installed on the 2nd Jindo Bridge.

protection from harsh surroundings. The deck/pylon nodes are mounted using one-directional magnets attached to the bottom of enclosure, and cable nodes are mounted on an aluminum plate with round interface to fit the cables. An anemometer is installed on a 5 m tall at the center of deck to prevent any interruption of the bridge. Most of the nodes are powered by 3 D-cell batteries, while 8 nodes (2 nodes on the tops of both pylons, and 5 cable nodes and 1 deck node of Haenam sub-network) are powered by solar panels with rechargeable batteries.

Each sub-network is controlled by a corresponding base station located at the tops of the concrete piers supporting the steel pylons of the 1st Jindo Bridge. The locations of base stations are selected to secure consistent line-of-sight communication with the sensor nodes. The base station is composed of an industrial personal computer (PC), an uninterrupted power supply (UPS) backup, a gateway node, and ADSL internet modem, and it is covered by an environmentally hardened enclosure. The parameters of the wireless sensor network are set after a series of laboratory and preliminary in-field tests.

The performance of the system has been evaluated in terms of hardware durability, software stability, power consumption, and energy harvesting options, as well as the accuracy of the identified modal properties by an output-only modal identification method shown in Figure 5.

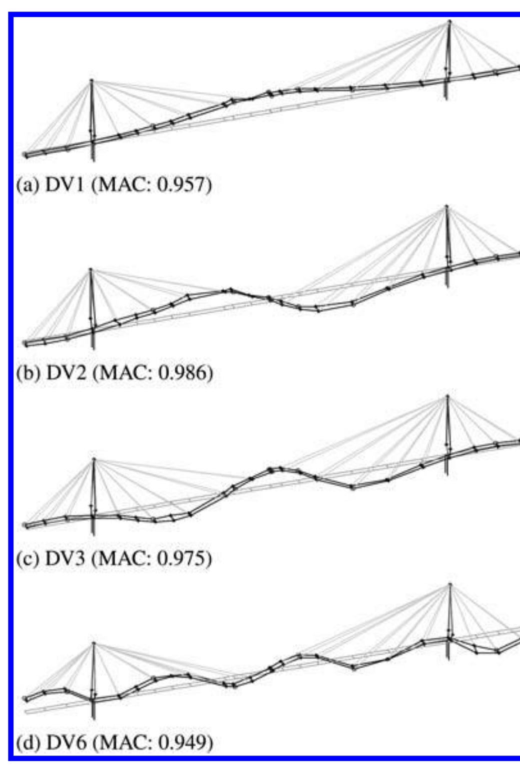


Figure 5. Identified mode shapes of the 2nd Jindo Bridge.

3 WIRELESS VIBRATION SENSOR NETWORK FOR BRIDGE MONITORING

3.1 *Electromechanical impedance-based SHM*

In this paper, multifunctional wireless impedance sensor nodes are presented for low-cost and low-power excitation/sensing, structural damage detection using embedded algorithms, and energy scavenging. The electromechanical impedance-based SHM techniques are embedded in the nodes. Recent advances in online SHM, including low-cost impedance measuring chip, on-board computing, and radio-frequency (RF) telemetry, have improved the accessibility of the impedance method for in-field applications (Grisso and Inman, 2005, Mascarenas et al., 2007 & 2009, Overly et al. 2007). Based on those prior researches, the present impedance-based wireless SHM node is developed by focusing on the following three objectives: (1) to develop in-field adjustable impedance-based wireless sensor nodes with on-board algorithms for structural diagnosis, (2) to incorporate the energy scavenging system for maintenance-free wireless sensor node, and (3) to investigate the feasibility of the impedance-based SHM system to real bridge structures.

3.2 *Wireless impedance sensor nodes*

The proposed wireless sensor node is composed of four functional subsystems: sensing interface, computational core, wireless transceiver, and power supply. (1) The "sensing interface" includes an interface to which a piezoelectric sensor and a temperature sensor can be connected, and an impedance chip (AD5933) for exciting a piezoelectric sensor and measuring the impedance signals. (2) The "computational core" consists of a microcontroller and a serial flash memory for computational tasks and system operations with various embedded algorithms. (3) The "wireless transceiver" is an integral component of the wireless system, which is composed of a RF transceiver (CC2420), a balun transformer, and an antenna to communicate with a base station and/or other wireless sensor nodes and to broadcast the structural condition. (4) The sensor node can be operated by one of three type "power supply" systems: 5 V AC-plug DC adapter, 3.6–7.2 V battery, or 5 V solar power system. The power can be monitored on the microcontroller using a general ADC, which transforms the analog signals acquired from batteries to the digital signals. For stable power supply to the sensor node during operations, LDO (Low-dropout regulator) is mounted for providing a fixed 3.3 V reference output to the sensor node. Solar power system for energy harvesting consists of single crystalline silicon solar cells ($120 \times 60\,\mathrm{mm}^2$) to generate the maximum power for its size, two AA Ni-MH re-chargeable batteries to stand high temperature and overcharging under sunlight, and a step-up DC/DC solar controller to protect the appliances and the batteries with over discharge prevention circuit. Figure 6 shows the impedance sensor node developed in this study and the features are described in Table 1.

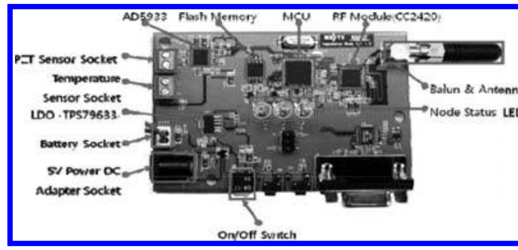


Figure 6. Proposed wireless impedance sensor node.

Table 1. Features of the proposed wireless impedance sensor node.

Frequency Range	$1 \sim 100$ kHz
Frequency Resolution	>1 Hz
Impedance Range	$1\,\mathrm{k\Omega} \sim 1\mathrm{M\Omega}$
Temperature Range	$-20 \sim 120°\mathrm{C}$
Temperature Resolution	$>0.03°\mathrm{C}$
Operating Frequency	2.4 GHz IEEE 802.15.4
Outdoor Transmission Range	150 m (2dBi Dipole Antenna)
Power Supply Options	5 V AC-DC Adapter; Commercial batteries (3.6–7.2 V); Ni-MH rechargeable batteries with Solar Panels
Feature	$150 \times 100 \times 70$ (mm); 310 (g)

For autonomous SHM using wireless sensor nodes, it is strongly required to construct the embedded data analysis system. In the proposed sensor node, multifunctional algorithms are implemented for temperature/power measurement, impedance measurement, and analysis engine for both structural damage detection and sensor self-diagnosis. Here, two algorithms are embedded on the microcontroller for the structural status monitoring: the RMSD metric and the temperature compensated CC metric calculated by EFS method. Sensor self-diagnosis is simply carried out calculating the slope of the imaginary part of admittance.

3.3 *Applications*

A field test has been performed to detect damages including loosened bolts and notches on the Ramp-G bridge in Incheon, Korea. The bridge is a traditional steel box girder bridge with a reinforced concrete deck as in Figure 7. One sensor node was installed near the bolted joint of the outer girder and connected to a PZT patch of $50 \times 50 \times 0.5\,\mathrm{mm}^3$ and a NTC thermistor surface-mounted at a distance of 200 mm from the bolted joint using epoxy glue. The maximum temperature variation obtained from a NTC thermistor was 15°C during the test of 2 days, and the voltage drop of battery was 0.004 from 3.476 to 3.472. The E/M

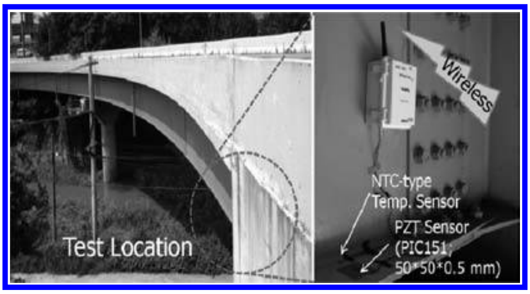


Figure 7. Experimental setup for impedance-based SHM on Ramp-G bridge.

Table 2. Damage scenarios for detecting multiple damages on a bridge girder.

Case	Damage Description	Case	Damage Description
1	No damages	5	Additionally loosened Bolt #1 (2 Turns)
2	Loosened Bolt #1 (2 Turns)	6	Additionally loosened Bolt #1 (2 Turns)
3	Hand-retightened Bolt #1	7	Additionally Notch #2
4	Notch #1	–	–

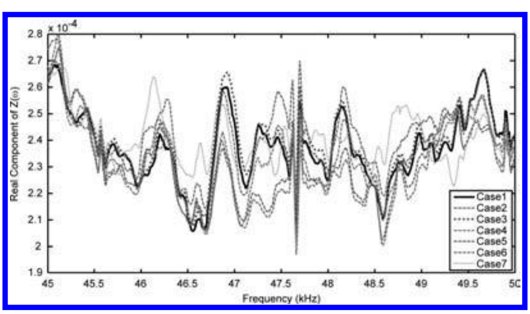


Figure 8. Measured impedance signatures at various damage cases.

impedances were measured at a frequency range of 45–50 kHz for the baseline. Here, it was assumed that there are not any damages within the sensing range of the attached sensor. Tests were carried out for seven damage scenarios described in Table 2. The first damage was simulated by two-turn loosened Bolt #1, and then Bolt #1 was hand-tightened. Cracks and multiple damages were induced in a sequence. Example cases of the measured impedance signals at different damage conditions were compared with the baseline data in Figure 8, from which significant variations is observed in the measurements. Figure 9 shows temperature-compensated CC values, which were computed on the on-board microcontroller and wirelessly transmitted to a base station. It is found that the CC value decreases when Bolt #1 gets loosened, and it is almost recovered as the bolt is hand-tightened. It should be

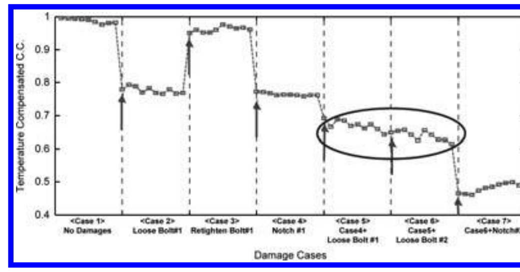


Figure 9. Temperature-compensated CC indices for multiple damage detection on a bridge.

noted that the condition of hand-tightened Bolt #1 is not same with original condition (baseline). Then, the CC value decreases as the damage severity increases due to loosened bolts and artificial notches. Here, the CC metric for Cases 5 and 6 have similar values, which might be because Bolt #2, which got loosened in Case 6, is located fairly far from the sensor. Through in-field test, it is expected that the actual damages can be effectively identified by the developed sensor node and on-board processed damage indices.

4 WIRELESS ACTUATION AND SENSING FOR GUIDED WAVES BASED SHM

4.1 *Guided wave-based SHM*

Guided wave based SHM techniques have attracted much attentions, because they are not only sensitive to small defects but also capable to cover over a long distance in plate and pipe like structures. A number of studies have demonstrated the potential of guided wave based SHM (Raghavan et al. 2007, Giurgiutiu 2008).

Piezoelectric lead zirconate titanate (PZT) elements could be a good candidate for such online applications due to its small size, easy installation, low cost, non-intrusive nature, and wide frequency response range. PZT transducers typically require wires to supply the power necessary for generation of guided waves and to transmit the sensed data, but the installation of wires in real structures can be expensive and labor-intensive. To overcome these problems, there are on-going efforts to integrate a PZT transducer with a wireless sensor unit (Lynch et al. 2006). One of major challenges in such wireless systems is to secure power necessary to operate the wireless sensors. However, because guided wave based active sensing devices demand relatively high electric power compared to conventional passive sensors such as accelerometers and strain gauges (Yeatman 2009), existing battery technologies may not be suitable for long-term operation of active sensing devices.

4.2 *Wireless actuation for guided wave based SHM*

The ultimate goal of the research is to develop an optical system for guided wave generation and sensing. Figure 10 shows an overall schematic of the proposed

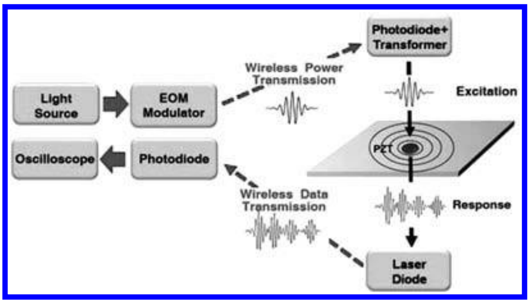


Figure 10. An overall schematic for optics based wireless guided wave generation and sensing.

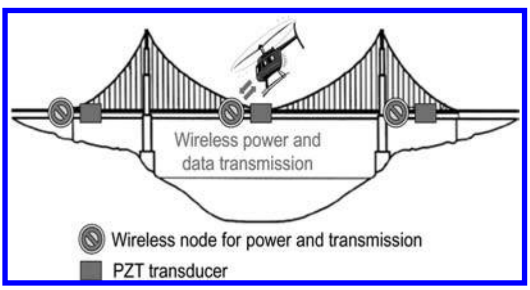


Figure 11. Integration of optics-based active sensing technology and unmanned autonomous inspection robot.

wireless guided wave excitation and sensing system. It takes advantage of optical techniques for both guided wave generation and sensing. First, a laser wirelessly transmits a generated waveform to a PZT transducer node that consists of a photodiode, a transformer and a PZT transducer. Then, the photodiode converts the light into an electric signal, and the transformer increases the voltage level of the created electric signal. The electric input signal excites the PZT attached on a structure, and the excited PZT consequently creates guided waves within the structure. Next, the corresponding reflected waves are measured by the identical PZT and re-converted into a laser and transmitted back to another photodiode located in the data acquisition unit for diagnosis.

Because power is remotely transmitted to the PZT transducer, no complex electronic components are necessary at the PZT node, making it possible to develop a self-contained, rugged, and non-intrusive PZT node. The transducer itself can be entirely passive and consequently will have a long lifetime when it has rugged packaging. In addition, any arbitrary waveform can be generated by a laser using an electro-optic modulator (EOM).

This paper mainly focuses on the excitation aspect of the overall system. A further research for wireless guided wave sensing is currently being developed by the authors using other optoelectronic devices, such as photovoltaic panels and laser diodes (Park et al. 2010). Finally, as shown in Figure 11, it is envisioned that the develop technology can be integrated with autonomous moving agents such as robots to

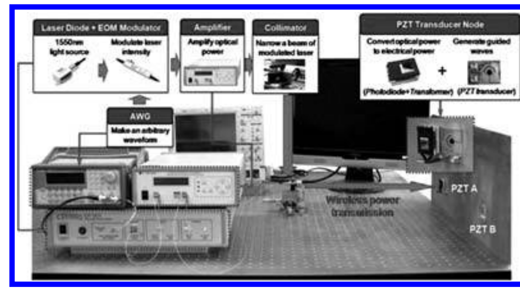


Figure 12. An overall experimental setup of the proposed wireless guided wave excitation system.

remotely inspect the integrity and performance of large distributed infrastructure systems such as bridges.

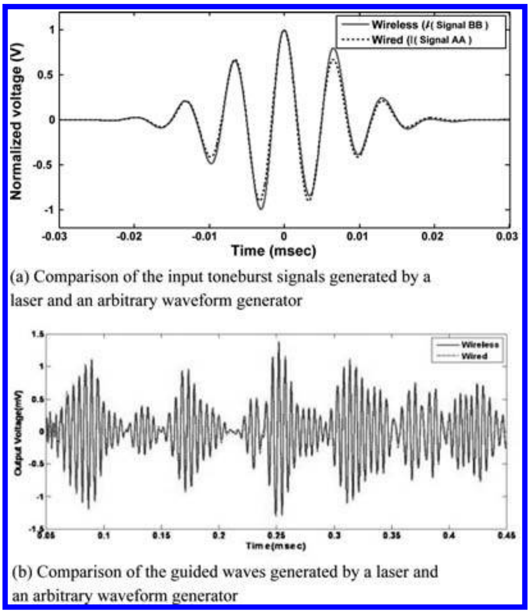
4.3 *Experimental characterization*

The feasibility of the proposed method for wireless guided wave generation has been experimentally demonstrated. The overall test configuration and the test specimen are shown in Figure 12. The system was composed of a laser diode as a power source, an EOM with an arbitrary waveform generator (AWG) for intensity modulation of the laser, an optical amplifier, a collimator, a photodiode, a transformer, and PZT transducers attached on an aluminum specimen.

The output power of the laser diode used in this experiment was 10 mW and controlled by the laser driver. Using the AWG, a toneburst signal with 2 V peak-to-peak voltage was generated at a driving frequency of 150 kHz and exerted to the EOM for intensity modulation of the laser. This modulated laser power was amplified up to 80 mW by the optical amplifier and transmitted by optical fiber to the collimator. The collimated laser was emitted into air and aimed at the integrated photodiode and transformer for conversion to an electric signal and amplification of voltage level as shown in Figure 13(a). Then, PZT A generated guided waves, and the responses were measured at PZT B. PZT A was also excited by using conventional wire connection to the AWG, and the corresponding responses were measured at PZT B for the purpose of comparison as shown in Figure 13(b). The results confirm that the laser-based guided wave generation technique exhibits the reasonable performance by comparing the conventional wiring guided wave generation method.

5 CONCLUDING REMARKS

This paper discusses hardware and software issues on smart wireless sensor and sensing technologies, and presents several recent developments and applications to bridge monitoring. Firstly, the deployment and evaluation of a state-of-the-art wireless smart sensor network on a cable-stayed bridge are reported. Then multifunctional wireless impedance sensor nodes are presented for low-cost and low-power excitation/



(a) Comparison of the input toneburst signals generated by a laser and an arbitrary waveform generator

(b) Comparison of the guided waves generated by a laser and an arbitrary waveform generator

Figure 13. Experimental results.

sensing, temperature/power monitoring, structural damage detection using embedded algorithms, and energy scavenging. Lately, an innovative wireless system is proposed for guided wave excitation and sensing based on laser and optoelectronic technologies. The feasibility has been experimentally demonstrated.

The wireless sensor and sensing technology is the enabler of realization of cost-effective monitoring system. The system-on-chip data acquisition system without tethering work lowers the cost of sensors themselves as well as of installation, maintenance, and replacement. The wireless data transmission removes the cost related to the cables, and therefore, promotes the free clustering of sensor networks. The on-board computation enables the quick and easy on-line monitoring without concerns on data inundation. However, there still remain some limitations in wireless communication and energy supply for wireless sensor networks. Wireless communication inevitably requires quick and reliable data communication protocol, recovery of missing data, and time synchronization among sensors. To overcome current capacity of battery technology, which may not be enough to operate wireless sensor networks in mid- or long-term, power management as well as power harvesting from ambient vibration, wind, and temperature, are now actively under development. Multidisciplinary researches as well as education can facilitate the rapid development as well as application.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Passive sensors for monitoring corrosion in concrete bridges

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ABSTRACT

A class of low-cost wireless sensor has been developed at the University of Texas at Austin to monitor corrosion in concrete bridges. The sensors are powered and interrogated in a wireless manner using inductive coupling between an external reader coil and the embedded sensor (Figure 1). The sensors are designed to be placed in concrete during construction and interrogated periodically over the service life of the bridge.

Previous research efforts have focused on detecting the onset of corrosion using a threshold sensor. The design employed a sacrificial steel wire that is physically connected to a resonant circuit. The corrosion of the wire turns the embedded sensor into an open circuit and alters its phase response. So, when the threshold sensor is interrogated two limit states can be easily recognized (Corrosion / No Corrosion).

In this paper, a new generation of sensors is presented that can be used to provide analog output describing the level of corrosion within the concrete. The analog sensor incorporates a sacrificial corroding element that is placed entirely outside the resonant circuit and interacts with it only by inductive coupling.

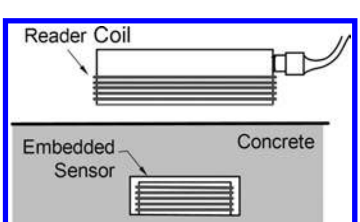

Figure 1. Wireless interrogation of embedded passive sensor using external reader coil.

The lack of physical contact between the corroding element and the resonant circuit in the new arrangement prevents corrosion products from reaching the circuit components. This configuration was selected to address a durability concern in the original threshold sensor (Puryear 2007).

Two alternatives for the corroding element were proposed and a number of parametric studies were conducted in air to understand the behavior of the analog sensor. The sensor was also subjected to wet/dry cycles in salt water to assess the change response as corrosion develops on the sacrificial element. The results of these preliminary tests are discussed in this paper.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Coupled resonant coil sensors with increased interrogation distance

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ABSTRACT

Wireless sensors are being explored for many structural health monitoring applications. Passive wireless sensors have the advantage of not requiring a power source on the sensor, as the energy is provided by the interrogation unit. Inductively coupled coil sensor is a class of passive wireless sensor. They operate by using a resonant coil coupled to a measurand that shifts the sensor coil's resonant frequency. Coupled coil sensors are found in many applications e.g., corrosion detection (Andringa et al. 2005), monitoring water content in civil engineering materials (Ong et al. 2008), strain measurement (Butler et al. 2002), and moisture measurement (Hapster et al. 2002). Most commonly measurement of the sensor coil's resonance is done through perturbations to the interrogator coil's impedance versus frequency. This technique is affected by the response of the interrogator coil. As the interrogation distance increases, the response of the sensor coil decreased rapidly and only the interrogator coil self impedance dominates, which is sensitive to the background environment. Thus this interrogation technique requires a small interrogation distance between the sensor and the interrogator.

In this paper, we present a new interrogation method which makes use of time-domain gating to eliminate the response from the interrogator coil and the exciting signal. This results in a measurement dominated by the response of the sensor coil, enabling detection of the sensor's resonant frequency far more precisely at an extended separation distance. The interrogator first energizes the sensor through inductive coupling between the interrogator coil and the sensor coil. This is the transmit mode. After enough time has elapsed to allow the induced energy in the sensor coil to reach equilibrium, the system switches back to the receive mode where energy is coupled back from the sensor to the interrogator. A delay is introduced when the system switches from the transmit mode to receive mode to eliminate the transient response of the interrogator coil.

Using this method a test sensor coil's resonant peak position was determined within 30 parts per million at a greater interrogation distance (20 cm) than that of the impedance measurement method (15 cm). This accuracy was much higher than some passive wireless sensors employing an impedance measurement technique (e.g., 6 cm interrogation distance with less than 1% measurement error (Ong et al. 2008), 0.6 cm interrogation distance with the peak position within 7740 parts per million (Butler et al. 2002), 8 cm interrogation distance with peak position within 1714 parts per million (Ong et al. 2002)). For a relative humidity sensor coil, this method was able to detect changes in relative humidity with less than 2%.

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Concentration-dependent piezoelectricity and strain sensitivity of ZnO nanoparticle-polymeric thin films

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ABSTRACT

Recently, zinc oxide (ZnO) nanomaterials (such as ZnO nanoparticles and nanowires) have been highly regarded as potential candidates for the design of next-generation piezoelectric nanocomposites. Previous studies have demonstrated that ZnO nanoparticle (NP)-polyelectrolyte (PE) thin films exhibit piezoelectricity and generate electric potential under applied strain. Thus, unlike traditional strain sensors (e.g. strain gages, MEMS strain sensors, piezoresistive materials, among others), ZnO-NP-PE thin films are self-sensing materials that do not require a constant power source for operation. As such, this novel sensor is low cost, of small form factor, and can be easily embedded within various types of engineering structural components.

In this study, a versatile thermal evaporation and annealing procedure is proposed for fabricating piezoelectric zinc oxide-based thin film strain sensors. Here, ZnO NPs are dispersed in poly(sodium 4-styrene sulfonate) (PSS) solutions and mixed with poly(vinyl alcohol) (PVA) for film fabrication. The nanoparticle and PSS concentrations are varied for optimizing thin film piezoelectric/self-sensing performance. Upon film fabrication, the piezoelectricity of 12 unique as-fabricated films (characterized by different weight fractions) is experimentally investigated by epoxy-mounting them onto thin poly(vinyl chloride) cantilevered beams; a traditional metal-foil strain gage is also mounted adjacent to the films for measuring the induced strains during testing. Then, an initial displacement is applied to the free-end of the cantilevered beam and released to facilitate free vibration response of the beam. The potential generated by each film is measured and compared against one another to identify the optimal ZnO concentration that yields the highest bulk film piezoelectricity and self-sensing performance.

Upon conducting the experimental study, first, piezoelectricity is validated for several of the zinc oxide-based films (Fig. 1). The maximum normalized

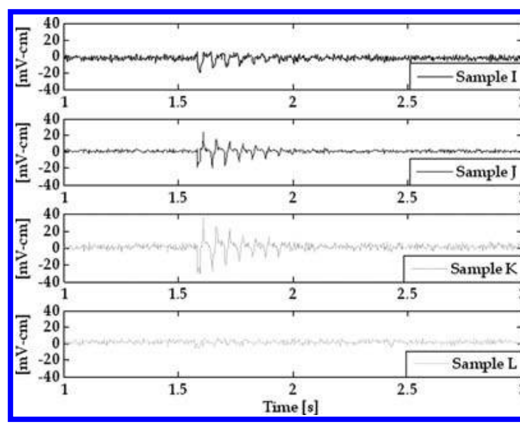

Figure 1. Piezoelectric response of ZnO-PSS-PVA films.

generated voltages have also been validated for films fabricated with a weight fraction of less than 33% to 37%. For films evaporated with lower ZnO weight fractions, piezoelectricity is still observed, but the performance is dramatically lower than those fabricated with ~35% weight fraction zinc oxide NPs. On the other hand, greater ZnO weight fractions resulted in films exhibiting high noise floors and no piezoelectric response to applied strains. It is hypothesized that high concentrations of zinc oxide induce clumping and agglomeration of nanoparticles that diminish their piezoelectric performance.

Second, the piezoelectric thin films' normalized generated voltages are directly compared with measured strains. Specifically, since piezoelectric materials respond to dynamic strain, the voltage time history plots are overlaid with the rate of change of strain. From these results, it can be confirmed that there is good correlation between the normalized generated voltages and the rate of change of applied strain. Preliminary results indicate that the strain rate sensitivity of the proposed nanocomposites varies with the absolute amount of nanoparticles embedded within the thin film.

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Prediction of displacement response of a suspension bridge using FBG strain sensors

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ABSTRACT

Predictions of displacement as an important factor in evaluating the robustness of large structures become the basis to determine the decrease of structural performance and the degree of aging in general. It is however well known that it is not easy to measure the displace response of large structure like suspension bridges due to the absence of measurement techniques despite the importance of measurements in the displacement response. In this study, a method estimating displacement responses is proposed using a mode decomposition technique. This is a method estimating the total displacement response combined with the each displacement response about the major mode of the structure and the quasi-static displacement responses.

Figure 1 shows that the estimated displacement response and the displacement response measured are identical. The outcome of comparison of the biggest amount of deflection of the inferred displacement response and the displacement response measured is seen in Table 1. As shown in Table 1, it was identified that the inferred displacement response by means of mode decomposition technique and the displacement response measured is similar. As shown in the Figure 2 was estimated that there is a similarity between measured displacement response and that with the use of 6 sensors. But when 4 sensors was used, it was observed that there is a big error. Thus when estimating the displacement response of simple span girder, it is expected that the reliable inference of displacement response can be made with the use of 6 censors.

When using mode decomposition technique, it is possible to infer the displacement response while using strain signals at a smaller number of points and to infer the displacement response at all of points. In addition to this, reliable measurement of strain signal in the large structures such as suspension bridge can be made with the use of FBG sensor with which multiple measurement is possible, generating little electric noise.

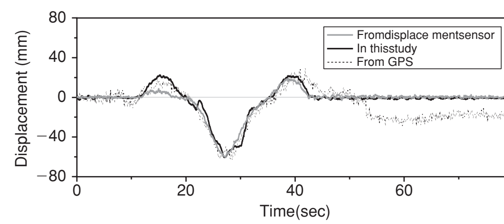

Figure 1. Comparison of estimated displacement response and measured displacement response.

Table 1. Comparison of estimated maximum displacement and measured maximum displacement

	In this study	From GPS	From displacement sensor
LC1	−35.6 mm	−37 mm	−31.98 mm
LC2	−32.5 mm	−50 mm	−34.03 mm
LC3	−60.6 mm	−63 mm	−60.68 mm

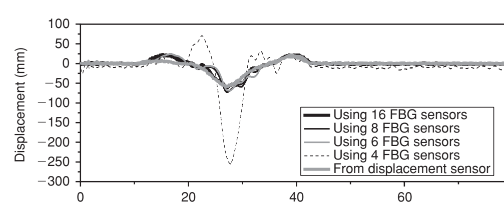

Figure 2. Comparison of estimated displacement responses.

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Structural health monitoring of a cable-stayed bridge using acceleration data via wireless smart sensor network

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ABSTRACT

There have been increasing interests in the wireless smart sensors in the field of structural health monitoring (SHM) of civil infrastructures due to its wireless communication as well as embedded computation capability. The SHM system based on the wireless smart sensor network (WSSN) may alter the conventional tethered SHM system which is suffered from the costly cabling work between sensor and data repository and inundation of raw data which may not be informative. Based on the advantages of wireless sensors, several researches to monitor long-span bridge structures using wireless sensors have been reported (Weng *et al*. 2008, Pakzad 2009).

The test-bed of the SHM system using state-of-the-art WSSN technology has been constructed on a cable-stayed bridge (The 2nd Jindo Bridge shown in figure 1) in Korea (Cho *et al*. 2009). A total of 70 wireless smart sensors are installed with high spatial density on the bridge, facilitating measurements of 3-axes of acceleration underneath of the deck, on two pylons, and on the cables. The measurement has been carried out in two sub-networks during the past 4 months using an autonomous monitoring system. The test-bed is to provide important insight into the opportunities and challenges for WSSN technology on long-term monitoring of large civil infrastructures. This effort is a part of trilateral collaboration between Korea (KAIST), the US (University of Illinois at Urbana-Champaign), and Japan (University of Tokyo). The details of constructed test-bed can be referred to Cho *et al.* (2009).

This study focuses on the SHM technique using the collected acceleration data at the test-bed. As prior works, a finite element (FE) model is constructed based on an in-depth study of the detailed drawings, and the acceleration data from the existing wired monitoring system is analyzed to be used as a reference data of WSSN performance. The acceleration data collected from the current WSSN are analyzed offline.

Figure 1. The 1st (right) and 2nd (left) Jindo Bridges.

Output-only modal identification (ID) is carried out to extract the modal properties of the bridge from the ambient acceleration data of the deck and the pylons. The extracted modal properties from the output-only modal ID method are validated by comparing with those from the wired monitoring system and the FE analysis. Tension forces are also estimated on 10 cables with large tension forces using 3-axis ambient acceleration data measured on the cables by a vibration method. The estimated tension forces are compared with the tension forces obtained from the previous regular inspection. Based on the results, discussions are made on the monitoring performance using the WSSN.

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Smart wireless tension force monitoring system for stay cables

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ABSTRACT

Tension force of a bridge cable is a structural parameter of interest for cable-stayed bridges in which the cable is a primary load carrying member. It should be monitored regularly during its service life to ensure the bridge is in safe operating condition without overloading. Though tension measurement devices, such as load cells, hydraulic jacks, and strain gauges, are available to measure tension forces directly on cables, they are rarely used due to their high-cost and tough maintenance. Vibration methods which estimate the cable tension force using modal properties and geometries of the cable are now widely used because they require less invasive instrumentation and are easier to implement cost-effectively.

Meanwhile, there has been an increasing interest in the wireless sensor technology in the structural health monitoring field. Low-cost wireless sensors without any cabling work are the enabler of realizing the structural health monitoring system economically and effectively on a large structure. The wireless sensors also have a big benefit of embedded computation, which can infer some important information from the measured data by itself (Lynch & Loh 2006, Lynch 2007).

In this study, a smart wireless tension force estimation system is developed by embedding a vibration method for cable tension force estimation into a wireless sensor. Developed system is composed of two parts, low-cost hardware and automated software. The low-cost hardware is composed of a wireless sensing unit (Wang *et al*. 2005), a signal conditioning board (Lynch *et al*. 2006), and a MEMS accelerometer. An automated peak-picking algorithm to extract natural frequencies of a cable from response spectrum without human intervention is developed and embedded into the wireless sensors in conjunction with a vibration method. To extract the accurate natural frequencies of the cable with the limited data storage of the developed system, Welch's method is additionally coded to be embedded into the developed system.

The system is validated from the laboratory test using scale-down cable model with a variety of cable tension and sag conditions. 18 test cases were carried out on the cable with various tension forces with corresponding sag conditions. The results of the experimental study are summarized as follows: (1) Consistent results can be obtained for the first three natural frequencies regardless of the sensor location along the cable using the proposed peak-picking algorithm. (2) The cable tension forces are estimated with good agreements with those measured using strain gauges for all the cases regardless of tension forces and sagging conditions, which indicates the applicability of the present system to actual cable-supported bridges. (3) Embedding Welch's method is a good solution for more accurate estimation of the natural frequencies by removing the nonstationarity of short-duration signals, which is a limit of the memory-constrained wireless sensors.

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An active sensor placement optimization strategy using data-driven Bayesian experimental design

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ABSTRACT

Active sensing for structural health monitoring is the process of imparting energy into a structure, detecting the resulting structural response, and extracting features from the response that correlate with the structure's state of health (Farrar and Worden 2007). A commonly-exploited implementation of this approach is the use of piezoelectric transducers to launch and detect ultrasonic elastic waves to probe for defects (Giurgiutiu et al. 2002).

Given typically limited actuator/sensor budgets, it is very desirable to place these transducers on the structure in a way that optimizes a certain objective. Using a general formulation of Bayesian experimental design, we develop a global optimality criterion within a detection theory framework based on a cost-weighted sum of the expected occurrence of Type I (false positive) and Type II (false negative) errors.

Our approach generalizes to any derived feature set under some simple assumptions, and it includes the possibility of non-uniform probabilities of damage occurrence on the structure. The optimization configuration space is searched by a greedy algorithm with a fitness function that evaluates the Baye's risk.

The method is unique in that it makes use of real-world cost specifications, which allows the optimal selection of both the transducer locations and the number of transducers to implement.

The target structure for this study was the Alamosa Canyon Bridge in southern New Mexico, USA. This is a decommissioned steel beam-reinforced concrete bridge which runs along-side the current Interstate-40 bridge. Each of the seven spans of the bridge contains ten steel connections which join the five primary longitudinal beams with the eight lateral beams using 14, 44.5 mm bolts spaced 760 mm apart. The objective was to determine the optimal placement of piezoelectric active transducers at each of these joints in order to detect the loss of preload on any of the fourteen bolts.

Past studies have determined optimal arrangements through development of numerical or analytical models and simulation of sensor network performance (Flynn & Todd 2009). This approach is necessary in design situations where there are too many damage modes to observe experimentally and/or in situations where the structure is too large or complex to instrument densely. In the application at hand however, it was possible to take advantage of the relatively low number of possible damage states as well as the repeated patterns of joints in the full structure. This enabled the implementation of data-driven models derived from experiment with direct observation of all possible damage modes.

We considered three optimization scenarios for this study. The first two involve variations on the form of the detection loss function with the exclusion of instrumentation cost, while the third incorporates a per-sensor cost function. Inconsistencies in the optimal sets of transducers among the three scenarios demonstrated the significant impact the relative costs have on the optimal transducer arrangement.

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Investigation of the control performance of the smart passive system based on MR damper using hybrid simulation

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ABSTRACT

This paper evaluates the control performance of the smart passive system employing a magnetorheological (MR) damper and an electromagnetic induction (EMI) device for suppressing the vibration of a stay cable. An MR damper is one of the most promising semi-active control devices (Dyke et al., 1996), and an EMI device which consists of permanent magnets and a solenoid coil can be considered as a power supply for an MR damper by generating the electricity from the vibration of structures according to the Faraday's law (Cho et al., 2005). To validate the effectiveness of the proposed system, a hybrid simulation which is combination of the experimental test part of the smart passive system (i.e., an MR damper and EMI device) and the numerical analysis part of a stay cable structure has been performed with the white noise and generated time history of wind load (Nakashima et al., 1992). Figure 1 shows the concept and flow of the hybrid simulation. The displacement at the damper location is calculated numerically, and it is used as an input command for the shaking table test with the prototype MR damper and EMI device, and then the damper force which is measured, is put into the numerical simulation. As shown in Figure 2, the responses of the controlled case are much smaller than those of the uncontrolled case. It is demonstrated from the results that the proposed system could be the promising control system for suppressing the excessive vibration of stay cables.

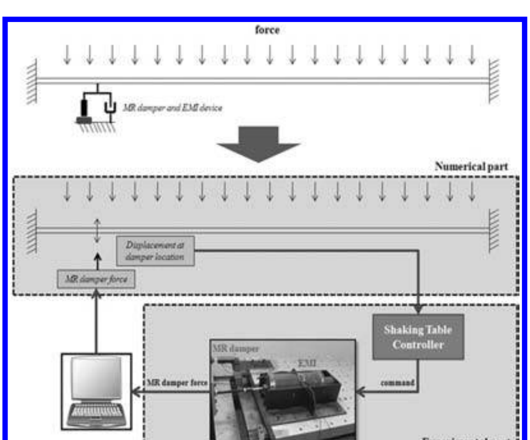

Figure 1. Concept diagram of the hybrid simulation.

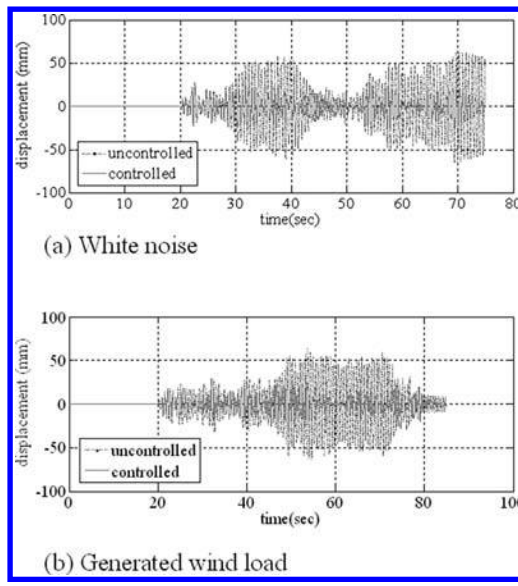

Figure 2. Time histories of the displacement at the mid-point of the cable.

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Autonomous structural health monitoring using wireless smart sensors on a cable-stayed bridge

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ABSTRACT

Structural health monitoring for long-span bridges using wireless smart sensor network (WSSN) has drawn significant attention. The benefits of the WSSN are low-cost, time efficiency, and post-processing-free operation. However, some drawbacks have remained with respect to implementation of WSSNs on long-span bridges, including power consumption, manual communication with the sensor network, and limitations on the hardware. In this paper, the solutions for these drawbacks are provided by an energy efficient autonomous monitoring strategy, combined with newly developed sensor boards. The elements of the autonomous monitoring strategy consist of energy efficient sleeping mode and automatic threshold-based sensing. A tri-axial acceleration board with temperature and humidity measurements and a wind speed sensor board are utilized. To demonstrate the efficacy of this hardware/software solution, 70 Imote2s loaded with the developed software package have been deployed on the Jindo Bridge in South Korea. For further energy efficiency, several of these nodes are solar powered.

Figure 1. Jindo Bridges.

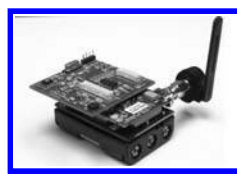

Figure 3. Smart sensor units.

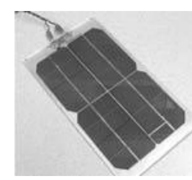

Figure 4. Solar panel.

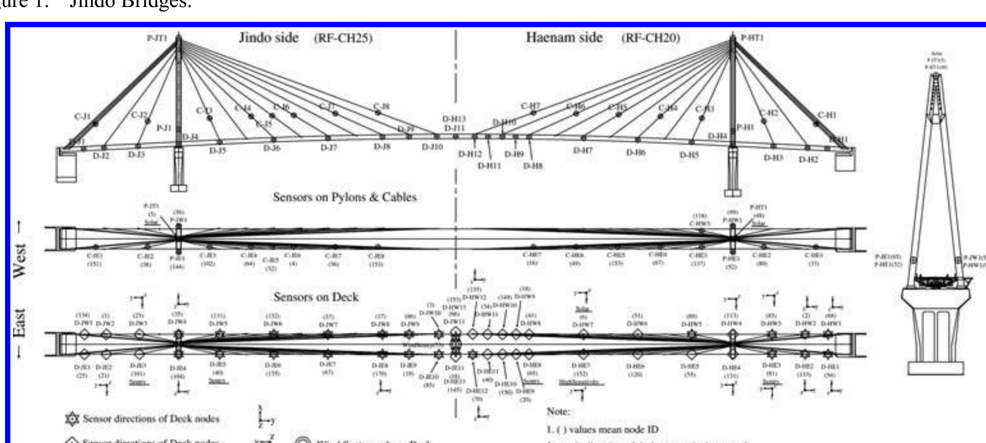

Figure 2. Sensor locations.

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Evaluation of ultimate performance of the reinforced concrete T-girder bridge using optical fiber sensors

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ABSTRACT

Electric resistance type sensor has been widely used for field measurement of civil structure such as bridge. This type of sensor shows some disadvantages for long term application, such as decaying, damage by lightning, noise from electromagnetic field, lack of workability etc. It is also known that this type of sensor, even in short term application, gives low credibility measurement due to the effects of electromagnetic wave when it is used for directly exposed structures as well as excessive time and expenses are consumed for complicated cabling. Studies for utilizing optical fiber sensor to compensate shortcomings of existing sensors are being carried out actively.

Diagnostic Load Test(DLT) that estimates load-carrying capacity by using response and measurement from static-dynamic test of vehicle load and Proof Load Test(PLT) that evaluates load-carrying capacity directly from applying large load on the structure until main members show nonlinear behavior are general methods used to get load-carrying capacity of real structure. In this study, both DLT and PLT are performed for short span RC-T type girder bridge which is not in use anymore. Fiber Bragg Grating sensor (FBG sensor) is used together with electric resistance type sensor for DLT to investigate the usage of optical fiber sensor for bridge load-carrying capacity evaluation.

FBG sensor can replace traditional electric resistance type sensor for the measurement of civil structures such as bridges at site due to the strong advantage of being free of environmental conditions, especially electromagnetic wave. Furthermore, FBG sensor can reduce measurement error since it could measure much clearer response. The result of load-carrying capacity test of bridge can be greatly differed due to measurement errors and/or modeling errors of FE model. Hence, more reliable load-carrying capacity analysis can be possible by minimizing measurement errors using FBG sensor at site.

Figure 1. Packaged FBG sensor.

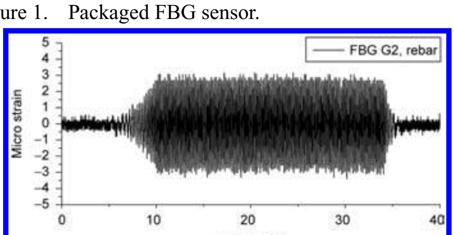

Figure 2. Response of exciting at 6.0 Hz (FBG sensor).

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Statistical damage assessment based on the extreme value distribution using vibration responses

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ABSTRACT

A baseline statistical distribution is often required using the dynamic response data of an undamaged structure so that the existence of damage can be confirmed in the context of statistical damage assessment. Generally, damage-sensitive dynamic response data of a structure manifest themselves near the tail of a baseline statistical distribution. In this regard, some researchers have paid attention to extreme value distribution for modeling the tail of a baseline distribution. However, few researches have been conducted to theoretically understand the extreme value distribution from a perspective of statistical damage assessment. This study investigates the asymptotic convergence of domain of attraction through parameter estimation of the generalized extreme value distribution (GEV). The outlier threshold value error and the outlier occurrence probability error are proposed to quantify discrepancy between the theoretical CDF and the estimated GEV with respect to the sample size [Figure 1]. The effect of the sample size on the false positive alarms in statistical damage assessment is quantitatively investigated as well. Finally, the validity of the proposed method is demonstrated through numerically simulated ambient acceleration responses of a two span continuous truss bridge in Figure 2.

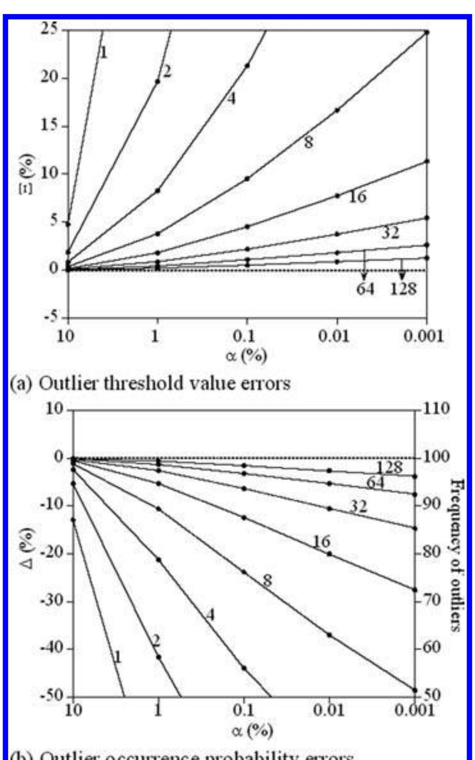

Figure 1. Quantification of discrepancy between the theoretical CDF of the Pareto distribution and the estimated GEV with respect to the sample size.

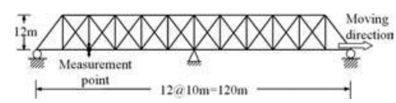

Figure 2. The geometry and the measurement point for acceleration data of a two-span continuous truss bridge with moving vehicle load.

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Prestress-force monitoring using impedance-based smart sensor nodes in PSC girder bridges

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ABSTRACT

In this study, a technique using wireless impedance sensor nodes is proposed to monitor Prestress-force in PSC girder bridges. In order to achieve the goal, the following approaches are implemented.

Firstly, a wireless impedance sensor node is designed for automated and cost-efficient Prestress-loss monitoring as shown in Fig. 1. The wireless impedance sensor node consists of the power supply, microcontroller, AD5933, and wireless radio.

Secondly, an impedance-based algorithm is embedded in the wireless impedance sensor node for autonomous structural health monitoring based on the decentralized wireless sensor network. The overall procedure of the operation scheme using root-mean-square-deviation (RMSD), correlation coefficient (CC), and peak-frequency shift is summarized as follows:

(1) Broadcast a start command for measuring impedance signatures by the remote control server;
(2) Acquire impedance signatures $\{Z_i(\omega), i = 1, \ldots, n\}$ and store in individual wireless impedance sensor node (WISN);
(3) Broadcast impedance signatures by the captain sensor node (i.e., WISN i) to all other wireless impedance sensor nodes;
(4) Compute impedance features $\{IF_i,\ i = 1, \ldots, n\}$ at individual wireless impedance sensor nodes;
(5) Transmit the impedance features extracted at individual wireless impedance sensor nodes to the captain wireless impedance sensor node (WISN i).

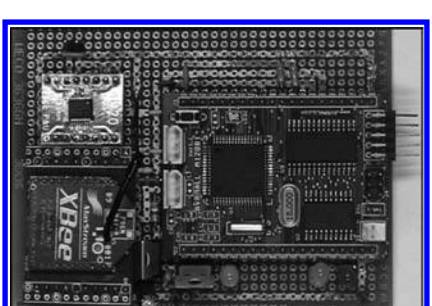
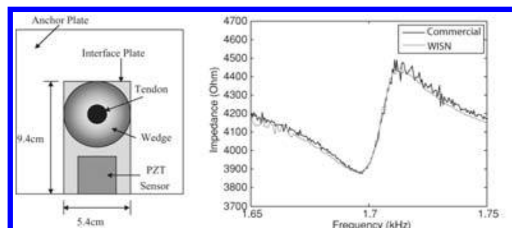


Figure 1. Prototype of wireless impedance sensor node.



(a) Interface plate (b) Measured impedance signatures

Figure 2. Measured impedance signatures on interface plate.

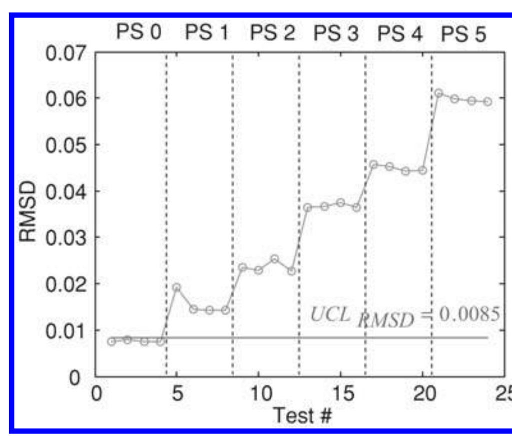


Figure 3. Prestress motoring results using RMSD by wireless impedance sensor node.

Thirdly, a Prestress-loss monitoring technique using an interface plate is proposed to overcome limitations of the wireless impedance sensor node such as measureable frequency ranges with narrow band as shown in Fig. 2.

Finally, the feasibility and applicability of the proposed technique were evaluated in a lab-scaled PSC girder model for which several prestress-loss scenarios are experimentally monitored by the wireless impedance sensor node. And though embedded SHM algorithm for prestree-loss of PSC girder, the wireless impedance sensor node successfully monitored prestress-loss in tendon as shown in Fig. 3.

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Tension monitoring of a prestressing strand for concrete bridge using in-tendon FBG sensors

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ABSTRACT

Since the first post-tensioned concrete bridge was built in Aue, Saxony, Germany in 1936, the concept has widely been used all over the world, and is still up to date. However, it was realized that the post-tension system is subject to long-term risk such as corrosion of tendon caused by ingress of water and chloride irons into partially grouted ducts. The tensile force of a prestressing strand can be varying due to various reasons of loss including instantaneous losses such as elastic shortening, friction, and anchorage set during transferring the prestressing forces, and time dependent losses due to steel relaxation, concrete creep and shrinkage that occur after transfer and during the life of the structural member. Accordingly, measurement of tensile force of the tendon becomes very important for long-term maintenance of the bridge as well as design purpose.

In this study, we propose a novel method for evaluating tension force of a prestressing 7-wire strand using in-tendon FBG sensor (Fig. 1). For the sake of demonstrating effectiveness of the proposed idea, we came up with 1.0m, 8.0m and 64.0m long tendon prototypes in which FBG sensors are embedded into the tendon, and their accuracy and effectiveness are shown with test results (Fig. 2).

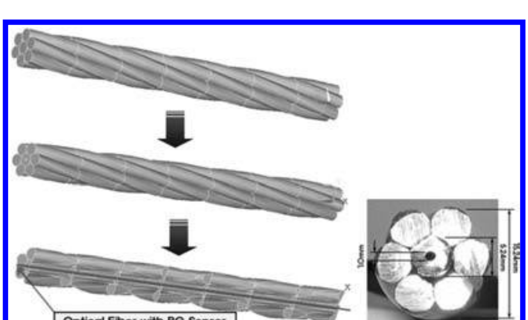

Figure 1. Concept of in-tendon FBG sensor.

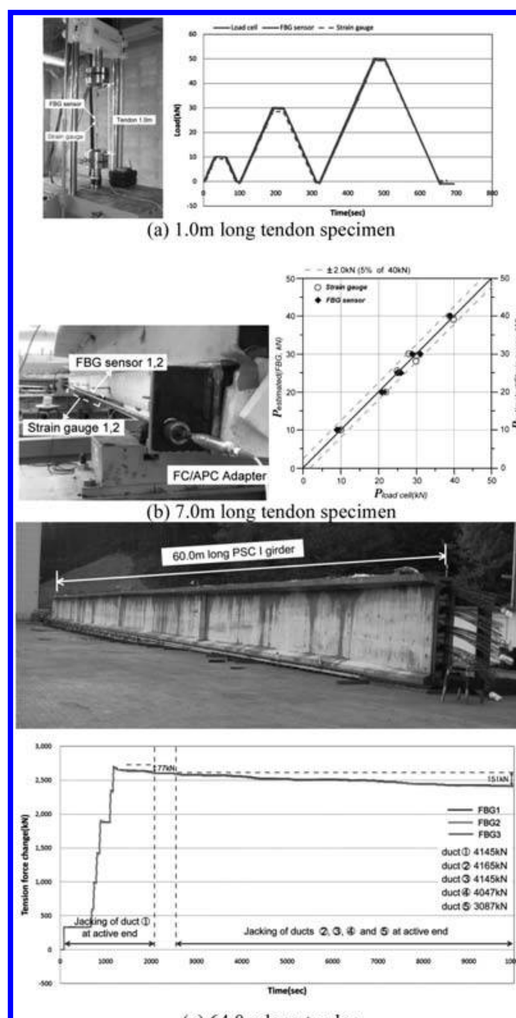

Figure 2. Experimental results of in-tendon FBG sensors for 1.0 m, 7.0 m and 64.0 m long tendons.

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Evaluation of the characteristics of local erosion of fine-grained soils in the west coast area of Korea

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ABSTRACT

As a recent global trend, the characteristics of soil erosion as the resistance factor against scouring are quantified and considered in the estimation of scour depths in fine-grained soils. As part of such efforts, the characteristics of local erosion of fine-grained soils in the West Coast area – where several sea-crossing long-span bridges have been planned and constructed in recent years – are analyzed through scour rate experiments. Three sites – Incheon Bridge, Choji Bridge, and Hwankyung Bridge – were finally selected for this study; scour rate tests were performed using 29 undisturbed soil samples from the sites.

According to the results of this study on the characteristics of erosion of fine-grained soils in the West Coast area, the erosion resistance ability (critical shear stress) of most soils except a few samples tended to be generally proportional to the undrained shear strength values of the soils. In addition, fine-grained soils containing relatively more cohesive clay recorded lower scour rates and far larger critical shear stresses at a steady water velocity than fine-grained soils without cohesive clay; hence the higher resistance to scour.

The average critical shear stress as the erosion resistance ability of soils in the target sites was 1.15 N/m^2 in Incheon Bridge, 2.51 N/m^2 in Ganghwa Choji Bridge, and 12.42 N/m^2 in Hwankyung Bridge, showing differences in keeping with the soil characteristics. Similarly, the average critical water velocity stood at 0.57 m/s in Incheon Bridge, 0.95 m/s in Ganghwa Choji Bridge, and 2.45 m/s in Hwankyung Bridge. In addition, the quantitative scour rate-shear stress curve was determined for each target site through tests, and a chart representing the erosion characteristics for each site was presented. These systemized data are provided as basic data for calculating more accurate scour depths and developing basic designs for the construction of long-span bridges in the West Coast area.

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Impedance-based structural health monitoring using neural networks for autonomous frequency range selection

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ABSTRACT

The impedance-based structural health monitoring (SHM) method comes to the forefront in the SHM community due to its practical potential for real applications. Regarding the impedance-based SHM method, the selection of the optimal frequency ranges plays an important role in improving the sensitivity of damage detection since improper selection of the frequency range might lead to erroneous damage detection results and provide false positive damage alarms. To tackle this issue, this paper proposes an innovative technique to autonomously select damage-sensitive frequency ranges using the artificial neural networks (ANN). At first, the impedance signals are obtained in a wide frequency band, and the signals are split into multiple sub-ranges of the wide frequency band. Then, the pre-defined damage ind0 is evaluated for each sub-range by comparing the impedance signals measured between the intact and the concurrent cases. Herein, the cross-correlation coefficients (CC) are used as the pre-defined damage index. The ANN is constructed using the CC values for multiple frequency ranges as multi-inputs and the real damage severity as a single-output. The ANN is trained using a number of impedance signals for various damage scenarios pre-selected, so that sub-sequent damage estimations may be carried out by selecting the damage-sensitive frequency ranges autonomously (Figure 1).

Experimental investigations were performed to validate the performance of the proposed technique for both automatically determining the frequency range and simultaneously detecting loose bolts and artificial crack damages inflicted on real building and bridge structures. It is found that the proposed approach using the autonomously selected damage-sensitive frequency ranges can be effectively utilized to evaluate the damage severity for various damage cases in real structures.

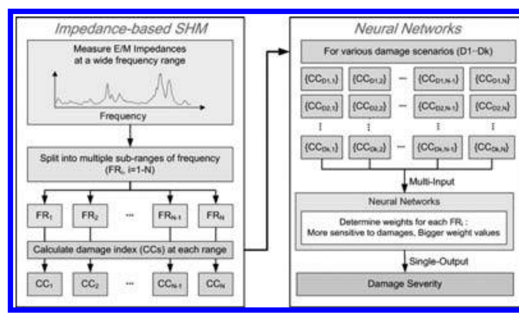

Figure 1. Scheme for autonomously selecting the most sensitive frequency range using artificial neural networks.

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Output only modal identification and damage detection of bridge-structures using time frequency and wavelet techniques

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ABSTRACT

The primary objective of this paper is to develop output only modal identification and structural damage detection. Identification of multidegree of freedom (MDOF) linear time invariant (LTI) and linear time variant (LTV-due to damage) systems based on Time-frequency (TF) techniques–such as short-time Fourier transform (STFT), empirical mode decomposition (EMD), and wavelets—is proposed. STFT, EMD and wavelet methods are proposed. In addition a Hilbert transform (HT) approach to determine frequency and damping is also presented. In this paper STFT, EMD, HT and wavelet techniques are developed for decomposition of free vibration response of MDOF systems into their modal components. Once the modal components are obtained, each one is processed using Hilbert transform to obtain the modal frequency and damping ratios. In addition the ratio of modal components at different degrees of freedom facilitate determination of mode shape. In cases with output only modal identification using ambient/ random response the random decrement technique is used to obtain free vibration response.

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Safety network system integration for bridge structures in Korea

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ABSTRACT

In general, structures in service gradually lose their original performance-level over time due to initial defects in design and construction, or exposure to unfavorable external conditions such as repeated loading or deteriorating environment, and in extreme cases, may collapse. So, in order to maintain the serviceability of structures at optimal level, advanced structure measuring system which can inform optimal time point and method of maintenance is required in addition to accurate prediction of residual life of the structure by periodic inspection. In Korea, the large-scale project for the safety network integration for long-term smart monitoring systems for bridge structures have been performed since 2006, and this is the 4th year of the project.

In this system, various structure types such as bridge, tunnels and cut slopes were considered with an emphasis on safety. The purpose of this system is to integrate the operation centers for these structure types in order to control and evaluate the structure's real-time safety level through measured behavioral data. For the bridge structure's safety network integration, four test-bed bridges are integrated by mainly using the FBG(Fiber Bragg Grating) sensor system. For the effective monitoring the state of the bridge safety level in the safety network operation centers, various analysis techniques for the bridge safety level evaluation are developed and used. In this paper, the explanations on various scenes from the evaluation techniques and the state of the art of the safety network system for the bridges in Korea are suggested.

The test-bed system implemented in this study successfully configured link-up items from the existing measurement system and conveyed realtime and history data, showing its capability to be the foundation of further measurement systems integration.

The present research has completed construction and supplementation of the 1st test-bed system and construction of the 2nd test-bed system. The integrated safety management network monitoring system will be complemented in 2011 by adding two more bridges.

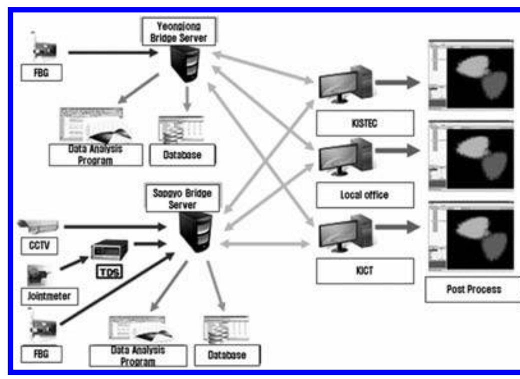

Figure 1. Schematic diagram of remote data transmission system.

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Damping properties identified from wind-induced vibration measurements of a suspension bridge

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ABSTRACT

In a suspension bridge, the damping ratio as an index suppressing bridge vibration could be considered as one of the important dynamic characteristics. But, in general, estimation of damping ratios can be a laborious task when the existing bridges are under ambient vibration conditions. In this paper, the damping ratios on Sorok Bridge, a suspension bridge in Korea, were estimated by using two different damping ratio estimation methods that can be properly applied under ambient vibration conditions. The first is using Hilbert-Huang Transform and the other is using extended Kalman filter as a nonlinear system identification techniques. When ambient vibration signals are measured from a bridge, it is not simple to separate extract viscous, Coulomb friction and aerodynamic damping properties from apparent damping ratios directly using the measured signals. But, if the viscous and Coulomb friction damping properties are classified as structural damping, the apparent damping ratios can be separated into structural and aerodynamic damping properties.

Half-power band width method and logarithmic decrement are well known for estimating damping ratio as an important parameter of bridges. However, these methods could be hard to apply for ambient vibration signals from existing bridges. In this study, Hilbert-Huang transform(HHT) and extended Kalman filter(EKF) was used for estimating damping ratio.

In this study, Sorok Bridge was selected as an estimation object. Sorok Bridge is a suspension bridge that has main span of 250 meters, and side span of 110 meters in South Korea. Although Sorok Bridge was completed in 2008, it had been closed to traffic because of incompletion of connecting road. After opening on March 2009, the bridge was a little traffic because of the same reason until the end of 2009. These conditions made a good subject to investigate the relation between wind velocity, acceleration and damping ratio of existing bridge.

At the result as comparing Figure 1 and 2, results by EKF could be shown more stable than result of Hilbert-Huang transform.

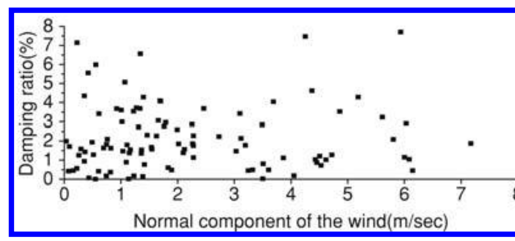

Figure 1. Damping ratio estimation results of Hilbert-Huang transform.

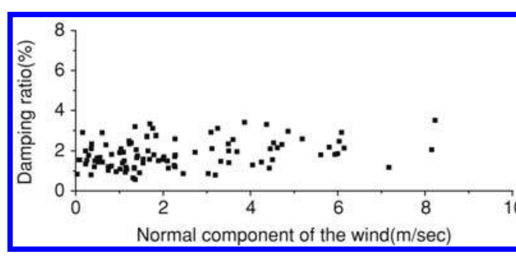

Figure 2. Results of estimation by extended Kalman filter.

And, Distinguishing structural damping and aerodynamic damping from appearance damping was hard only using appearance damping with wind. It could be possible by adding results of exciting test with each other RMQ value of acceleration on center of main span.

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Debonding condition monitoring of a CFRP laminated concrete beam using piezoelectric impedance sensor nodes

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ABSTRACT

Recently, carbon fiber reinforced polymer (CFRP) laminated concrete structures are being very widely used in various engineering fields because of their several advantages. But they always have the riskiness of structural collapse accidents initiated from the debonding conditions that might occur between CFRP and concrete surface (Kim et al. 2008). This paper employs an electro-mechanical impedance-based wireless structural health monitoring (SHM) technique applying PZT ceramic patches to identify the debonding condition of CFRP laminated reinforced concrete beam (Park et al. 2003; Park et al. 2009; Koo et al. 2008). In the experimental study, CFRP-reinforced concrete specimens were fabricated and the impedance signals were measured from the wireless impedance sensor node according to different debonding conditions inflicted between concrete and CFRP as displayed in Figure 1. To investigate quantitatively the changes of the impedance measured at the PZT patches due to the debonding conditions, cross correlation (CC)-based data analysis was conducted and its results are showed in Table 1. From this study, it was verified that the impedance-based wireless SHM technique can be effectively used for debonding monitoring of CFRP laminated concrete structures.

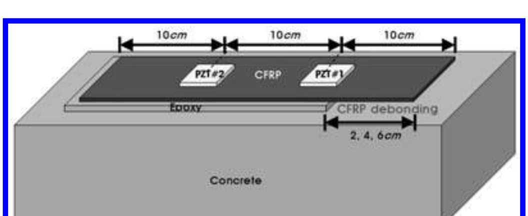

Figure 1. Test specimen with CFRP debonding condition.

Table 1. Damage index: 1-CC (Cross Correlation Coefficient).

PZT	1-CC (Cross Correlation Coefficient)	
Condition	PZT#1	PZT#2
Perfect bonding	0	0
2 cm debonding	0.15	0.03
4 cm debonding	0.18	0.05
6 cm debonding	0.23	0.07

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Development of a wireless power and data transmission system using laser and optoelectronic devices for guided wave-based structural health monitoring

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ABSTRACT

In recent years, guided wave based structural health monitoring (SHM) techniques have attracted much attentions, because they are not only sensitive to small defects but also capable to propagate over a long distance in plate and pipe like structures. A number of studies have demonstrated the potential of guided wave based SHM.

These guided waves in a structure can be generated and sensed by a variety of techniques (Su et al. 2006). The ultimate goal of our research is to develop an optical system for guided wave generation and sensing as in Figure 1. This paper mainly focuses on the excitation aspect of the overall system. The proposed wireless technique transmits power necessary for PZT excitation using laser and optoelectronic devices. First, a desired waveform is generated and the intensity of the laser source is modulated accordingly using an electro-optic modulator (EOM). Next, the modulated laser is wirelessly transmitted to a photodiode connected to a PZT. Then, the photodiode converts the transmitted light into an electric signal and excites the PZT to generate guided waves on the structure where the PZT is attached to. Finally, the corresponding response from the sensing PZT is measured. The feasibility of the proposed method for wireless guided wave generation has been experimentally demonstrated. (Figure 2).

The results confirm that the laser-based guided wave generation technique exhibits the reasonable performance by comparing the conventional wiring guided wave generation method. Because power is remotely transmitted to the PZT transducer, no complex electronic components are necessary at the PZT node, making it possible to develop a self-contained, rugged, and non-intrusive PZT node. The transducer itself can be entirely passive and consequently will have a long lifetime when it has rugged packaging. In addition, any arbitrary waveform can be generated by a laser using an electro-optic modulator (EOM). This wireless power transmission scheme also can be expanded to transmit power through optical fibers and generate guided waves (Lee et al. 2009). A further research is under way to use other optoelectronic devices, such as laser diodes and laser vibrometers for wireless guided wave sensing and improve the aiming capability of the laser source.

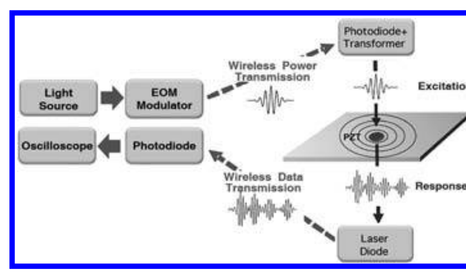

Figure 1. Overall Schematics of the optic-based wireless power and data transmission system.

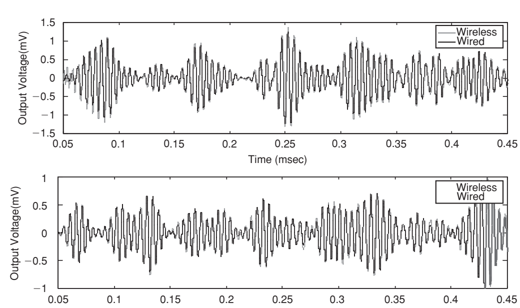

Figure 2. Comparison of the guided waves generated by wired and wireless systems.

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Development of structural health monitoring systems for railroad bridge testbeds

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ABSTRACT

There has been a demand to monitor the health of infra structures, in order to ensure safety and serviceability to the community. Structural health monitoring (SHM) is a methodology to monitor the performance of a structure and identify incipient failure conditions, which helps in improving the safety and life span of the structure. The SHM system often offers an opportunity to reduce the cost for the maintenance, repair, and retrofit throughout the life-cycle of a structure.

During the past two decades, the SHM systems have been widely studied and implemented on many infrastructures in the world (Barrish et al. 2000, Sumitro et al. 2001, Masri et al. 2004). However, in the conventional structural monitoring systems, the data acquisition systems were transported frequently to the monitoring site along with skilled personnel for acquiring data from the structure (Sridhar et al. 2009). In addition, each SHM system has been operated and managed independently by its respective management agencies (Kim et al. 2009). Not only those cause much time, cost, and efforts for monitoring and assessment a structure, but also make difficult to cooperate with other related SHM systems.

Recently a challenging project has been carried out for construction of a national network for safety management and monitoring of civil infrastructures in Korea (KISTEC 2007). The main objective of this project is to establish real-time SHM systems for different types of infrastructures, such as bridges, cut slops, tunnels, embankments, and playgrounds, and integrate each SHM system through on-line networks. Thus the behavior of each infrastructure could be monitored at an integrated monitoring center, which leads effective maintenance and management of national infrastructures. Additionally damages or abnormalities in structural behavior can be observed at an early stage and early alarm system will be operated to managers and users of the structure.

As a part of the project, SHM systems have been established on railroad bridges. This paper presents the current status of railroad bridge health monitoring testbeds. Emerging sensors and monitoring technologies are under investigation. They are local damage detection by monitoring electro-mechanical impedances using piezoelectric sensors and structural strains using optical fiber sensors; vibration-based global monitoring using accelerometers, and temperatures. All of measured data are transmitted to the integrated control center, where real-time measurements are provided under train-transit and environmental loadings. Overall schematics of national network systems are introduced and long-term behaviors of a railroad bridge testbed are investigated using measured data and signal processing results.

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Development of a benchmark laboratory structure for finite-element model updating

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ABSTRACT

Finite element models often do not represent the actual physical characteristics of existing structures. There are many factors that affect the accuracy of the numerical model, these include but are not limited to construction errors such as inappropriate boundary conditions, unplanned loads on the structure, and material imperfections. The objective of model updating is to adjust the numerical model of the structure so that the model predictions are in agreement with the test results. Implicit in many model-updating methods is that the test results are a "true" representation of the actual structure.

Recently, a benchmark structure in a two-dimensional grid form was developed for general bridge health monitoring at the University of Central Florida (Caicedo *et al.* 2006; Catbas *et al.* 2006). However, there has been no benchmark problem that is devoted to assess various modeling techniques and updating algorithms with realistic three-dimensional configurations.

Thus, this paper presents a research initiative toward a well-defined benchmark model updating problem so that participants can implement their updating methodologies and compare various methodologies on a common test-bed. An 8-bay steel truss structure (shown in Figure 1) is designed and constructed to serve as the benchmark test-bed to evaluate FE model updating strategies. Most of discrepancies from real-life structures come from neglecting variable joint stiffness at connections, idealizing support boundary conditions, and overlooking any unknown priori structural flaws or erection errors. Therefore, the structure has been designed with unique features that can simulate these often-overlooked structural characteristics and abnormalities in structural integrity. There are provisions in the model to simulate realistic structural changes due to damage, degradation, erection and fabrication errors, etc.

One of the highlighted design features is its reconfigurability of structural types although it is in limited extent. In addition to that, its capabilities to simulate various physical structural modifications are deemed the most advantageous in assessing modeling techniques and updating algorithms. Finite element models that are close to the physical structure have been suggested with introduction to model parameters. Experimental impact testing results showed that the natural frequencies from FE model are fitting well with experimental results. To disseminate this benchmark structure, sponsorship from the ASCE (American Society of Civil Engineers) EMI (Engineering Mechanics Institute) committees (e.g. dynamics committee and structural health monitoring and control committee) will be sought after.

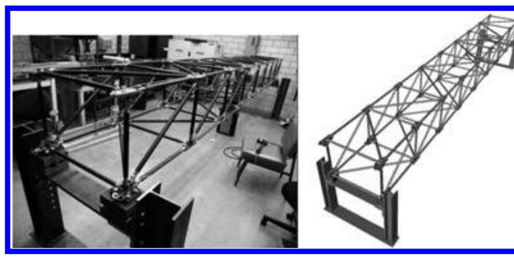

Figure 1. Final 3D Truss Structure and Virtual Reality Model at the University of Akron.

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Variation of eigen-properties of a PSC bridge due to prestressing force

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ABSTRACT

A finite element formulation for the undamped dynamic motion of a beam-tendon system is derived based on the principle of virtual work. The proposed dynamic equation of motion has been verified through a comparison of the analytical results with those obtained from the results of laboratory experiments available in the literature, which shows significant variation in the natural frequencies due to the prestressing force. The proposed algorithm has been also applied to the analysis of actual PSC girder bridges with prestressing.

To compute the first natural frequency in terms of the prestress directly, a normalized equation of Equation 1 can be obtained. In the equation, we can observe that the variation of the first natural frequency due to the prestress is a function of the relative ratio of the prestress P and Euler's critical load P_{cr} of a simply supported column.

$$\frac{(\omega_1)_p}{(\omega_1)_0} = \frac{(f_1)_p}{(f_1)_0} = \sqrt{\frac{\left(\frac{\pi}{L}\right)^4 \frac{EI}{m} + \alpha\left(\frac{\pi}{L}\right)^2 \frac{P}{m}}{\left(\frac{\pi}{L}\right)^4 \frac{EI}{m}}} = \sqrt{1+\alpha\frac{P}{P_{cr}}} \qquad (1)$$

Figure 1 shows the variation of the normalized ratio (f_p/f_0) of the first natural frequency with the increase of the non-dimensional axial force indicator (P/P_{cr}).

To investigate the effects of presterssing force on PSC railway girder bridges, the proposed model has been applied.

Table 1 summarizes the computed critical axial load for each girder type and the natural frequencies computed from the proposed finite element model. The effect of prestressing force on the first natural frequency is within 1.5% for all girder types when Equation 1 is applied.

When the method was applied to actual PSC girder bridges, the increase of the first natural frequency due to the prestressing force was less than 1.5% among the tested cases. This slight increase in the natural frequency is attributed to the amount of the prestressing force being negligibly small relative to the critical axial load of a typical PSC girder. Therefore, it can be concluded that it may not be necessary to consider the effects of the prestressing force on the natural frequencies of actual PSC girder bridges.

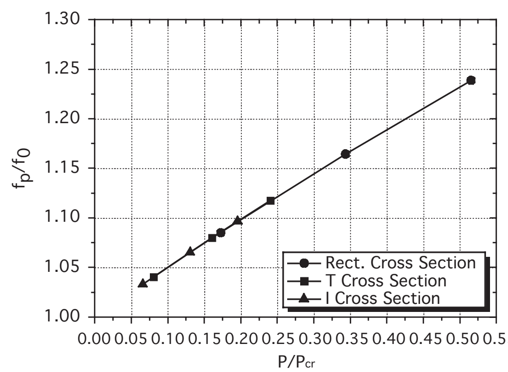

Figure 1. Variation of f_p/f_0 with P/P_{cr} from the proposed model.

Table 1. Natural frequencies by Eq. 1 and proposed model.

Section	A-20	A-23	A-25	B-25
P/P_{cr}	0.0156	0.0283	0.0233	0.0259
f_p/f_0 (Equation 1)	1.008	1.014	1.012	1.013
f_p/f_0 (grillage analysis)	1.011	1.018	1.015	1.015

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Imaging ultrasonic waves in complex structures using a scanning laser Doppler vibrometer

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ABSTRACT

Ultrasonic wave propagation through metallic and composite structures is of considerable interest in the context of non-destructive testing (NDT) and structural health monitoring (SHM). This paper presents images of ultrasonic wave fields in specimens varying in structure and composition. The ultrasonic waves are generated in the structures using surface bonded piezoelectric wafer actuators. A scanning laser Doppler vibrometer is used to image out-of-plane ultrasonic velocity field across the surface of the structure. The images thus obtained give valuable insight into the interaction of ultrasonic waves with various structural components (like stiffeners, bolts, lap joints and variation in thickness etc.) as well as with damages (like notches and impact damages). A proper understanding of such interactions would hopefully lead to improved damage detection in complex specimens in future.

Elastic waves in solids have been extensively used for NDT and SHM applications for many years. This includes the applications of ultrasonic guided waves, acoustic emissions and vibro-acoustic techniques. Ultrasonic guided wave based SHM techniques come with the advantages of moderately large inspection ranges and high sensitivity to small flaws. It, however, require the sensors and interconnections to be embedded into the structure, – thus adding further complexity to an already complex structural system (like a bridge or an aircraft). This limitation of applicability of ultrasonic techniques has inspired the development of non-contact ultrasound measurement systems like the one presented in this paper.

The present study uses a laser Doppler vibrometer to remotely sense ultrasonic field in a structure. Optics based generation and measurement of ultrasound has been studied in the past (Culshaw et al. 2003). More recently, Leong et al. (2005) studied the interaction of ultrasonic guided waves with fatigue cracks in a plate using a laser Doppler vibrometer. The objective of the present study is to expand the scope of using laser vibrometry to detect defects in complex structures. To this effect, the interaction of ultrasonic waves with structural components (like bolts, stiffeners and joints) has been studied in this paper and attempt has been made to distinguish such interactions with those happening at defect locations (notch and impact damage). An example result for a composite plate with impact damage (Figure 1) is shown in Figure 2.

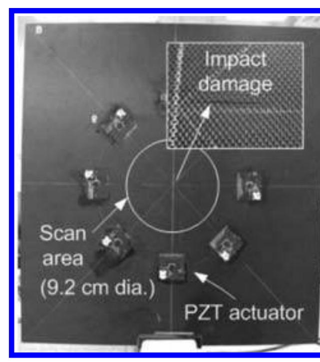

Figure 1. 1.8 mm thick composite plate instrumented with Kapton coated PZTs.

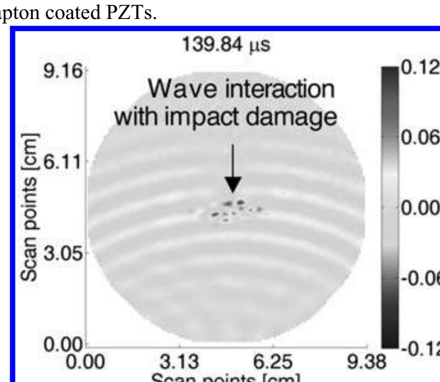

Figure 2. Contour plot of the out-of-plane ultrasonic velocity field for the specimen shown in Figure 1.

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Integrated wireless powering and data interrogation for civil infrastructure monitoring

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ABSTRACT

Post-event inspection of bridges after unexpected events (e.g. an earthquake) is an extremely important step in the path toward damage assessment and functionality restoration. We present how a team of structural, mechanical, electrical, and computer engineers are designing and deploying a variant on sensor networking to address this. The work will present a hybrid approach to sensor array powering and interrogation, where both power and interrogation commands are conveyed via autonomous vehicles. A prototype system described here is demonstrated on a full-scale bridge.

A post-event assessment may be particularly important if the system is itself a critical link in both the short-term emergency response and in the long-term economic viability of the community it serves. An example could be a bridge over which rescue vehicles must travel for first response and people must traverse for safety (short term), and which significant commercial traffic must utilize (long term). The vast majority of these post-event assessments, particularly for civil infrastructure such as bridges, are currently done by human visual inspection. In many instances, however, human visual assessments cannot be made quickly or efficiently, either a result of inspector unavailability or life safety issues posed to the inspector(s). These aspects are further highlighted when the structure itself is expansive and/or has a number of areas that are not easily accessible.

The field of structural health monitoring (SHM) is an integrated paradigm (Farrar et al. 1999) of networked sensing and actuation, data interrogation (signal processing and feature extraction), and statistical assessment (classification of damage existence, location, and/or type) that approaches structural health assessment in a systematic way. In the last decade, particularly after the 1994 Northridge, California, earthquake, some automated post-event strategies that take advantage of this SHM paradigm and do not rely upon direct human visual inspection have been developed. These techniques broadly fall into two general classes: (1) advanced data mining strategies from conventional sensor networks (e.g., extracting vibration properties from an accelerometer array) and (2) data mining strategies from remote sensing modes, such as satellite imagery or LiDAR, used in conjunction with geographic information systems (GIS). A recent literature review report considering a number of techniques for the former class (not necessarily only applied to civil infrastructure) may be found in Sohn et al. (2004), while a more specific report of some of the techniques' applications to post-earthquake building assessment may be found in Naiem et al. (2005). A recent article discussing the merging of digital photogrammetry with a GIS for post-earthquake infrastructural damage assessment may be found in Altan et al. (2001). In both classes of techniques, an appropriate sensor network is required as a first line of attack in observing the structural system behavior in such a way that suitable signal processing and damage-sensitive feature extraction on the measured data may be performed.

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Multi-scale wireless sensor node for impedance-based SHM and long-term civil infrastructure monitoring

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ABSTRACT

This paper applies a recently developed extremely compact, wireless impedance sensor node (WID3, Wireless Impedance Device) for use in impedance-based structural health monitoring (SHM), sensor diagnostics and low-frequency vibrational data acquisition. Many SHM projects involving wireless sensor nodes have been conducted, but usually over very short time frames with most tests lasting only a matter of days. In this paper, the WID3 is cast as the integral component of a permanent SHM system at a remote civil infrastructure installation, such as a highway bridge. The current generation WID3 is equipped with an Analog Devices AD5933 impedance chip that can resolve measurements up to 100 kHz, a frequency range ideal for many SHM applications. The WID3 combines on-board processing, data storage, wireless communications capabilities, and a series of internal and external triggering options into a single package to realize a truly comprehensive, self-contained wireless active-sensor node for SHM applications. The WID3 requires less than 70 mW of power to operate; it's extremely low duty cycle requirements and its ability to operate in various wireless network paradigms make it ideal for permanent monitoring of remote civil infrastructure installations. Experimental results supporting the ability of the WID3 to accurately detect damage in a permanently installed configuration over a period of months are presented.

Structural health monitoring (SHM) is the process of detecting damage in structures. The goal of SHM is to improve the safety and reliability of aerospace, civil, and mechanical infrastructure by detecting damage before it reaches a critical state. A more detailed general discussion of SHM can be found in Farrar et al. (1999). In order for any SHM system to be successful, there must be a reliable stream of operational data available for analysis, and little or no intervention should be required on the part of engineers to obtain such data. This study addresses the reliability and longevity issues surrounding the collection of data from a remote civil infrastructure site when there are long intervals between on-site maintenance of the sensing and data acquisition systems. Such a system requires a robust sensor network.

While the majority of permanently installed sensor networks today employ this wired architecture, development and deployment of wireless sensor networks has exploded in recent years. Most wireless sensor network paradigms fall into one of the categories outlined by Farrar et al. (2006). However, while much exploratory work has been done to assess the feasibility of wireless networks for SHM, very little has been done toward a permanently installed wireless sensor network for SHM.

This paper addresses several of the requirements for a robust and reliable wireless sensor network for a permanently installed SHM system. The key components required for such an installation addressed in this paper are the measurement devices (sensor nodes), the permanently installed sensors, and the specific networking strategies required to collect and analyze the data. Laboratory proof-of-concept results for various aspects of the sensing system are presented, as well as experimental results from field tests at the proposed permanent wireless SHM system installation site in southern New Mexico.

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Energy harvesting and wireless energy transmission for powering SHM sensor nodes

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ABSTRACT

The goal of this investigation has been to examine the feasibility of several different energy harvesting and wireless transmission approaches to power embedded sensing hardware. The proper management of energy resources is essential in the development of a robust wireless sensing system (Park et al. 2008). For long-term deployments, innovative power methods must be developed to supplement, or replace, the finite lifespan of traditional battery technologies. In addition, sensor nodes must be designed with energy efficient operation as a fundamental design criterion. This paper presents recent investigations into the use of energy harvesting and wireless energy transmission to power a wireless SHM sensor node previously developed by the authors (Taylor et al. 2009).

For this energy harvesting study, field data were first collected from a local highway bridge and replicated in the laboratory environment through the use of an electromagnetic shaker. Bimorph configurations of the piezoceramic lead zirconate titanate (PZT) were used as electromechanical transducers. We observed that it required 912 seconds to charge a 0.1F super capacitor to 3.5V. Once voltage levels reached 3.5V, energy was released to the sensor node, which powered itself on and measured the electrical impedance of three sensors used to monitor bolt preload. The sensor node performed a 100 point measurement sweep on each sensor, computing the maximum impedance, and storing this data to onboard flash memory. This result indicates that the PZT-based harvester system is capable of powering a piece of hardware that is capable of interrogating the structural health of a number of different systems.

Another approach to traditional energy harvesting systems is the use of RF energy transmission to remotely power embedded sensor nodes. The operation of the sensor node exclusively from power obtained through wireless energy transmission was first demonstrated in the laboratory and then performed at the Alamosa Canyon Bridge. Figure 1 depicts the charging profile within the 0.1F supercapacitor as the RF energy was being transmitted. The system was capable of charging the capacitor to a voltage of 3.7V in an average time of 53 seconds at a distance of 1.2–1.3 meters. In this figure the wireless transmission was initiated and approximately 55 seconds later the power conditioning circuit was triggered, causing the sensor node to become active and make a measurement of the piezoelectric sensors, as indicated by the voltage drop from 56 to 61 seconds.

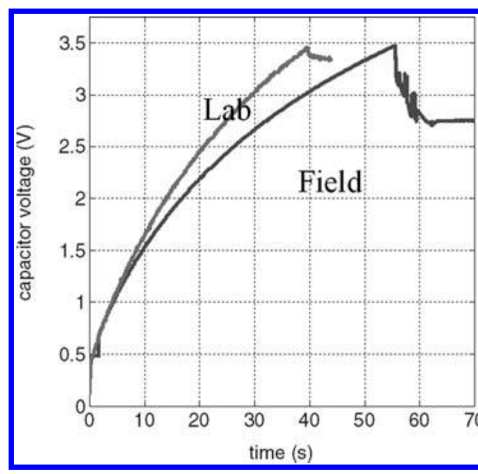

Figure 1. Voltage profile for a 0.1F supercapacitor being charged by RF energy at 5.6 GHz.

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Feasibility investigation for identifying bridge's fundamental frequencies from vehicle vibrations

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ABSTRACT

Monitoring aged and deteriorated structures including bridges have been an important technical issue in developed countries. A lot of researches on the bridge condition screening so called bridge health monitoring (BHM) using vibration data of bridges has been reported (e.g. Doebling et al. 1996). Most of all the existing studies relating to the bridge condition screening have focused on the modal properties and quantities of bridge structures (e.g. Doebling et al. 1996, Wenzel & Pichler 2006). The fundamental concept of this technology is that modal parameters are functions of physical properties of the structure. Therefore changes in physical properties, such as reductions in stiffness resulting from damages, will cause detectable changes in those modal properties.

This paper investigates how vehicle's vibrations are correlated with bridge vibrations to clarify feasibility of extracting frequencies of bridges (e.g. Ling et al. 2005) from vibration data of the vehicle traveling on bridges as a part of the drive-by-inspection system which the authors are developing (Kim and Kawatani 2009).

In this study, a traffic-induced vibration analysis of a bridge considering roadway roughness is performed as a preliminary feasibility investigation to detect bridge's frequencies from vehicle's vibrations. A moving vehicle laboratory experiment is conducted in order to confirm the feasibility of extracting bridge's fundamental frequencies from the vehicle's vibration considering roadway roughness.

From the analytical study, it is observed that the frequency (2.34 Hz) near the bridge's frequency (2.36 Hz) appears in the vehicle's vibration data as shown in Figure 1. However, it shows difficulties extracting bridge's frequency from the vehicle's accelerations clearly. On the other hand, from the experiment, the frequency (2.44 Hz) near the bridge' frequency (2.54 Hz) is more easily detectable from the response spectrum of the vehicle in comparing with the analysis as shown in Figure 2. It needs, however, further investigations on the condition of high possibility to detect the bridge's frequencies from vehicle vibrations, which is next step for this study.

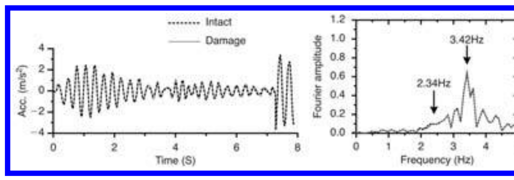

Figure 1. Acceleration responses and Fourier spectra of vehicle taken from analysis.

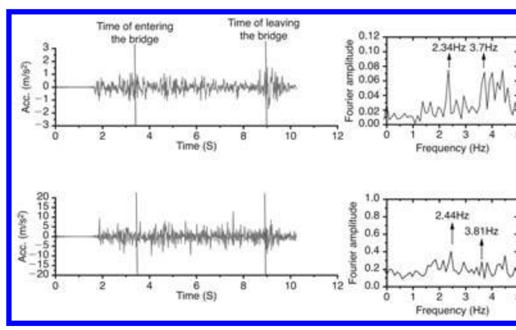

Figure 2. Responses of experimental vehicle V2 (M=21.6kg f=3.76Hz) at speed 0.93 m/s.

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Analysis and prediction for bridge maintenance costs based on life-cycle and Markov approach

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ABSTRACT

The bridge life-cycle is referred to the whole process on the planning, design, construction, operation, management and maintenance or recycling for bridge. The traditional maintenance method for bridges pays main attention to the current performance and management, but not thinks about degradation of the structural behavior and the optimal maintenance and management during the whole bridge service life-cycle or predicting life time. These cause the some problems that the just repaired bridge rose to the new defects, and even the maintenance cost is very high.

In China there had been many bridges constructed near 1990s, these bridges have being serviced for 20 years by now. The degradation and damage phenomenon of bridge became serious. However the available capital for maintenance and strengthening of bridge is very limited. It is worth to studying and concerning how to make the available fund play the biggest benefit, at the same time, make the bridges get most effective maintenance, ensure the bridges to be in the accepted service level and operation quality.

Based on condition monitoring and evaluation of the on-line bridges, this paper gives the condition assessment index system of the typical beam bridge, analyzes the performance degradation of the type of bridges causing by time-varied factors, studies the performance degradation of bridge predicted by using Markov approach and the influence of performance degradation on bridge maintenance costs. Finally, based on the analysis thought of the life-cycle, the calculation formula of the maintenance costs thinking about the influence of interest rate on the maintenance costs is proposed. The obtained results show that it is not only the least comprehensive costs, but also reliable technique, economic and reasonable maintenance prediction method when the safety operation can be ensured during bridge life-cycle.

Key words: Bridge life-cycle; Performance degradation; Condition evaluation; Maintenance costs, Markov approach; Analysis

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Transmissibility-function-based structural damage detection with tetherless mobile sensors

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ABSTRACT

In order to reduce the cost and inaccuracy associated with human inspection, prototype mobile sensors are developed for the damage detection of steel structures. The individual flexonic mobile sensing node consists of three substructures: two 2-wheel cars and a compliant connection beam. Each 2-wheel car contains a body frame, motors, batteries, a wireless sensing unit, as well as infrared (IR) sensors and Hall-effect sensors with associated hardware circuits. Meanwhile, an accelerometer is fixed at the middle of the compliant connection beam, which serves for attaching/detaching the accelerometer onto/from the structural surface. When a measurement is to be made, the two cars are driven towards each other to make the compliant beam buckle downwards to the structural surface. With the help of the small magnets fixed around the accelerometer, the accelerometer is firmly attached on the surface. When the accelerometer is to be detached, the two cars move in opposite directions to lift the accelerometer away from the surface and straighten the compliant beam. The mobile sensing nodes are capable of autonomously maneuvering on ferromagnetic surfaces.

Till now, many vibration-based methods have been developed for structural damage detection. Among these methods, transmissibility function analysis has attracted considerable attention because of its effectiveness in identifying damage using output data only. The transmissibility function T_{ij} between the output degree of freedom (DOF) i and reference-output DOF j is defined as the ratio between two frequency spectra A_i and A_j. Based on the calculated transmissibility function T_{ij}^u of the undamaged structure and T_{ij}^d of the damaged structure, an integral damage indicator between DOFs (i.e. locations) i and j is defined. The locations with the largest damage indicators are then taken as the most possible damage locations. In this research, transmissibility function analysis is adopted for detecting structural damage using the data collected by mobile sensing nodes.

A 2D laboratory steel portal frame is constructed for exploring structural damage detection using mobile sensing data. Hinge connections are adopted at the bases of the two columns. Three acceleration measurement locations are assigned on the left and right columns, respectively. Five acceleration measurement locations are uniformly assigned on the beam. A steel mass block of 0.575 kg is bonded to the left column to simulate a reversible damage. During the validation experiments, two mobile sensing nodes are assembled for simultaneously measuring the vibration at one pair of neighboring locations. To reduce the effects of experimental uncertainty, measurement is repeatedly taken for 20 times at each pair of locations. Based upon the averaged transmissibility functions obtained for ten pairs of measurement locations, the damage indicators between undamaged structure and damaged structure are calculated. In addition, repeatability indicators are computed for the data collected from the undamaged structure, as well as for the data collected from the damaged structure. The results show that the largest damage indicator occurs at the location pair where the damage was introduced, while the repeatability indicators illustrate that the experimental uncertainties have limited effects to the damage localization.

In this study, the advantage of mobile sensors is demonstrated as the high spatial resolution measurement that requires limited number of sensors and little human effort. Such advantage will allow mobile sensor networks bring transformative changes to future practice of structural health monitoring. Future research will be conducted to enable the mobile sensing nodes with the capabilities of autonomously detecting potential damages in the structure, as well as with the capabilities of maneuvering on more realistic civil structures.

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Long-term structural health monitoring for Tamar Suspension Bridge

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ABSTRACT

For structural health monitoring purposes, it has become extremely important to understand, model, and compensate for the environmental variations on both of static and dynamic characteristics of structures under ambient operation conditions. This paper presents a long-term structural health monitoring on Tamar Suspension Bridge to understand the environmental effects of temperature, wind speed and traffic volume. The monitoring system consists of two parts: a static monitoring system for cable tensions, wind, temperature measurements and a dynamic monitoring system for deck and cable accelerations. The data-driven stochastic subspace identification (SSI) method was implemented for modal parameter identification of the lower 5 natural frequencies. Environmental effects were investigated for a record of over 6 month and it was found that cable tensions appeared to be dominated by the temperature change while the frequency variations were appeared to be affected by all of the wind, the temperature and the traffic loading with different contributions in each mode. Analytical validation for the observations and identification of a feasible model are underway.

Figure 1. Tamar suspension Bridge, Plymouth, UK.

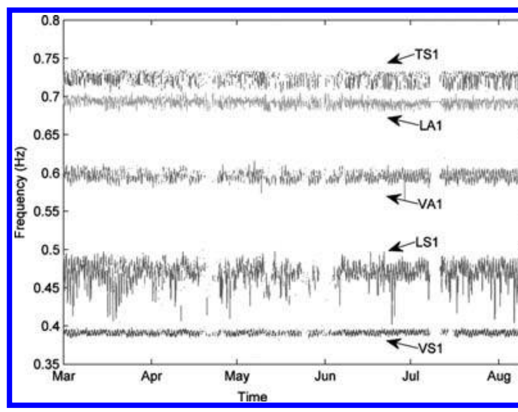

Figure 2. Long-term variations of the lower 5 natural frequencies.

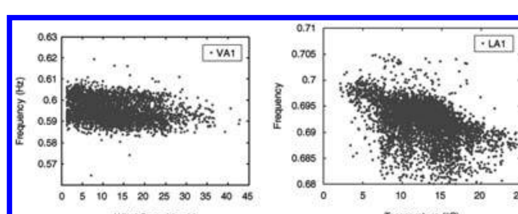

Figure 3. Wind speed and Temperature effects on the natural frequencies.

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International collaborative research and education on smart sensors and monitoring technologies

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ABSTRACT

Bridges account for a large part of the capital investment in constructing road networks and represent a key element in terms of safety and functionality. As such, informed bridge management is important to provide the public with timely transportation while maintaining a high level of safety. Similarly, detecting critical damage at an early stage can reduce the costs and down-time associated with its repair. Therefore, smart monitoring systems are being advanced with a new generation of sensing technologies and computational tools emerging from interdisciplinary researches between civil engineering and other engineering disciplines.

This study presents recent international collaborative activities on research and education for smart sensors and monitoring technologies in the Asia-Pacific region. (1) A US-Korea joint collaboration is first described exploring various emerging sensors on test-bed bridges in Korea. The heterogeneous set of sensors such as wireless sensors, elasto-magnetic (EM) stress sensors, and piezoelectric active sensors is fused into a single comprehensive structural monitoring system capable of capturing the global and local behavior of a structure, thereby leading to improvements in structural health assessment. (2) US-Korea-Japan joint research on Jindo cable-stayed bridge is presented, which aims to develop an integrated middleware services-based system for monitoring the state of cable-stayed bridge. It utilizes advanced multi-scale sensing, digital signal processing, wireless communication, and damage diagnostic methods to take advantage of a smart sensor's on-board processing capabilities. (3) International education activities are finally introduced, which are organized ANCRiSST to enhance students' understanding of the cross-disciplinary technological developments on the emerging subjects of smart structure technologies and structural health monitoring application. The summer school is a 5 year program among Korea, the US, Japan, and China. The first one was hosted by KAIST, Korea in 2008. The second one was held at the University of Illinois at Urbana-Champaign in 2009. About 50 graduate students were attended in each year. Students gained valuable experience of global engagement in a culturally rich learning environment.

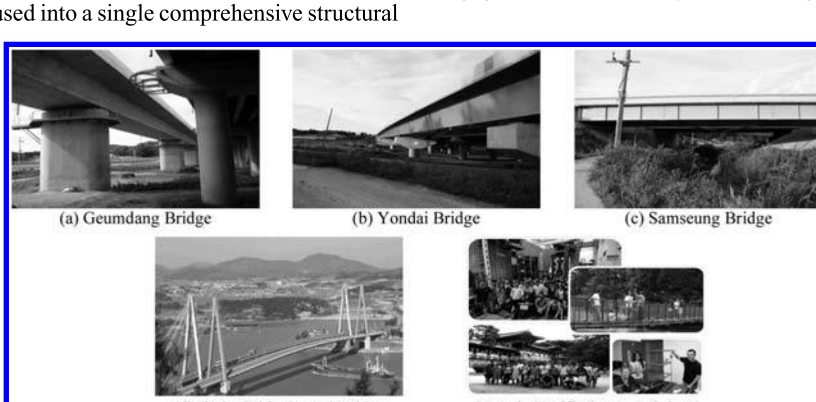

Figure 1. International collaborative researches and educations.

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Piezo paint-based smart tape sensor for bridge diagnosis

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ABSTRACT

Fatigue-induced crack monitoring and corresponding retrofit actions will lead to a prolonged life and enhanced reliability of structural systems. Manual inspection is plagued with problems such as intensive labor, time consuming process, and subjective results; therefore it is unsuitable for rapid assessment of structural conditions. In this regard, on-line structural damage detection methods with automated procedure are highly desired. Acoustic emission (AE) is the elastic wave generated by sudden energy release within a material, it provides real-time information on damage progression within a structure.

Recently, the feasibility of using piezoelectric paint to make broadband low-profile surface-mount AE sensor has been investigated by Li and Zhang (2008a,b). To achieve a low viscosity of fresh paint mixture (thus suitable for spray painting) and flexibility of cured paint, the vol-ume fraction of piezoelectric ceramic powder in the paint is controlled below 50%. The values of piezoelectric charge coefficient d33 for such piezoelectric paint formulations are comparable to PVDF. The flexible piezoelectric paint uniformly cures at ambient temperatures. Figure 1 shows flexible piezoelectric paint tape (with 8 sensor nodes) mounted on a CHS steel member. Compared with conventional acoustic emission sensors, piezopaint has several advantages: (i) Flexible piezo paint sensor can be applied to the near field locations in hot spots, even directly atop the irregular surface of weldment. (ii) It has a broadband frequency response in an ultrasonic frequency range up to 1 MHz, which is due to the high dielectric loss associated with piezo paint. (iii) The sensor has a low-profile and can be made at much lower cost. The piezo paint sensor operates in d31 mode and thus the thickness of piezo paint sensor measures only a few millimeters.

This paper describes the recent work on developing a disposable low-profile acoustic emission sensor based on flexible piezoelectric paint for bridge monitoring. The use of piezopaint for ultrasonic signal measurements is discussed along with a series of ultrasonic tests performed to verify the ultrasonic sensing capability of piezopaint. A probabilistic framework for bridge prognosis is also proposed. Issues associated with field implementation of piezo paint sen-sors on a steel highway bridge in South Korea is discussed in this paper.

Figure 1. Flexible piezoelectric paint sensor mounted on a circular hollow section (CHS) steel member.

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MS2: Monitoring & assessment of bridges using novel techniques

Organizers: A. Strauss & D.M. Frangopol

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Analysis of the structural response to a moving load using empirical mode decomposition

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ABSTRACT

The ability of empirical mode decomposition (EMD) to detect structural damage as it happens has been demonstrated in the literature (Huang & Shen 2005, Xu & Chen 2004, Yang et al. 2004), but it has not been tested for a structure with the measurements derived from the crossing of a moving load. Unlike other available methods, the EMD method has the capability to deal with nonstationary and nonlinear data. The EMD method is based on the assumption that any data consists of different simple intrinsic modes of oscillations. Each intrinsic mode, linear or nonlinear, represents a simple oscillation which has the same number of extrema and zero crossings. The oscillation is also symmetric with respect to the local mean. Each of these oscillatory modes is represented by a Intrinsic Mode Function (IMF).

In this paper, the structure has been modeled as a one-dimensional simply supported discretized finite element beam. Damage has been simulated as a decay in stiffness that covers a beam length related to the severity of the damage. The moving load has been assumed to be constant and driven at uniform speed over the structure. Then, accelerations due to the passage of the load have been simulated at a number of sections across the beam. The EMD method breaks the acceleration signal into the relevant number of IMFs. Then, an intermittency or cut-off frequency is applied to the first IMF. Results have shown EMD is able to detect the damage through a spike in the filtered first IMF of the transformed signal. The spike takes place at the instant the load passes over the damaged location regardless the measurement point. A sensitivity study has been carried out varying observation point, velocity of the load (5, 10, 15 and 20 m/s), beam length (10, 15, 20 and 30 m) and severity of damage (5%, 10% and 20% ratios of crack height to beam depth). In all cases, EMD has accurately located the damage. Finally, the acceleration has been corrupted with noise to simulate real measurements with signal to noise ratios of 20, 10 and 5. In the presence of high levels of noise, the spike has been found in the filtered second IMF more clearly than in the filtered first IMF. The result of applying EMD to the simulated accelerations is shown in the figure below. Figure 1(a) corresponds to the midspan acceleration response of a 10 m span beam with a 20% damaged section at 6 m from the bridge support as a load crosses at 5 m/s. Figure 1(b) shows the result of calculating the first IMF of Figure 1(a) with an intermittency frequency of 100 Hz. In the latter, a peak associated to damage is detected at 1187 scans when scanning at 1000 Hz (these scans represent $1187/1000 \times 5 = 5.94$ m, very close to the true damaged location at 6 m).

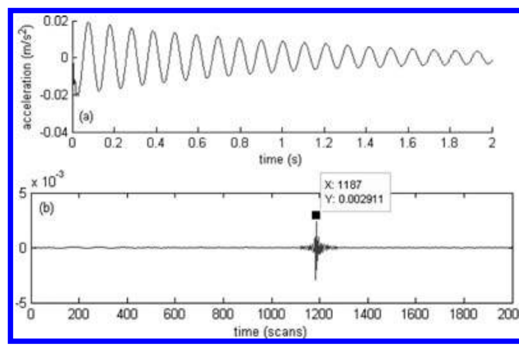

Figure 1. EMD damage detection algorithm applied to the midspan acceleration response: (a) Acceleration response, (b) Filtered IMF1 with a cut-off frequency of 100 Hz.

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Measurement tool kit for corrosion and defect monitoring of bridge tendons

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ABSTRACT

One of the most important tasks and challenges of Life-Cycle-Engineering and bridge management is to ensure the safety, serviceability and robustness of the prestressed members during service life. Here, corrosive and mechanical influences on the steel reinforcement may strongly affect the long-term performance and durability of internal and external tendons of concrete structures. On this account, the condition assessment of prestressed members should be addressed to the detection of existing damages, such as grouting faults, corrosion defects and fractures of the steel elements.

But unfortunately for grouted interior or for ungrouted exterior tendons no accurate and reliable non-destructive methods are available today for the monitoring of the corrosion state, for the localization of fractures or for the measurement of the actual tensile force of tendons.

Several novel techniques for force measurement resp. for corrosion and fracture identification at bridge tendons have been developed by the authors within the framework of the Collaborative Research Center SFB 477 'Structural Health Monitoring' at the University of Braunschweig in the last decade.

Based upon a the three-level monitoring strategy (Fig. 1) these redundant NDT/NTE-methods offer an efficient monitoring tool kit for tendons of P/C-bridges as well as for ground anchors and ropes.

The main focus of this paper lays on the practical testing and verification as well as on the specification of achieved improvements and further developments of the innovative sensing and monitoring techniques, listed below:

– Magnetoelastic coiled sensors for the force measurement and material defect detection of prestressed tensile elements.
– Smart calibration-free filament sensors for corrosion monitoring consisting of a single or several parallel arranged, 0.065 to 0.5 mm thin steel wires, working as "watch-dog-sensors" for the identification of steel corrosion initiation and progress in the concrete cover resp. injection grout of ducts.

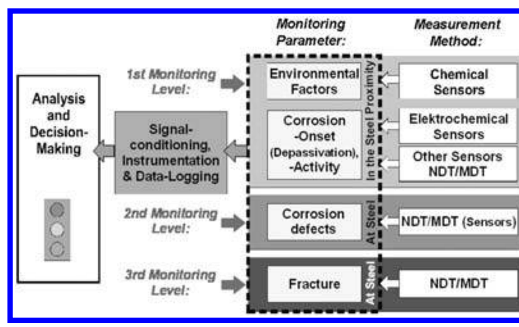

Figure 1. Schematic diagram of a possible corrosion monitoring strategy for prestressed elements, extended acc. to Holst et al. (2006); NDT/MDT- non/minor destructive techniques.

– A novel type of electrochemical chloride electrode sensor for the measurement of the free chloride content in the pore solution of concrete and
– The electromagnetic resonance measurement method (frequency domain reflectometry, FDR) in combination with electromagnetic field strength investigations for the detection and localization of prestressing steel fractures and of other defects based upon microwave technology.

The functionality as well as the application of the distinct methods, showing different stages of development, were tested and verified under field-conditions, e.g. at a 18 m long prestressed trial concrete bridge and at first applications at real structures. Based upon this, the pros and cons of the methods are discussed.

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Field application of innovative scour monitoring techniques for bridges

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ABSTRACT

Following the global climate change, flood disasters news has been reported worldwide. Consequently, Taiwan is well known for its famous topography features that the central mountain range bisects the island from north to south leading to most rivers have short courses and rapid streams and is subject to several typhoons and flood events each year during the summer and fall seasons. Flood water can intensify the scour menace to bridges since the unsteady nature of the flow. Furthermore, floating or submerged debris induced by flood may deepen the scour depth of bridge foundations. Thus the hydraulic causes of bridge failure have focused engineering attention on the bridge scour problem.

However, in-situ bridge scour monitoring is still one of the rugged works for field application researchers. Challenges include bridges under condition of the uncertainty, harsh environment of system servicing events, long-term performance of sensors and packaging modules, ease of installation and cost of deployment and maintenance. In this paper, we focus on the use of innovative technologies for monitoring bridge scour processes. This study applied the innovative scour monitoring techniques which have been designed and developed in the laboratory condition to field bridge sites. Two developed scour sensing techniques were combined into one scour monitor tube, which comprises optical fiber sensors and micro-electro-mechanical systems based sensors and designed to be installed on adjacent bridge foundations for scour monitoring. The layout of the field deployment of bridge scour monitoring system proposed showed in figure 1.

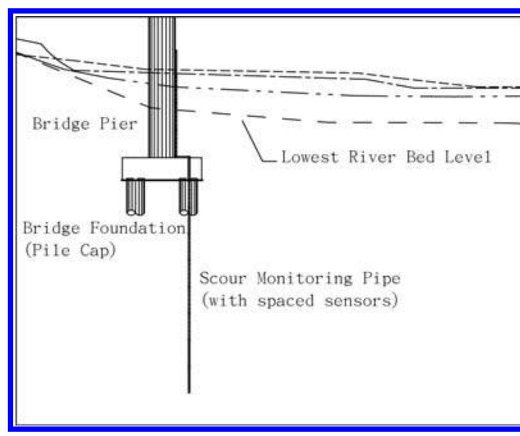

Figure 1. Layout of the field scour monitoring system.

The system can detect the depth of scour near bridge foundation under flood water, and provide reciprocal verified result for the monitoring information with two sensing techniques included. The system proposed in this paper has been installed on in-situ bridges and be ready for presently application research. The purpose of the field deployment was to evaluate the preliminary design in terms of durability of packaged sensors, reliability of communication under real operational environment. With the experience of the success of this system settled in field, further study could be conducted with acquired field data for scour condition interpretation. More appropriate countermeasures to protect the bridge foundation and reduce scour effect could be drawn up as well.

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Performance evaluation of bridge seismic bearings based on in-situ quick-release tests

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ABSTRACT

A nonlinear model-based system identification method is proposed, formulated, and implemented. The proposed method is applied to quick-release field experiments on a three-span continuous base-isolated bridge in order to investigate aging and temperature dependent effects of lead-rubber bearings.

A quick-release (or pull-back) test is a free vibration test method where lateral forces are applied to the superstructure and released quickly to introduce a free vibration to the bridge. In the present study, transverse and rotational rigid-body motions of the bridge superstructure are formulated into two degrees-of-freedom dynamic governing equations. To model hysteretic behavior of lead-rubber bearings (LRBs), the Menegotto-Pinto model is used. Among several quick-release experiments the authors investigate: i) aging effects of LRBs by utilizing two experimental data sets taken several years apart, and ii) temperature effects of LRBs by utilizing two experimental data sets conducted in warm and cold weather.

For given dynamic governing equations and an input excitation (or free vibration), the resultant motion is a function of the parameters in the governing equations. System identification is a process to determine values of those parameters in a model that can reproduce the measured responses. Therefore, system identification frequently becomes an optimization problem to find optimal values of the parameters to minimize the difference between measured and reproduced responses.

Because the existence of a unique solution cannot be guaranteed, system identification is referred to as an ill-conditioned or an ill-posed problem. Another difficult feature of the present problem is the large number of parameters to be identified. The proposed system identification, therefore, is divided into two phases as in Figure 1.

Results of applying the proposed method are summarized as follows.

- The two degree-of-freedom governing equations for transverse and rotational rigid-body motion of the superstructure can successfully capture free-vibration motion in quick-release tests.
- Regarding aging effects, increases of the pre-yielding stiffness and the post-yielding stiffness are observed.
- Regarding temperature dropping effects, the decrease of energy dissipation capacity and the increase of the pre-yielding stiffness are observed.
- Current practice does not consider changes in the pre-yielding stiffness; however, results from system identification indicate its effects on response are significant.

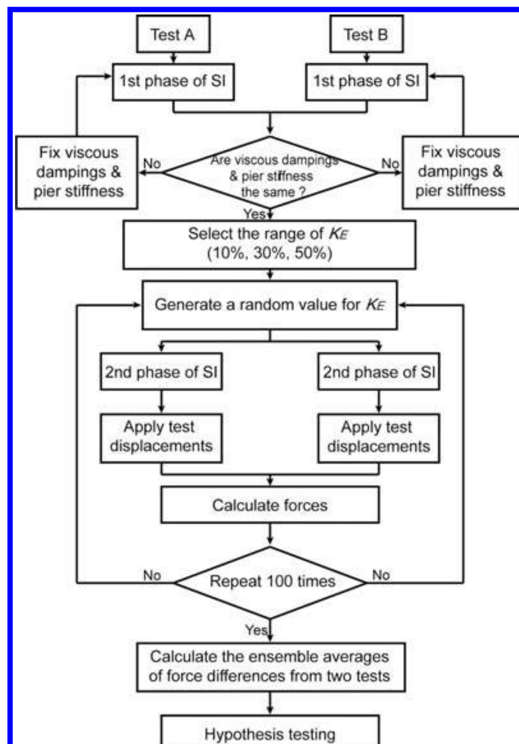

Figure 1. Two-phases System Identification.

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Experimental assessment of prestressing force in concrete bridges

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ABSTRACT

Assessment of the prestressing force of existing prestressed concrete bridge superstructure is a main object of the paper. It has been assumed; that proper level of prestressing force is one of the most essential factors from the structural safety point of view.

They are some signs that might prove excessive reduction of prestressing, e.g. residual deflection of span or visible cracks on the surface of span girders. But more exact evaluation of prestressing force needs some authentic and empirically verified procedures.

Suggested experimental procedure of evaluation of prestressing force concerns such aspects, as:

- Analysis of stiffness of girder under applied proof load, based on load-deflection relationship for uncracked element. The load-deflection curve is generated using experimental influence surface of deflection for prestressed element.
- Moment of crack formation in prestressed element, when we are able to cause cracks under load. In that case assessment is based on the comparison of the real crack moment with calculated value.
- Comparison of crack pattern of prestressed concrete bridge element and cracks generated in its structural model, in case of cracked structure. The general analysis method corresponds to the best match between structure and its structural model.

If the prestressing force is below design value, in most of cases the reason of the lack of prestressing is the key to assessment of the safety of the structure. Improper level of prestressing may be result of some negligence under construction, excessive losses of prestressing forces and also corrosion, which is the most dangerous case for the safety.

In the Figure 1 it has been suggested general scheme of procedure for the assessment of the safety of bridge span based on the evaluation of real prestressing force in the structure.

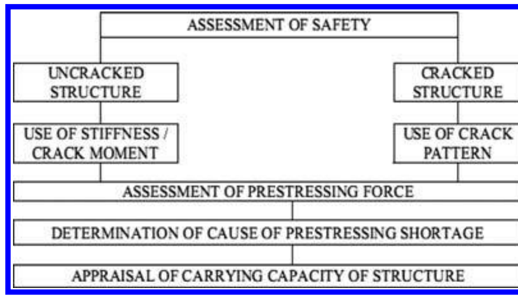

Figure 1. Flowchart of assessment of safety of prestressed concrete structure.

In case of existing prestressed concrete bridge, generally we may have two possibilities:

- they are no cracks in all of essential elements of bridge,
- cracks exist in some of its elements.

In case of uncracked structure the analysis of stiffness under applied load is the most important resource of information about prestressing force.

In case of cracked structure comparative analysis concerns the crack pattern of real structure and the map of stresses in structural model of the bridge may be used.

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Assessment and strengthening of prestressed damaged beams

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ABSTRACT

The paper describes the evaluation of the actual conditions, the repairing design criteria and procedures, and the final validation tests carried out on high damaged prestressed beams of the viaduct "Fiumara of Tito" on the highway RA005 Sicignano-Potenza in Basilicata – Italy.

The structure, built in 1970, was affected by serious damages involving the prestressed beams. In particular, a state of diffused cracks and uncontrolled water flows has induced the partial corrosion of the steel prestressing cables, causing a reduction of the bearing capacity of the decks.

In a first phase several tests have been carried out on the beams of the viaduct with the aim of assessing their structural conditions. First of all, ware carried out specific measurements for the identification of the cables layout using electromagnetic and radar detectors.

Then were performed direct and borehole visual inspection of the cables. In many cases resulted non-corroded strands but absolute absence of injection mortar.

For evaluating the stresses acting on concrete due to the residual prestressing force and to the dead loads were performed some release tests both on concrete and on the steel strands. The results of this test have pointed out very low stress values in concrete till about 1.5 MPa indicating an important loss of prestressing force near the intrados of the beams.

Release tests on the prestressing strands have also been performed in order to estimate the residual tension acting in the cables. This tests have confirmed the loss of prestress since have been measured values till 600 MPa that are about the half of the initial value of 1000 MPa.

On the basis of the in situ test results a specific reinforcing system, consisting in external unbounded cables and in a FRC continuous reinforcement of the bottom of the beams, has been designed with the aim of restoring the original structural performances of the beams.

The tensioning procedure of the external cables of each beam was then monitored by strain-gages, load cells and by measuring the vertical deflection using laser optical levels.

The reinforcing system applied to the beams allowed the increasing of the global safety factors and the reduction of the risk of cracks opening, protecting, in this way, the existing cables from further corrosion.

The validation of the repairing works was finally performed by static load acceptance tests.

Figure 1. Repaired beam.

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Development of rapid, reliable, and economical methods for inspection and monitoring of highway bridges

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ABSTRACT

Highway bridges are vital links in the transportation network in the United States, providing the public with routes for daily commutes and businesses with the infrastructure needed to supply goods and services. Identifying possible safety problems in the national inventory of more than 600,000 bridges is generally accomplished through labor-intensive, visual inspections, which are required at least once every two years. Pending legislation would increase the inspections for fracture-critical bridges to once a year and likely strain DOT resources that are already stretched to the limit. This paper outlines ongoing research sponsored by NIST to improve inspection practices by providing the technology and methodology for real-time monitoring of bridges.

Past monitoring systems for bridges have been time-consuming to install and unreliable in operation. However, with improving technology, the ability to develop resilient systems that are simple to install and maintain are possible. A likely candidate for long-term monitoring are fatigue sensitive areas in fracture-critical steel bridges. The concentration of the study is on developing a long-term monitoring system that is economical and rugged—able to accommodate the severe temperature ranges, humidity fluctuations, and natural weathering processes that bridges experience. To make installation easier, the focus is on wireless sensors designed to continuously monitor fracture-critical bridges with a targeted 10-year battery life. The sensor nodes will be capable of supporting multiple sensors with sufficient computing power to process raw sensor data and send notifications off-site when a threshold level of damage occurs.

The use of an advanced monitoring system will enhance detection of distress without the need to mobilize an inspection crew or disturb traffic, while giving transportation officials the tools to better allocate inspection resources.

A LabVIEW program based on simplified rainflow counting was developed to assist in diagnosing and estimating the fatigue life of a bridge. The program was implemented on a bridge for nearly two weeks and compared to the results of the onboard rainflow algorithm on a CR5000 data acquisition system. Data was further analyzed using the LabVIEW program to understand the fatigue implications using Palmgren-Miner's rule. This paper will present the results of that field study.

Understanding the results from a rainflow analysis is beneficial for evaluating the health of a bridge and its components. Due to the similarity between the CR5000 bin data and the bin data from the LabVIEW-based program, the LabVIEW-based program was validated for future use. Also, changing the time period for outputting bin results did not significantly affect the results for the time periods investigated (0.5 hour, 2 hours, 12 hours, and 24 hours). Part of the small variation between bin results for the different periods is due to the negligible influence of temperature-induced drift for the installed sensors. However, the similarity between the results also suggests that 0.5 hour (30 minute) segments can be used for rainflow analyses without counting too many cycles in the close out period and skewing the results. Finally, having 30 minute segments reduces the memory requirements of the wireless devices while minimizing the effect of temperature-induced sensor drift.

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Wireless sensor performance monitoring of an innovative bridge design in New York State

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ABSTRACT

With many of today's bridge superstructures deteriorating at such a rapid rate, there is a need to develop new structures that can last longer and stand up to the effects of both greater loading and harsh environmental conditions. As innovative materials continue to find their way into civil engineering design, monitoring their performance is of utmost importance. Discussed in this paper is the deployment of a custom designed wireless sensor system on an innovative bridge superstructure in New York State. The structure utilizes glass fiber reinforced polymer (GFRP) bridge deck panels as a replacement to a deteriorated steel grated bridge deck. A wireless sensor system developed in the Laboratory for Intelligent Infrastructure and Transportation Technologies (LIITT) providing acceleration and strain measurements at critical locations of the superstructures is deployed on both bridges. The wireless system provides independent conditioning of acceleration, strain and temperature measurements with high rate, real-time lossless transmission capabilities. These measurements serve to provide static and dynamic load testing parameters and modal characteristics of the superstructure under both ambient excitation and service loads. The results offer performance evaluation as well as verification of design methodologies employed for the newly constructed bridges.

Testing was completed over a two day period where both strain and acceleration measurements were captured under forced and ambient loading conditions. Strain transducers were deployed at the midspan of each girder in addition to the locations near the abutment of the center girder. Neutral axis locations, transverse load distribution factors and end fixity were detected based on loading from a calibrated H-truck in three separate loading lanes. The results indicated minimal composite action between the FRP panels and steel girders. A typical strain time history of top and bottom flange measurements at an exterior girder can be seen in figure 1. Strain readings at the abutments indicated a strong level of fixity present in the integral abutment design. Load distribution factors suggest that the vehicle demand shed within the superstructure.

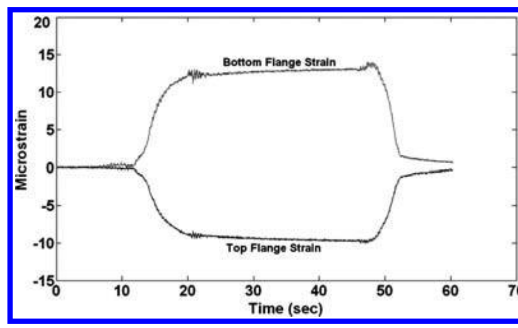

Figure 1. Typical strain time series for an exterior girder.

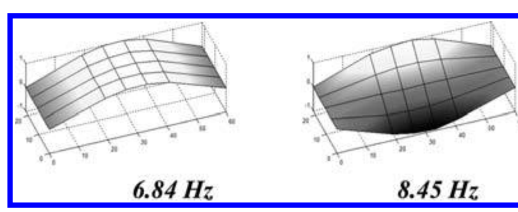

Figure 2. Mode shape estimates as reconstructed from output-only system identification.

Multiple tests were completed in each lane to validate the data. A strong correlation is seen among the results.

Modal analysis from ambient loading conditions was completed on day two. Peak excitations from vehicular traffic rarely exceeded 10 mg. Nonetheless, structural poles within the average power spectrum were clearly identified. Output-only system identification was performed using Stochastic Subspace Identification. The first 8 lower order modes are presented in the paper, with the first 2 shown in figure 2 above.

The results from the deployment provide indication of the performance of the bridge superstructure. With testing completed near the beginning of the bridge service life, the data obtained can be compared to future testing results for monitoring the long term health and performance of the FRP design.

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The benefit of monitoring for bridge maintenance

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ABSTRACT

Apart from conventional bridge control, more and more measurements and monitoring concepts for structures have been carried out in the last years meant at giving information on the way the condition of bearing structures changes with the passing of time and at supporting bridge investigation by means of recording objective data. Still, these technologies did not reach in the past the desired degree of application as they tried to position them primarily on the basis of vibration measurements with only few sensors. This approach however could not find any acceptance with bridge owners as influences, like temperature, on the resonance frequency are far greater than the influences of damage.

Recent experience has shown that monitoring concepts need specific change if they are to be used more often. The following theses are at the centre of the approach to applying monitoring presented here:

(1) Global methods do not permit early diagnosis of damage at acceptable cost.
(2) Data-based investigation is ideally suited to observing known problems or damage and changes in these over time.
(3) Objective data are collected as input parameters for further investigations.

For each thesis benefits of monitoring may be achieved. As an example the implementation of the second thesis is shown on a famous bridge in the full paper.

1 MONITORING SYSTEM

The Erdberger Bridge with a total length of 147 m and a width of 42,30 m built in the years 1968 to 1971 representing a milestone in bridge engineering due to the design concept based on a reinforced concrete shell.

Figure 1. View of the Erdberger Bridge.

During a recent bridge inspection a relatively poor condition of the bearing structures with lots of damage was revealed. The bridge authority decided prior to the general rehabilitation of the structure to monitor the evolution of the bridge condition.

The system observes automatically horizontal movements of the arch foundations by laser and the connection of old, existing mechanical extensometer sensors, bridge temperature, behavior of backstay tendons and overall changes in length. The design and results of the monitoring system are described in the full paper.

2 SUMMARY

The processing of monitoring data is an efficient and reliable method of monitoring the safety of an structure. Through the connection of the unit to the internal fiber optics network of ASFiNAG, information on the load bearing behavior can be timely provided and if need measures can be taken.

Analysis done on the basis of performance data is best suited for observation of known problems or special questions like in our example. This application is extremely interesting given that in case of problems the bridge owners consider elaborate measuring and analysis to be very reasonable in order to keep records of the evolution of the condition of certain parameters.

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Monitoring method for curved concrete bridge girders using long-gauge deformation sensors

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ABSTRACT

As a consequence of their shapes, curved concrete bridge girders have complex internal force distributions and it is recommended to verify the real structural performance of curved girders upon construction and to monitor their behavior in long-term. A structural health monitoring method for curved concrete girders, based on the use of various to pologies of long-gauge deformation sensors combined with inclinometers and temperature sensors, is presented in this paper.

The method is loosely model-driven – the characteristic cross-sections for monitoring are first selected based on a generalized structural analysis of the girder, and then each cross-section is equipped with a sensor topology or a combination of sensor topologies, that can characterize in the best manner the expected influences.

A parallel topology is used for the monitoring of cells subject to bending, assuming that Bernoulli hypothesis is valid. It consists of two sensors with equal gauge lengths, parallel to the elastic line of the beam and installed at different levels of the cross-section. A crossed topology consists of two crossed sensors installed with a predefined angle with respect to the direction of normal strain lines. The aim of this topology is to detect and quantify the average shear strain in the plane of the sensors.

Finally, the results of monitoring of each cross-section are "linked" together in order to assess the global structural behavior. The method allows for global monitoring of axial strain, horizontal and vertical curvature changes, torsion, average shear strain, rotations in both vertical planes, and deformed shapes in both vertical and horizontal planes. The sensor network resulting from the method is given in Figure 1.

The method is illustrated and its performance is evaluated through a practical example: a 36-meter long

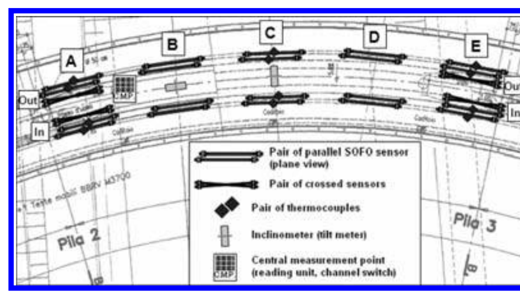

Figure 1. Sensor network for curved beam, plan view, sensors not to scale.

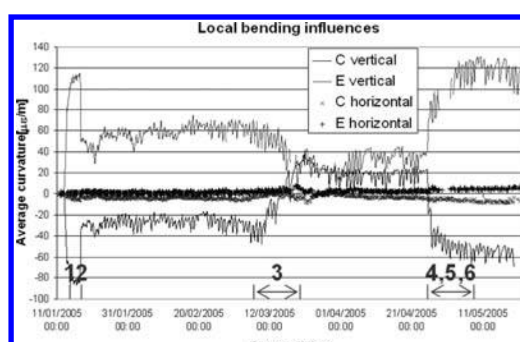

Figure 2. Average curvatures in cells C and E.

curved post-tensioned bridge box girder equipped with fiber-optic sensors. Important parts of the structure life such as construction and post-tensioning are registered, analyzed and presented. An example of results is given in Figure 2, where numbers 1 to 6 represents various construction works.

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Multiple sensor subsurface condition assessment of reinforced concrete bridge decks

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ABSTRACT

This paper reports on a study aimed at accurately assessing the subsurface damage conditions of reinforced concrete bridge decks with multiple non-destructive evaluation (NDE) techniques. Information gained from such evaluations may significantly enhance the effectiveness of deck maintenance activities. The present state-of-the-art is that multiple NDE techniques are available, but that the accuracy and reliability of the methods are not guaranteed. NDE techniques generally employ electromagnetic, electrochemical or elastic wave principles. The sophistication ranges from the simple chain drag, to complicated multi-channel radar and ultrasound instruments. Recent field studies comparing NDE methods with each other and ground truth data have indicated a relatively high degree of variability and disagreement among the sensors. The foci of this study are threefold: 1. Compare the sensing data from multiple sensor methods applied to bridge deck specimens presently undergoing accelerated degradation in the laboratory. The sensors include multichannel ultrasound, ground penetrating radar (GPR), anode ladder, inductive rebar heating with infrared imaging and half-cell electrochemical potential. The accelerated degradation combines salt bath cycling with mechanical loading. 2. Attempt to understand the effect of different stages and type bridge degradation on the sensor signals. In particular, reinforcing corrosion, cracking around rebars and delamination are different, but related damage patterns. These different damage types can produce different effects on the sensors. For example, corrosion and water-chloride contamination may produce a strong absorption of radar waves, but produce relatively modest changes in ultrasound and chain drag tests. Conversely, air-filled delaminations are difficult to detect with standard GPR, but are readily detected with chain drag and impact-echo. 3. Explore methods of fusing the electrochemical, electromagnetic and elastic wave data based on degrees of belief in sensor information to produce an enhanced assessment of subsurface conditions. The ultimate goal is to produce an automated easy-to-use multisensor bridge deck assessment system.

Figure 1. Ground penetrating radar assessment of bridge deck.

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Output-only substructural identification for local damage detection

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ABSTRACT

With declining state of aging infrastructures such as bridges and highway systems, the ability to detect any damage at the early stage would help to reduce maintenance costs and increase safety to the public. In this regard, there has been increasing interest in R&D on structural health monitoring. For large and complex structural, however, this is a difficult task. Not only a large number of sensors are needed, but also formidable computational challenges are encountered in order to achieve satisfactory results for the purpose of damage detection. The task becomes much more challenging if input force measurement is not possible and only acceleration data at certain locations are available. To this end, this paper presents an output-only substructural identification for detecting damage within a large structure based on acceleration measurements to infer changes in its stiffness parameters.

The study uses an improved genetic algorithm method to identify the stiffness and damping parameters of substructure. A key advantage of the proposed substructure strategy is its ability to simultaneously identify structural parameters and input excitation force within the substructure. This is done ingeniously by adopting a predictor-corrector algorithm to correct the output responses of internal acceleration, velocity, and displacement that are predicted using numerical integration. Instead of using the predicted (or measured) internal responses, the corrected responses are adopted in the numerical integration to predict the responses in the next time step, resulting in reducing the accumulation of errors due to measurement noise.

To demonstrate the performance of the proposed strategy, numerical study is carried out for a long continuous truss bridge (as shown in Figure 1). Using acceleration response contaminated by 10% noise in the numerical simulation, stiffness parameters of a substructure of a two-span continuous truss bridge are identified with mean error of less than 5%. In addition, simulated damages in this substructure with two damage extents are successfully detected and quantified with mean error of less than 3%. The results show that the proposed strategy can be used for local damage detection. The proposed substructure strategy is further substantiated by an experimental study of a laboratory-scale steel frame model.

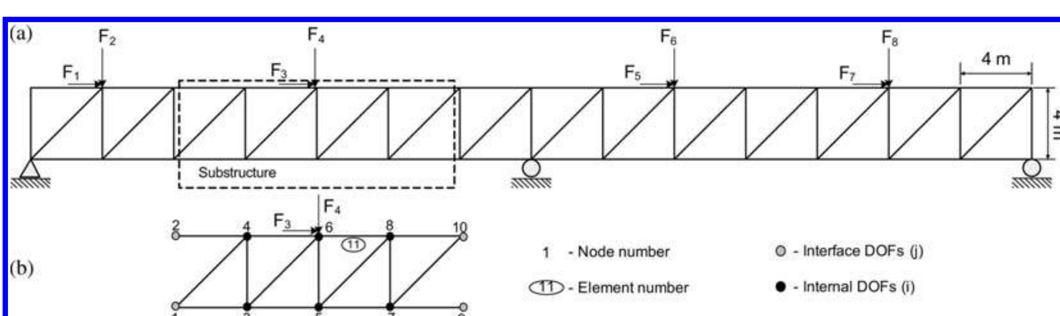

Figure 1. Long-span truss structure: (a) full structure; (b) substructure.

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Steel beam fatigue life prediction using acoustic emission amplitude histograms and backpropagation neural networks

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ABSTRACT

This paper applies nondestructive evaluation (NDE) methodology to predict fatigue life in structural steel. Specifically, it applies acoustic emission (AE) nondestructive testing (NDT) to monitor the development of fatigue crack growth in bridge structures simulated in this research as transversely loaded beams. A backpropagation neural network (BPNN) is then utilized to perform fatigue life prediction. Tests were done on fourteen transversely fatigued S4 × 7.7 I-beams made out of A572-G50 steel. Although the I-beams were tested under the same conditions, the material and slight geometry differences between the specimens still produced significant variations in the fatigue life of 13.6% coefficient of variation. The networks prediction based on the data for first, second, and third quarter of the fatigue lives was 18% and 16% and 5% error, respectively for training on five specimens and test on five. This trend shows good promise as the prediction accuracy increases with the stage of fatigue life.

1 INRODUCTION

As the structures age and funding for new infrastructure decreases there is a need to maximize the use of the existing infrastructure. Safety however is a primary factor because of this strong and reliable evaluation and prediction techniques are necessary. Fatigue failure investigation of structural steel has been well researched during the last several decades. The universal consensus is that when it comes to health monitoring of steel structures, a NDE methodology is desirable. AE is among the most suitable techniques when used in conjunction with other NDT methods such as visual inspection.

1.1 *Acoustic methodology*

Acoustic emissions function on the basis that sounds can be transmitted throughout a material. Where AE is truly powerful is that even signals as small as microstructure slipping or cracking would be picked up by using a piezoelectric transducer. The transducer senses the vibrations on the surface where it is mounted as result of the acoustic source. These signals serve as parameters for the Neural Network.

1.2 *Artificial neural network*

The backpropagation neural network (BPNN) is widely used in predictive learning, and in cases in which a substantial amount of data is available with complex relations.

2 CONCLUSIONS

The research presented here is a continuation of research for prediction of fatigue lives for tension members where even with variability of different stress levels the networks still predicted with less than 12% error. For the I-beams although the loading conditions were kept the same slight material differences created a 13.6% coefficient of variation in the fatigue lives. The networks prediction based on the data for first and second quarter of the fatigue lives was 18% and 16% error for training on five specimens and test on last five respectively. The prediction based on the third quarter of life had error of about 5%. This is because the crack propagation is more predictable than crack initiation. For many applications such as monitoring of bridges it is actually preferable that the prediction is more accurate for the later part of life as that is what would happen in the field.

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Health monitoring of concrete bridges: model simulations of pre-stressed beams under static environmental loading based on experimental data

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ABSTRACT

As recent literature has highlighted, the effects of environmental parameters on the sensitivity of the proposed damage detection techniques have to be deeper investigated. For this reason an experimental monitoring program has been performed in order to follow the long-term strain measurements of two pre-stressed concrete beams (one undamaged and the other one progressively damaged) placed not in a controlled laboratory environment but outdoor under environmental conditions. This work is focused on the development of the numerical simulations based on these experimental strain measurements. Particular attention has been focused on the comprehension of the reliability of the performed laboratory experiment. Damages of different intensities and locations have been introduced into the model.

Simulated strain histories show that the models well catch the temperature-induced variations in strain, and they are very well correlated to the temperature variations (Fig. 1). By the way, a deviation can be observed between the simulated and the measured responses. The strain versus temperature plot for both responses shows that the slope of the relationship is the same for the simulated and measured responses, so to confirm the fact that the model well catches the thermal behavior of the beam. However, the relationship for the real structure shifts in the first months of the monitoring before reaching a stable trend. As these shifts are usually associated to changes in the mechanical system, that means that something not controlled occurred in both beams in the first period of the monitoring, probably a change in the support conditions or a general settlement of the structure. Furthermore, the plot shows a thermal hysteresis in the measured structural response indicating that the relationship between strain and temperature is non linear, as the structure has a thermal inertia that the current model is not able to describe.

The observed discrepancy highlights the doubts related to the use of model-based approaches in disclosing damage. This work shows the difficulties encountered in the construction of a numerical model catching the behavior of a simply supported beam under environmental conditions. A model really representative of the mechanical system is hard to realize, in particular when complex structures have to be modeled. A non model-based approach could give better performances when a long reference period is available. Furthermore, the computation of the mechanical strain after removing the thermal strains seems to be promising because it could help to detrend the strain histories from environmental events.

Work supported by the European Community and FEDER founds within the duratiNet Interreg (Atlantic space, project N° N° 2008-1/049).

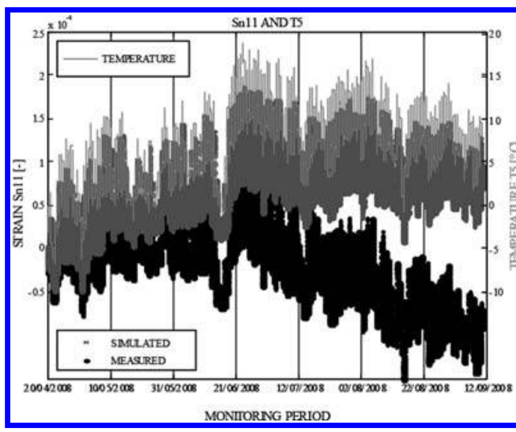

Figure 1. Simulated and measured strains (Sn11) and corresponding measured temperature (T5).

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Jointless bridge: Determination of fracture mechanical parameters values for nonlinear analysis

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ABSTRACT

For complex stochastic nonlinear analyses of bridges (old or new ones) the following steps are required:

1. Laboratory or in-situ tests to determine values of basic fracture mechanical parameters and their statistics.
2. Inverse analysis to identify values of other parameters needed in numerical model.
3. In case of old bridges the damage detection (position and magnitude) using structural health monitoring data.
4. The development of advanced numerical computational model based on stochastic nonlinear fracture mechanics and knowledge of possible damaged parts and material parameters from laboratory tests and identification.
5. Stochastic nonlinear analysis to assess reliability, risk and lifetime of a structure.

The proposed paper is focused on first two steps of the whole procedure – on the determination of fracture mechanical parameters values of concrete used for casting of newly built Jointless Bridge in Austria. For that purpose six specimens of nominal size $100 \times 100 \times 400$ mm with the edge notch of the depth about 1/3 of the depth of the specimen in the centre of the beam were tested in three-point bending (3PB) configuration. Loading span was equal to 300 mm. Example of the tested specimen is in Figure 1. After three-point bending test, two broken parts of each specimen were cut to nominal size $50 \times 50 \times 50$ mm using diamond saw and than subjected to compression tests.

Subject of interest were following parameters of concrete: specific fracture energy, modulus of elasticity, tensile and compressive strength, effective crack elongation, effective fracture toughness and effective toughness. Determination of parameters was done using two techniques – (i) direct evaluation from experimental load–deflection ($l-d$ diagram) by effective crack model and work-of-fracture method (Karihaloo 1995); (ii) inverse analysis using artificial neural network based method (Novák & Lehký 2006). Both techniques provided results which are close to each other including basic information on variability (COV). Inverse analysis provided additionally values of tensile strength of concrete. Results efficiently serve as input data for stochastic nonlinear simulation of studied bridge (Podroužek et al., 2010).

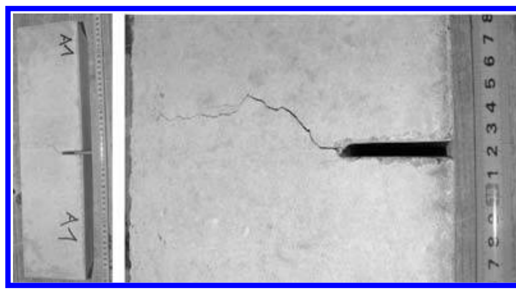

Figure 1. Selected specimen after three-point bending test; detail of crack path (right) [photo: B. Kucharczyková].

ACKNOWLEDGEMENT

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Coupled field monitoring and structural analysis to assess scour conditions

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ABSTRACT

The Indian River Inlet Bridge (IRIB) is a steel I-girder bridge subjected to strong tidal flows, causing significant scour holes to form on both sides of two in-water piers (in the direction of flow). Because of this, a monitoring system consisting of tilt sensors placed at the top of each of the in-water piers has been installed. The usefulness of this system in evaluating bridge strength and stability through comparison of the monitoring data to results of structural analysis models is the primary focus of this paper.

Two different types of structural analysis tools are developed in this work. These consist of a pushover capacity analysis of the pier to evaluate stability of the substructure and finite element analysis to determine stress conditions in the superstructure as a result of pier movements.

The displacements obtained from the monitoring are compared to the results of the pushover capacity analyses to ascertain the potential for instability of the piers. Due to uncertainties in both soil properties and potential scour conditions, a parametric study on the influence of these variables on stability is necessitated. The resulting "yield deflection" is taken as a point where system instability occurs. These results for five load cases and three soil strengths are shown in Figure 1.

Because Cases B and C were deemed to be overly conservative, it is suggested that a 4 in. displacement at the tip of the pier could be an appropriate benchmark for signaling instability of the IRIB substructure. While this is a conservative recommendation, the unknown material properties at the site and other simplifications in the analysis are compelling reasons to further increase the conservatism of this recommendation if desired by the owner.

In the finite element analysis, prescribed support movements are introduced into the model and the resulting stresses are examined. Support movements in various directions are studied. Assuming linear-elastic behavior, these results can be scaled to determine the stress conditions under other magnitudes of displacement. Minor modifications to these analyses allow for considering larger displacements causing inelastic behavior.

The finite elements results give insight into the locations of highest stress as result of pier displacements, the corresponding magnitudes of stresses, and the type of displacements that may be most severe. It was found that the highest stresses typically occur in the girder webs over the outer piers. In general, the stress in the web at the outer piers appears to be more influenced by deflections in the direction parallel to traffic flow than the direction parallel to channel flow. It can be inferred from this data that a pier tipping in the traffic direction would generally be of greater concern than a tilt in the channel direction for the IRIB. The most critical case increases the average web stress by 5 ksi compared to the control case. The data from the analysis can be combined with future information from the tilt sensors to estimate how the scour conditions are affecting the stresses in the bridge.

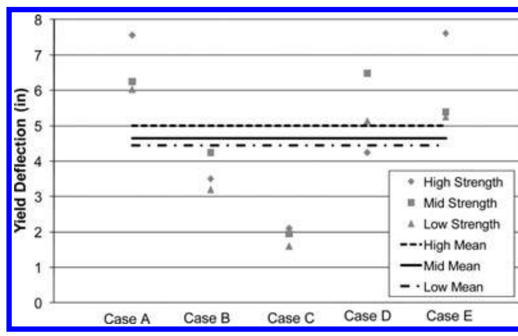

Figure 1. Substructure analysis results.

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Proof load testing supported by acoustic emission. An example of application

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ABSTRACT

The use of load testing in the assessment of existing bridges is justified by the fact that in many experimental loading processes in bridges, the structure responds with a higher capacity than expected based on the theoretical results from available numerical models. In situ tests show that sometimes bridges have a reserve strength that is not accounted for in design codes or standard assessment methods.

Three main possibilities exist based on the load level imposed to the bridge during the test execution: soft, diagnostic and proof load testing. In the present paper, proof load testing advantages and disadvantages are discussed and a practical way of bridge monitoring during the test execution is proposed.

The aim of the proof load test is to discover hidden mechanisms of response that can not appear under "normal" levels of load, but that develop at higher ratios of load and may increase the bridge load capacity. For this reason, in such test, the load introduced in the bridge is relatively high and due to the risks of damaging the structure, this type of tests is restricted to bridges that have failed to pass the most advanced theoretical assessment or when such theoretical assessment is not possible due to the lack of bridge documentation. The objective of this test is to directly obtain the maximum allowable load in the bridge with a required safety level.

Acoustic emission has been identified as a useful technique in the follow up of the loading process in proof load tests in order to stop the load increase before any damage can be inflicted to the bridge.

In the paper the results of field-test of Barcza bridge, a three span concrete bridge made of pre-stressed precasted beams, are presented.

The technique of incremental placing concrete slab layers and steel block layers was used during test (Fig. 1). The investigation range contained deflection, support displacement, strain, temperature and acoustic emission measurements and visual examination.

Figure 1. The technique of placing concrete slabs and steel blocks was used during test.

Thanks to the AE signals it was possible to evaluate the cracking limits without introducing any significant damage to the girders. The simple follow up of the deflection-load diagram or strain-load diagram as incremental loading is introduced in the bridge, stopping the test when some sign of non-linearity is detected, does not guarantee the possibility of not creating any damage to the bridge. In fact, in the case of Barcza bridge even after the detection of the cracking by visual inspection, the load-deflection diagram continued to be linear and no sign of change in the slope was detected.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Jointless bridge: Reliability assessment

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ABSTRACT

The performance of the jointless bridge, recently constructed in Austria, has been evaluated using stochastic nonlinear analysis incorporating the material properties obtained from experimental testing and artificial neural network based identification (Lehký et al. 2010). The computational model of bridge (Fig. 1) is developed using FEM software ATENA. Monitoring based analysis (Strauss et al. 2010) served for fine tuning of the FEM model. Stochastic model of the bridge is created considering material parameters and loading as random variables. Randomization is done using small-sample Monte Carlo simulation implemented in FReET software (Novák et al. 2009). Limit state functions for both ultimate capacity and serviceability are defined, theoretical failure probabilities and safety indexes are calculated, resulting plots in figures 2 and 3.

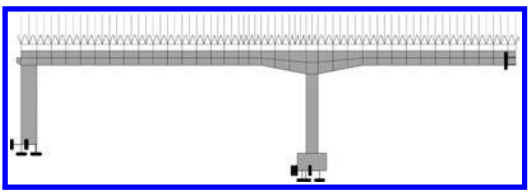

Figure 1. Half-symmetrical topology with constraints and continuous load (ATENA FEM model).

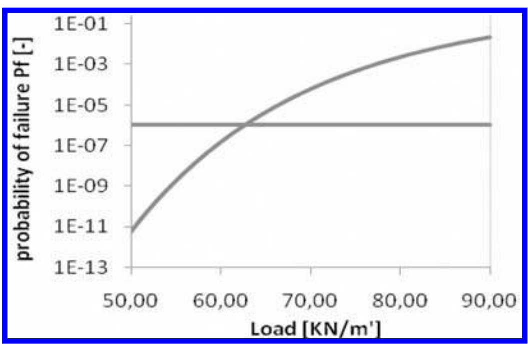

Figure 2. Probability of failure for the ultimate limit state.

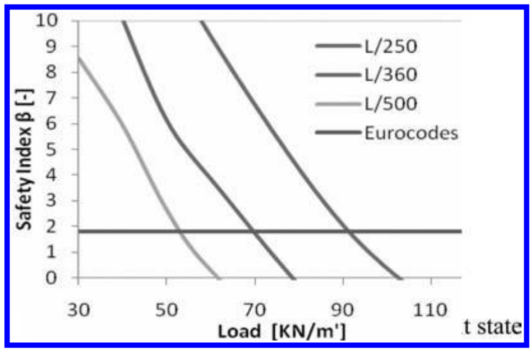

Figure 3. Safety index for the serviceability limit state.

Table 1. Selected randomized parameters of concrete.

Parameter	Unit	Mean	COV	PDF
Elastic Modulus*	[GPa]	39,5	0.10	N
Poisson's ratio	[–]	0,20	0.05	LN
Tensile strength*	[MPa]	2,90	0.09	Weibull
Compressive strength	[MPa]	28,9	0.10	LN
Specific fracture energy*	[N/m]	178	0.13	Weibull

ACKNOWLEDGEMENT

This outcome has been achieved with the financial support of the project No. MSM0021630519 from the Ministry of Education of the Czech Republic and the research project Eurostars E!4351 RLACS.

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An innovative approach for dynamic damage detection in bridge girders

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ABSTRACT

In recent years, the conditions of aging transportation infrastructure have drawn great attention to the maintenance and inspection of highway bridges. Therefore, a clear need for structural health monitoring exists for various types of bridges. In this paper, the effects of cracking damage on the dynamic characteristics of reinforced concrete bridge girders are investigated using a newly proposed method. The possibility of simulating equivalent single degree of freedom system through dynamic testing for in situ bridges is evaluated. Damage is considered as a reduction in the flexural stiffness with increasing degree of cracking. A vibrating motor with varying eccentricities is used for producing the dynamic harmonic loads. The variation in natural frequency, amplitude of vibration, damping and bending stiffness with increasing eccentric mass and for increasing degree of cracking are evaluated through steady state vibration near resonance at mid-span. The changes in the bending stiffness of girders in the mid-span with increasing degree of cracking are obtained. Damping values are calculated from free vibration decay function using logarithmic decrement method. Characteristics of cracked girders are simulated by using a beam model that consists of uncrack segments are connected by elastic rotational springs at crack locations. Analytical expressions for stiffness variations in girders are found as a function of cracks locations and damage intensity. The bridge girder itself is modeled by using the finite element method too. Excellent agreement is observed between the analytical expressions, and the numerical results. In summery, the paper presents a method for the calculation of stiffness of the cracked girders in the mid-span and then damage functions have been proposed in order to describe the possible damage pattern. By updating three parameters of n, w, and d, (i.e. the number of cracks, width of crack and depth of maximum crack) the damage patterns and their magnitude can be successfully determined. The authors' method is the alternative approach which is simply applicable to any kinds of simple girders, it does not require complicated solution steps, and it considers the crack morphology in a detailed way. The results obtained so far can only prove the assumptions made to be correct. The method is being extended at the present time, to more involved applications. The authors are planning to conduct their own experimental investigations.

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Definition of structural parameters span structures of bridges by results of their tests by mobile loading

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ABSTRACT

The operation of bridges is connected to periodic realization of difficult repair work, which are necessary for maintenance them in an efficient condition. The total cost of such repair work directly depends on existing opportunities on early revealing of all undesirable changes in designs. The important role in planning sequence of realization of repair work is played monitoring a condition of a structure during the certain time. Thus the definition of parameters of designs and tendencies in their change is of the important compound general process of diagnostics. Operational loading for the bridge is the mobile loading, therefore diagnostics of a condition of designs on their reactions just on mobile loading and is by the most rational approach.

The reaction of span structure of the bridge on travel of the automobile can be divided into two sites: first, when loading is on the bridge (active reaction – AR) and second, when it already has moved down (free reaction – FR) – a Figure 1.

On a site of active reaction fluctuations is the sum of two components: quasi-static reaction (QSR) and dynamic reaction to driven loading. On a site of free fluctuations there is only one component – dynamic reaction of free fluctuations. Allocating QSR it is possible to receive lines of influence of the given reaction, distribution of efforts between girders of span structure, dynamic load factor, and at known weight of the truck – experimental stiffness of span structure. Analyzing reaction on a site FR, we receive dynamic parameters of span structure: natural frequencies and them damping ratio. Having natural frequency of first mode and the stiffness of span structure is possible to define its mass.

The tests of bridges by mobile loading and analysis of its results according to the submitted recommendations are the effective approach at research of bridges. Analyzing the compelled and free fluctuations, and also comparing them with results of accounts FE-model, it is possible to define static and dynamic parameters of span structure. All this parameters further are used for monitoring of the bridge or additional information deviation from the project. The technique is well fulfilled in practice and is simple in performance. The real example of realization of test according to the specified recommendations is considered.

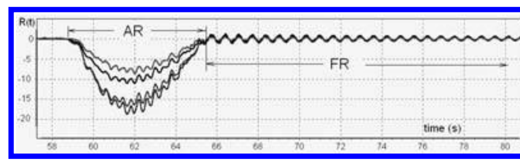

Figure 1. Response (deflections) of girders of the bridge on movement of the truck.

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Repair monitoring and experimental work associated with Ferrycarrig Bridge

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ABSTRACT

This paper presents details of an onsite bridge management project and an experimental study aimed at examining the relative efficiencies of various repair methods in resisting the ingress of chloride ions, which lead to reinforcement corrosion. Details of the repair and instrumentation of a marine bridge in the South East of Ireland are presented. The latest results from the onsite monitoring system are also presented.

The bridge in question is Ferrycarrig bridge which carries the N11 single carriageway over the River Slaney in Wexford. Built in 1980, the 125.6 m long structure consists of 8 spans of precast, prestressed beams with a reinforced in-situ concrete infill deck. In August 2002 an inspection of Ferrycarrig bridge identified extensive cracking on the bridge's crosshead beams and South abutment. Following a structural analysis and a more detailed inspection a decision was made in 2007 to repair Ferrycarrig bridge. The rehabilitation works afforded a unique opportunity to gather information regarding the efficiency of typical alternative concrete repair options in Irish marine environments. It was therefore decided to utilise five different concrete repair strategies for the seven crosshead beams. Six crossheads would be instrumented and remotely monitored so that the relative efficiency of the various methods could be studied over time. The repair methods employed were as follows:

- OPC with standard 50 mm cover
- OPC with increased cover (70 mm)
- 60% GGBS as partial replacement in OPC mix
- OPC with mixed in corrosion inhibitors
- OPC with silane treatment

Each of the crossheads was instrumented with corrosion potential probes, corrosion rate probes and chloride ion penetration depth probes. Detailed results of the output from the monitoring system to date are presented in the paper. At this early stage in the probe monitoring (just 22 months in) the long-term benefits of each repair strategy cannot be determined due to the fact that the responses observed may be to a greater or lesser degree as a result of the extensive disruption to the crossheads during the repairs. However, the patterns of the probe data can be observed and interpreted to give an indication as to what is happening within the crosshead beams.

A notable rise has been observed in the corrosion potential probe readings over the past 22 months. This has resulted in a shift in the corrosion risk classification from medium risk to low risk. This would indicate that the crossheads are stabilising with time.

Four of the six corrosion rate probes are displaying higher rates of corrosion than would be expected at low or medium risk of reinforcement corrosion. These readings are likely to be due to the initial corrosion which was occurring in the steel when the concrete for the crosshead beams was placed. In general the readings show a drop in the rate of corrosion penetration over the period of December 2007 to October 2009. It is likely that this drop is due to the progression of the re-passivation process. The chloride depth probe readings show very low chloride contents at the level of the reinforcement as would be expected following the crosshead beams' extensive repairs.

The paper also presents details of an experimental study which is being undertaken at Trinity College Dublin in parallel with the Ferrycarrig bridge monitoring project. The initial phase of testing, which is concerned with the time to initiation of corrosion, is currently underway. The testing is aimed at establishing the ability of six repair methods to prevent the ingress of chlorides into concrete. The six repair methods being examined are those used at Ferrycarrig bridge, with the addition of a 30% pulverised fuel ash partial replacement mix.

In the coming months the laboratory aspect of the project will provide comparative information on the relative merits of the concrete repair strategies in resisting the ingress of chlorides.

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Analysis of in-service data collected during biennial inspections on typical bridges

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ABSTRACT

A program has been underway in recent years to collect in-service strain response data from a number of typical highway bridges in Delaware. This has been done in conjunction with the regularly scheduled biennial inspection. Strain data is captured using the "In-Service Bridge Monitoring System" (ISBMS), which is usually mounted near mid-span of the most heavily loaded girder. The system monitors 24/7 for a period of about two weeks, and records only the peak strain induced by heavy vehicles. A simple histogram or timeline plot summarizes the results and provides a quantitative record of how the bridge is performing under load. ISBMS data has been collected from a sample of twelve slab-on-steel girder bridges in the state. The first data sets were obtained in 2006 and 2007; new data sets will be collected in 2009 and 2010. Presented is a summary of the results of the data collected thus far. Once the new data sets are collected comparisons will be made between the first and second sets. Differences could be due to normal variability, changes in traffic, or changes in the condition of the bridge. Also discussed is how the normal variability will be estimated, assessment of changes in traffic, and steps to be taken if changes can be attributed to changes in the condition of the bride. It is this type of data that can become a part of the permanent bridge inspection record, and can be used to better manage and maintain the state's bridge inventory.

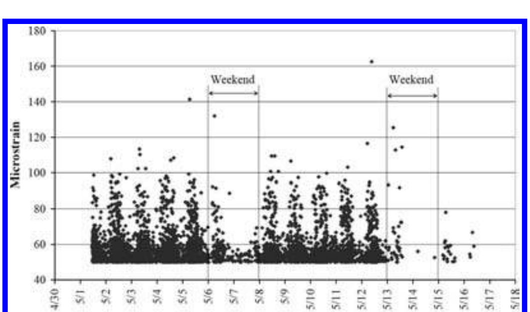

Figure 1. Timeline plot of recorded strain for bridge 1–826.

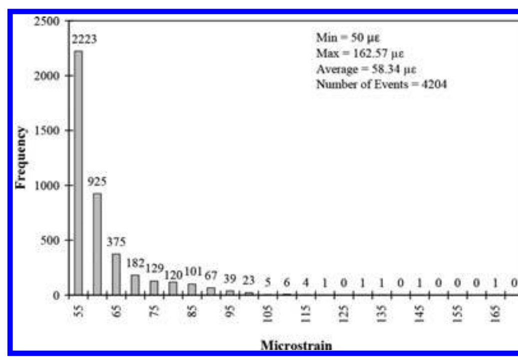

Figure 2. Histogram of recorded strain for bridge 1–826.

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Determination of concrete bridge ageing by structural health monitoring

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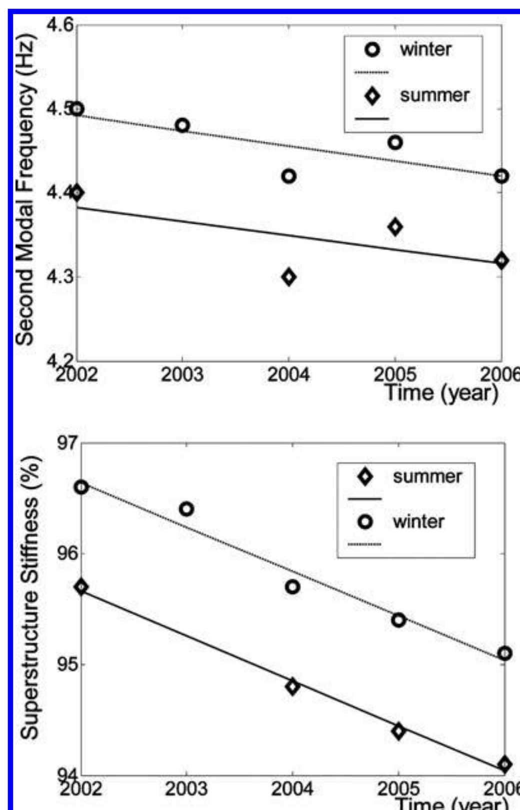
ABSTRACT

Change in the stiffness of a concrete bridge was determined by vibration-based monitoring over 5 years. Jamboree Bridge is located in Orange County, CA. This three span 111-m long bridge is instrumented with 13 acceleration sensors at both the superstructure and the columns. The sensor data are transmitted to a server computer wirelessly. The modal frequencies and the shapes were identified by processing traffic induced excitations. Bridge structural parameters, stiffness and mass, and the soil spring values were identified utilizing the neural network technique. The identified modal frequencies vary within +/−10%. The identified stiffness of the bridge deck varies within +/−3%. These results are shown in the following two figures. Based on the statistical analysis of the collected data for each year, 5% decrease in the first modal frequency and 2% decrease in the stiffness of the bridge deck were observed over the 5-year monitoring period. Probability density functions were obtained for stiffness values each year. Stiffness threshold values for the design life of the bridge deck under the operational loading can be determined. Therefore, the information obtained in this study is valuable for studying ageing and long-term performance assessment of similar bridges.



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Monitoring based performance checking of the jointless Marktwasser Bridge S33.24

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ABSTRACT

In bridge engineering maintenance strategies are highly influenced by construction type and quality of design. Nowadays bridge designer and owner tend to include life-cycle cost analyses in their decision processes regarding the overall design by trying to optimize structural reliability and durability within financial constraints. However efforts to reduce maintenance costs over the expected lifetime by adopting well established design principles leads to unknown risks. Monitoring solutions can reduce this associated risk of new designs by a permanent observation of performance of structural components during prescribed time periods.

In this research monitoring is applied to the jointless Marktwasser Bridge, with emphasis on assessing the effects of time dependent processes including creep, shrinkage and temperature. The Marktwasser Bridge S33.24 is a three span bridge over the Danube in Lower-Austria, Austria, with 19.50 m, 28.05 m and 19.50 m spans, see Fig. 1. The bridge is designed for the five lane highway S33 according to the national standard B-EN1992 for reinforced concrete bridges. In order to reduce construction and maintenance cost, the Austrian Highway Financing Company ASFINAG instructed the engineering offices to convert the original design to a jointless bridge structure by omitting expansion joints and bearings.

A fiber optic monitoring network has been commissioned in the first lateral south western field of the Bridge S33.24, in order to capture effects due to (a) the consecutive concrete pouring of the bridge fields, (b) the removal of the formworks, (c) the pavement loading, (d) the variability in the environmental temperature, (e) the creep and shrinkage processes in the first three years, and (f) a proof loading procedure which is planned in Spring 2010. In addition, numerous concrete samples were extracted during the casting procedures in January 2009. These samples served for the determination of the concrete strength and its associated fracture mechanical properties and provided essential information for the initial inverse analysis of the static system. Finally, a probabilistic approach was used to assess the influence of uncertainties on structural response (load-carrying capacity).

The current investigations demonstrate a very good identification of time dependent mechanical and chemical (e.g. creep and shrinkage) processes and a continuous observation of the structural loading response by the installed permanent monitoring system that consists of several different sensor types.

Further research in this area will include the application of the methodology to the evaluation of the soil structure interaction, and the calibration of finite element models among others.

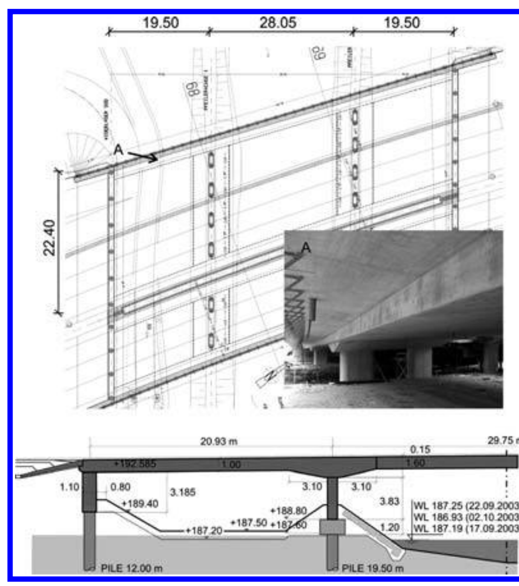

Figure 1. Marktwasser Bridge, S33.24.

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Monitoring based verification of the soil structure interaction of the Markwasser Bridge S33.24

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ABSTRACT

In structural bridge engineering maintenance strategies and thus budgetary demands are highly influenced by the quality of design in general as well as the chosen construction type in particular. Nowadays bridge owners and planers tend to include life-cycle cost analyses in their decision processes regarding the overall design trying to optimize structural detailing with respect to robustness, durability, exchangeability of certain structural members and reliability (Frangopol et al. 2008, Strauss et al. 2009). However efforts to reduce maintenance costs over the expected lifetime by adopting well established design principles leads to unknown risks concerning for instance boundary conditions.

Monitoring solutions can reduce the associated risk of new design concepts by constant supervision of critical structural characteristics and in combination with data analyses allows for the verification of new numerical and analytical models. This paper especially focuses on the layout and analysis of monitoring data obtained by an integrative multi-sensor-monitoring system targeting the soil-structure interaction of frame bridges.

Based on an application example of a three-span frame bridge with a total length of 67 m different aspects of the design such as build up of earth pressure against the abutment wall, the bedding of the drilling piles, the functionality of the chosen slab detail to accommodate changes in length caused by temperature loads in combination with the respective regulations in national and international standards were to be evaluated. To that end four different sensor systems were installed behind the southern abutment, consisting of 2 extensometers, 20 electrical strain gages, 10 fibre-optic strain sensors and 3 vertical inclinometers. Figure 1 shows the installation of the top fibre-optic sensor layer. Additionally 20 fibre optic strain and temperature sensors were placed in the deck of the southern span.

Finally the installed monitoring system will allow the calibration of suitable finite element models such as shown for instance in Figure 2. Those models then can serve for the analysis and further optimization of structural details and thus ensure efficient and safe design.

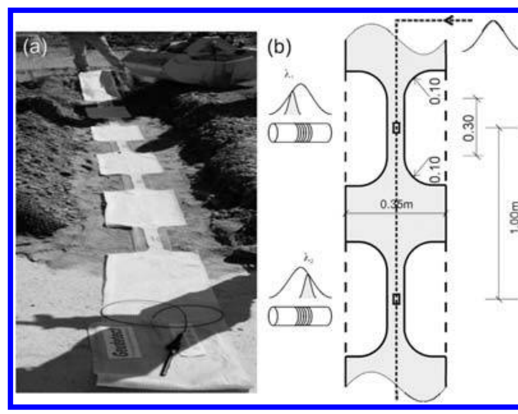

Figure 1. Installation of (a) top fibre-optic sensor layer in sand bed, and (b) sensor principle, and geometry of geotextile layer.

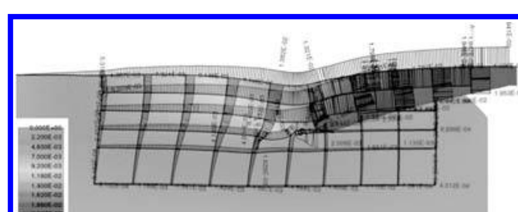

Figure 2. Total horizontal deformation in the dilatation area between abutment and earth dam due to contraction of the structure by 20 mm (winter).

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Experimental trials on the detection of reinforcement breaks with the magnetic flux leakage method

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ABSTRACT

Hardly any fatigue breaks of reinforcing bars have been reported so far. Reasons may be that fatigue breaks are not actively searched and that they are hard to detect by non-destructive testing methods. The magnetic flux leakage (MFL) method can fulfill this gap. The applicability and the limits of the MFL method will be investigated in connection with large-scale fatigue tests.

The MFL method is based on the physical fact that a break in a steel bar leads to a change of sign in the magnetic flux density, which can be measured using hall sensors. Prior to the measurement, the steel bar has to be magnetized with an external magnet that will be moved alongside the steel bar. Figure 1 shows 4 aligned disc magnets besides a reinforcing bar. The sensor (Fig. 2) has also to be moved alongside the reinforcing bar to measure the longitudinal and radial component of the magnetic field.

In preliminary tests first experimental trials on single unbonded steels bars and concrete specimens with predefined reinforcement layouts including breaks in reinforcing bars have been executed.

The measurements of the magnetic flux density were recorded and compared with theoretical considerations. It could be found that the data correlate well.

In a second step, measurements with the MFL method are prepared for application on a reinforced concrete frame which will be loaded cyclically. Breaks of the reinforcement are expected to occur in the frame corners. Acoustic emission is used to determine the time of break. Once a break is assumed, loading is halted and the measurements with the MFL method are performed.

After the experiment has been finished the reinforcement will be uncovered and the positions of the measured breaks will be compared with the effective ones. Due to the much denser reinforcement layout of this specimen the data interpretation of the measured magnetic flux values will be more complicated as in the preliminary tests. Evaluation of the data will give a first impression of the capability of the MFL method.

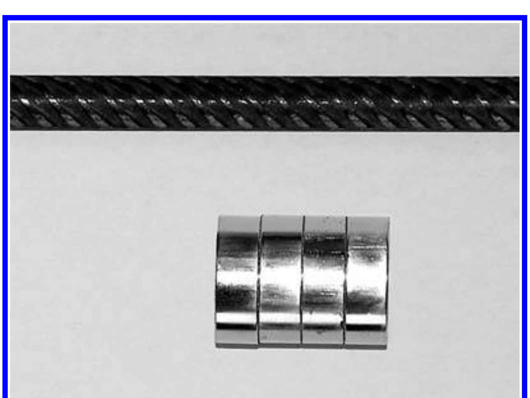

Figure 1. Permanent magnet besides a reinforcing bar.

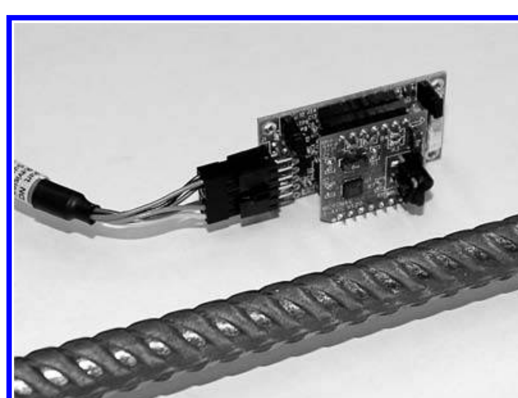

Figure 2. 3-axis sensor module besides a reinforcing bar.

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Innovative structural health monitoring for Tamar Suspension Bridge by automated total positioning system

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ABSTRACT

This paper presents an innovative structural health monitoring system using an automated Total Positioning System (TPS) and its application to Tamar Suspension Bridge. Conventional TPS also known as Total Station used for surveying have evolved to have a functionality of automated and unmanned operations. For SHM purposes, an automated TPS can be utilized for monitoring 3D deflections of a structure.

Figure 1. Automated TPS: Leica TCA1201M and Reflector.

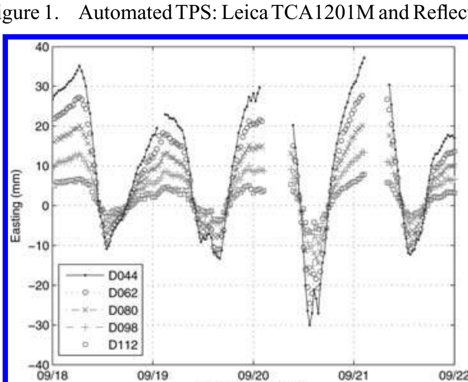

Figure 2. Easting deflections.

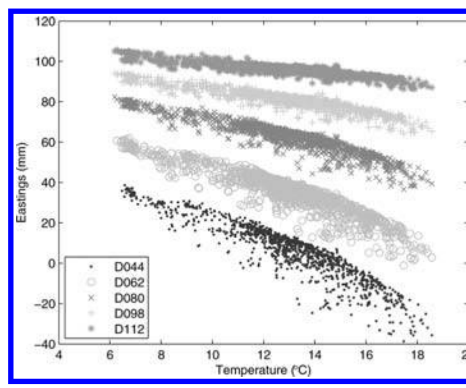

Figure 3. Temperature vs. Easting deflections.

Tamar Bridge, Plymouth, UK has been equipped by an automated TPS on the roof of the bridge office for monitoring of the bridge deck and tower to understand complex behavior of the suspension bridge system under environmental loadings such as temperature, wind and traffic loads. From Sept 2009, 15 points around the bridge deck and towers have been monitored and the global deflections were recorded. Based on the three months monitoring records, temperature change was found to be the dominant contributor of bridge deflections in the longitudinal and vertical directions while the transverse deflections are not understood yet. Further analysis on the deflections is underway. Limitations of the current technology were discussed.

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MS3: Present & future of bridge inspection & evaluation
Organizers: S. Alampalli, A.K. Agrawal & M. Ettouney

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Inspection needs of deteriorating bridge components

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ABSTRACT

Currently, about 25% of highway bridges in the United States are considered either structurally deficient or functionally obsolete. Even though these deficient bridges are safe for the legal loads allowed on them, they are in need of corrective maintenance, repair, rehabilitation, or replacement to bring them to current standards with no load restrictions or inconvenience to the travelling public. This number is expected to increase due to limited capital funds available to reconstruct these aging bridges. In such a situation, an inspection program that can effectively address both safety and performance needs is required.

The deterioration of bridge elements is influenced by the combined effects of many complex phenomena (e.g., corrosion, concrete degradation, creep, shrinkage, cracking, and fatigue). In the absence of mechanistic-based models that require quantitative contribution of these complex phenomena, a model using bridge inspection data may be used to estimate deterioration of bridge elements.

This paper at first presents a review of national bridge inspection standards and data collected through bridge inspections. This is followed by the review of gaps and needs presented by the recent ASCE/SEI-AASHTO ad hoc group, formed in the aftermath of the recent Minnesota bridge collapse, that discussed the bridge inspection and rating practices to identify the gaps, needs, and issues associated with current practices and policies for the condition assessment of the bridges. Finally, the paper gives a brief description of two studies, authors are involved with, meant to improve bridge inspection and management practices. The first study is to evaluate the reliability of the New York State highway bridge inspection process and the second study is the development of deterioration curves for bridge components based on inspection data.

A Federal Highway Administration (FHWA) study completed in 2001 reported that bridge inspection ratings on a national level have high variability and questioned the quality and reliability of visual based inspection procedures. Due to the methodology used, these results may not be applicable to reliability of the bridge inspection program of individual states. For example, the NYS bridge inspection program is more robust and detailed than what is mandated by FHWA. Thus, a research project has been initiated to quantitatively document the variability associated with the bridge inspection program, suggesting improvements to policy and procedures and areas requiring further training, if needed, to reduce this variability and improve the reliability/consistency of the program. This paper gives some background details and the scope of the on-going project.

A new approach using Weibull distribution has been presented to calculate deterioration rates based on the historical bridge inspection data for highway bridges in New York. Advantages and capabilities of the Weibull-based approach are demonstrated by three cases comparing Markov Chains and Weibull distribution-based approaches. These rates can be used for estimating remaining life, comparing alternatives for repair and retrofits, and analyzing effects of factors on deterioration of bridge elements. These can also be used to select or customize advanced assessment techniques during the life-cycle of the structure to augment visual inspection and to collect data useful for bridge management functions.

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Periodic NDE in support of structural health monitoring of bridges

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ABSTRACT

In the last two decades, the U.S. Federal Highway Administration (FHWA) invested significant effort into the development of nondestructive evaluation (NDE) tools that may be suitable for an objective condition assessment of bridge structures. Parallel developments explored the concept of structural health monitoring (SHM) and its application to constructed systems – especially bridges. Generally there is a wish to have a wider utilization of the various technologies that have been developed in the last two decades and to make a transition from the current practice of highway and bridge engineering to one that is based on objective measurements and the lifecycle benefit/cost of the system. Accordingly there is a need for strategies to incentivize and facilitate the application of these technologies.

The strategy presented in this contribution is supplementary to the global SHM; it can be regarded as a practical and less costly implementation for tracking mission critical performance for much of the nation's 600,000 bridges. It focuses research and development on techniques to facilitate quality control and periodic lifecycle applications of measurements including periodic and multi-sensor NDE measurements with the following combined strategy:

- Multi-sensor NDE to improve reliability of data
- NDE for structural condition assessment to the level of detail to perform any bridge maintenance, preservation and rehabilitation type actions – use of periodic NDE as a strategy for bridge preservation, as opposed to the standard concept of NDE inspection for safety improvement (Jalinoos et al. 2009).

Thus periodic inspection can be a part of the regular biennial inspection or performed on-demand in response to an extreme event such as hurricane, storm, earthquake, blast, collisions, fire, and superload.

Examples using periodic GPR for SHM of bridge deck deterioration, detection of ice (compare Fig. 1), and rebar corrosion are presented. Furthermore a

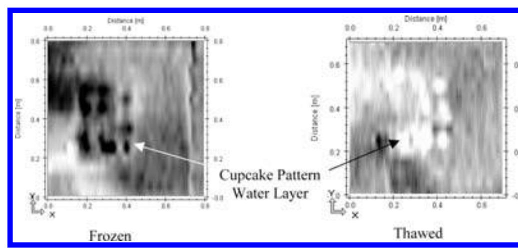

Figure 1. Ground penetrating radar measurements with 2.3 GHz handheld antenna on concrete slab specimen with frozen and thawed cupcake pattern water layer.

benchmark approach project (Arndt & Jalinoos 2009) using and evaluating state of the art NDE methods for SHM of bridge decks is outlined and first results documented. These are already very promising in the laboratory but its final application in the field is very challenging; a periodic multi sensor NDE for SHM requires a high standard of repeatability for data comparability and advanced processing, which means a high level of automation is required (Huston et al. 2008).

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To see is to know: visualization in bridge inspection and management

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ABSTRACT

A naturally designed user interface, especially in the engineering software systems, innovates the way people work, improving their efficiency and having a positive impact on the whole branch. Today, a rapid development of computer graphics hardware allows to model software systems as interactive real-time visualizations, even when using affordable mid-range machines.

Among many software genres that this fact makes evolve quickly, there are Bridge Management Systems (BMS). A demand for better user experience in BMS yields migration from traditional data forms into model-centric interfaces. In model-centric systems, most user operations are based on real-time interaction with a visual model of the infrastructure, represented in a convenient level of detail, both spatial and temporal. The same structure, or a part of it, can be represented in a variety of aspects, depending on the current context in which the user is working at the moment. All geometrical, physical, and chemical properties of the structure, as well the extents, intensity, and specifics of existing or predicted damages, can be shown exactly where they exist in a real world – or where they can be expected, accordingly.

The paper presents diverse types of interactive visualizations that can be applied into certain areas of BMS, in diverse phases within the lifespan of a structure. For example, when a bridge is already in operation, the used representations and models often do not need to be as detailed as during the design, which reduces complexity and requirements for the computational power. The discussed solutions include: component-based modeling of bridge inventory information, visual recording of damage data during an inspection, and representing predictions of technical state changes in dynamic environments. Visualization examples from the finalized projects are presented, as well as several future prospects in the field.

For the BMS, we propose a classification of visualization techniques based on a variability of the image over time. The static category includes invariable elements such as maps, photograph, and non-adaptive drawings and models, and the dynamic category includes elements that change over time, such as films, TV, adaptive drawings and models, and variable traffic signs.

Proposed classification of the geometry models is presented in Figure 1. The columns represent the dimensionality of space used for visualization, from a single point to the full three dimensions, accordingly. The rows indicate the dimensionality of models that can be represented in the corresponding spaces. A notation for the dimensional level of detail has been defined as $e^x s^y$, where x is the dimensionality of elements, and y is the dimensionality of space.

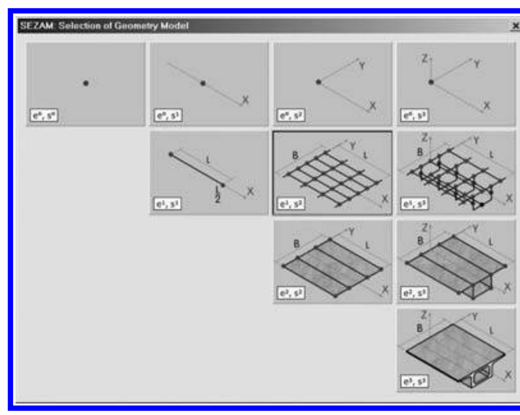

Figure 1. The geometry model matrix (Bień 2002).

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Knowledge-based expert tools in bridge management

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ABSTRACT

The paper presents a procedure of expert tools creation by means of multi-level hybrid networks together with some practical applications of the pilot versions supporting bridge management. An expert tool can be based on various components dedicated to specific techniques of symbolic or non-symbolic knowledge representation. Selection of a representation technique depends on the form and content of the available knowledge. In the presented applications, the techniques of neural networks, fuzzy logic and multi-dimensional mathematical functions are involved in the process of knowledge representation.

For creation of each individual component and for configuration of the final network, the specialized software system NEURITIS was developed. Details of the system were presented by Bień & Rawa (2004). In this system the hybrid network can be created – depending on the problem that should be solved and on the type of available information – by means of the following components:

- neural component, based on non-linear multi-layer artificial neural networks (ANN) trained using the supervised back-propagation method;
- fuzzy component, based on the fuzzy logic with the ability of fuzzy inference;
- functional component that enables implementation of the analytical functions.

Experience in design, testing and preliminary applications of the following expert tools is presented and discussed: "Bridge Evaluation Expert Function" (BEEF) supporting assessment of railway bridge condition (Fig. 1), expert system "Masonry Bridge Damage Evaluator" (MyBriDE) for evaluation of load capacity of damaged masonry arch bridges proposed by Kamiński (2008) and conception of the expert system "Concrete Bridge Degradation" (CoBriDe) supporting forecast of the degradation processes of concrete bridge structures.

Preparation of network components and the creation of practically effective networks is a very time-consuming process, engaging civil engineers, knowledge engineers, computer scientists, etc.

The proposed expert tools enable easy composition of data and knowledge accumulated in the BMS and ensure uniformity of the decision procedures within the whole management system. Utilization of the expert tools is user-friendly and enables even the users with limited computer experience to analyze and successfully solve the complex bridge engineering problems arriving in bridge management.

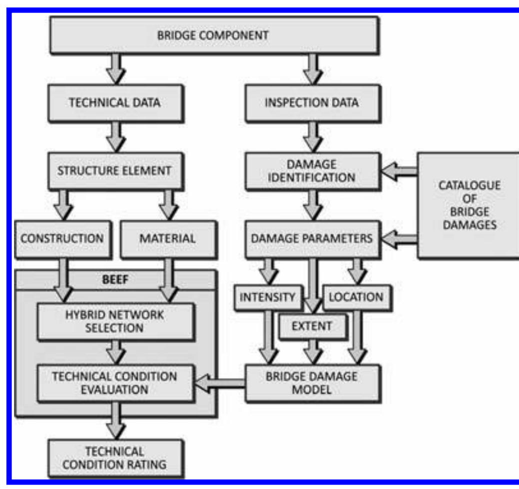

Figure 1. Technical condition rating in the RBMS "SMOK" supported by the Bridge Evaluation Expert Function (BEEF).

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Enhanced bridge management via integrated remote sensing

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ABSTRACT

Commercial remote sensing (CRS) as robust health monitoring techniques offer unique features that are lacking from current embedded structural health monitoring systems, including geo-referencing, spatial view and high-resolution top view. Beginning in 2007, a research partnership (University of North Carolina at Charlotte, ImageCat Inc., Boyle Consulting, Charlotte DOT and North Carolina DOT) has completed a proof-of-concept project to develop ground-based LiDAR scan and sub-inch resolution aerial photography into the IRSV (Integrated Remote Sensing and Visualization) bridge data diagnostic platform. CRS technology for structural health monitoring differs from conventional technologies in that the measurements are from a distance away from the object without physically touching the object.

The terrestrial 3D laser scanners, operating in the narrow near infrared wavelengths and with scanning capability, can generate dense point clouds of bridge geometric data. The manipulation of the 3D geometric data allows damage detection and quantification of surgical damages to a structure. Applications of the scanning LiDAR also include deflection measurements during load testing. A 3 mm resolution of displacement measurements can be achieved using LiDAR. The results are also ideal for FE model validation, which has been demonstrated using a bridge model for a steel-girder bridge.

The use of geo-referenced, high-resolution (sub-inch) aerial photography and automated imaging allows surface mapping of the wear surface conditions. To achieve the high-resolution imagery, the Small Format Aerial Photography (SFAP) technology is adopted along with onboard GPS system to establish geo-referencing and geo-stamping of all imageries taken. Application of SFAP is demonstrated in a bridge rating technique solely based on crack detection capabilities of the high-resolution SFAP.

Issues that limited the application of CRS-GSI technology to conventional bridge monitoring have been identified as: 1) limited acceptable bridge inspection procedure; 2) inconsistency in bridge management styles; 3) lack of in-depth understanding of CRS-GSI technologies; and finally 4) complexity in multivariate data integration. Because CRS data exist in image format and bridge data exist in PDF or text-file formats, there is the challenge in integrating both data into BMS, so that the bridge managers can visually study the combined or "fused" data and other information.

An IRSV system is proposed that would allow the fusion of different remote sensing data with existing bridge data into a multi-variate viewing platform. IRSV represents a critical juncture towards the "Total View Integration (TVi)" infrastructure monitoring concept, which is defined in this paper, and provides further incentives for CRS development. This paper presents an overview of some technologies adopted in IRSV and discusses the potentials of CRS tools to enhance bridge inspection and data management.

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Full field mapping of bridge deformation using digital speckle photography

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ABSTRACT

Many of our country's bridges are aging structures that require periodic and frequent inspection. Currently the prevailing mode of inspection is visual inspection by human. Not only this is a tedious and time consuming process, inspectors with different background and training may render different conclusions. Inspection using instrument such as thermal imaging can only inspect a small region at time. In this paper, we present a new technique whereby the entire bridge deformation pattern can be mapped simultaneously in a single step. The new technique called digital speckle photography [Chiang, 2003] does not need expansive equipment and is easy to set up. And the data can be obtained in semi-real time.

A speckle pattern is a collection of random dots or particles that show up distinctively when photographed. They can be used for quantitative measurement of displacement and strain. In this study, we built a 10 ft by 1.25 ft truss bridge using commercially available pine wood planks. One surface of the wood bridge is painted with a thin coat of retro-reflective paint which has imbedded minute glass particles. As a result when a light beam illuminates the paint surface most of the light is reflected back to the light source. Two 200 w flood light are used to illuminate the bridge. They are situated at 36 ft away from the bridge and on either side of a digital camera. The bridge is loaded evenly with 50 lb sandbags totaling 600 lbs. The painted surface of the bridge was photographied before and after the application of load. Each digital image of the bridge is subdivided into subimages of 128 × 128 pixel arrays and then "compared" using a specially design software call CASI. The result is a displacement vector map of the entire bridge. The deflection of the bridge along the entire span is compared with finite element result as shown in Fig. 1. If local strain distribution is needed, the camera can zoom into a small region and calculate the detailed strain distribution. The technique can also be used in dynamic situation in which the bridge is oscillating (Chiang, 2009).

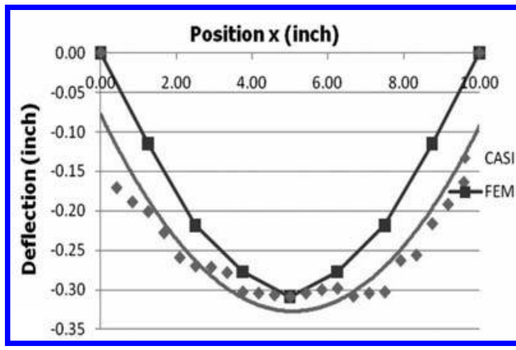

Figure 1. Bridge main beam deflection from CASI and FEM.

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Design for inspection – one way towards durable infrastructures

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ABSTRACT

New materials developed in the last three decades as well as the significant advances recorded in engineering and computering are currently enabling us to figure out new infrastructures based on new design, new materials and challenging smart structures.

In the field of Civil Engineering, the design of new infrastructures like bridges and roads must allow ease of structural health monitoring, as one of the main goals. To achieve this, an appropriate design procedure must be implemented, including several complementary research topics : Materials science – Mechanical Engineering – Instrumentation (NDT and SHM, computering). In addition, given the new rules governing the politics worldwide, new infrastructures must meet sustainable development requirements.

This work will address the points listed above in the case of bridge infrastructures.

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Non-homogeneous Markov Chain for bridge deterioration modeling

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ABSTRACT

Bridge components or elements deteriorate with time. Deterioration modeling is an important step in bridge management operation. In the popular bridge management system Pontis, a Markov Chain model is used to describe bridge element deterioration. The core part of this model is its transition probability matrix, which describes the deterioration process using a probability measurement for the likelihood that the element deteriorates (transits) from a condition state to another. Namely, those elements that deteriorate faster are assigned higher transition probabilities from a better condition state to a worse one. It is interesting though to point out that this approach does not take into account the age of the element. In other word, an older bridge element is assigned the same transition probabilities from better conditions to worse ones as a much younger one. This model is thus referred to as a homogeneous Markov Chain. In addition, due to typically long life span of bridge elements in decades, small inaccuracy in transition probability matrix estimation for short time intervals (a few years) can cause large errors in the final result of service life prediction.

This paper proposes a new model of non-homogeneous Markov Chain to take into account the element's age, with detailed elaboration on the issues discussed above. Namely, the transition probability matrix is now age-dependent not constant for all ages. Therefore, multiple such matrices will be needed to model the entire life span of the bridge element's deterioration process. Given inspection data of condition state evolution, a fitting scheme is also suggested here to estimate the non-homogeneous transition probabilities depending on age. In addition, a new screening scheme is proposed in this paper for initial data screening to filter inconsistent raw data. Application of the proposed model to Michigan bridge inspection data shows improved modeling results. This approach was also applied to the bridge condition rating data in the National Bridge Inventory format and thereby produced more realistic predictions.

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New tools for inspection and evaluation of steel truss bridge gusset plates

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ABSTRACT

Gusset plate connection performance has become an important issue for many transportation agencies after the collapse of the I-35W Bridge in Minnesota, USA. Due to this failure, steel truss bridges are undergoing additional scrutiny focused on the gusset plate connections. Connection evaluations require accurate as-built drawings and typically use simplified specification-based design methods for analysis.

A new methodology has been developed that permits collection of accurate field measurements, extraction of data to production of as-built CAD connection drawings and direct finite element (FE) modeling and analysis of as-built connections. The method uses close-range photogrammetry techniques to rectify field-collected digital images taken with consumer-grade cameras to produce scaled orthographic photographs (orthophotos) of the bridge connections as illustrated in Figs. 1 and 2. From these orthophotos, the user collects true-scale measurements and exports the plate and fastener geometric information. The geometric data are used directly to create CAD drawings thereby limiting data entry errors. These as-built drawings can be compared with design drawings or used in simplified analysis and ranking methods. The same geometric data are ported to ABAQUS using scripts that generate finite element models of the gusset plates. When combined with member forces applied at the fastener locations, the FE analysis can be performed (including buckling and nonlinear analyses), and the results can be used to rate the connections.

The approach enables rapid and accurate collection of field measurements and allows direct assessment of the connection behavior as compared to traditional simplified methods. The implementation procedure is straightforward and does not require specialized knowledge of photogrammetry or finite element methods. It can be practicably employed under field conditions using current technology and personnel. Integration of the techniques allows seamless data flow and sophisticated evaluation of the complex plate stress interactions to enable bridge ratings of existing steel gusset connections and represents a new era for quantitative inspection of transportation infrastructure.

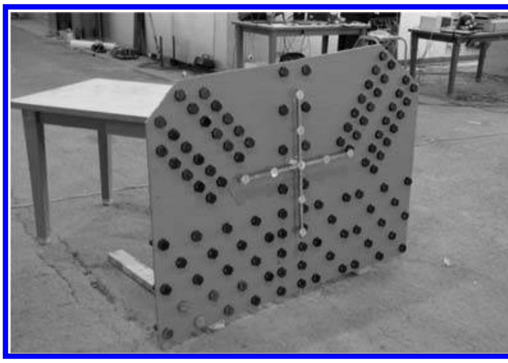

Figure 1. Original image of mock gusset plate.

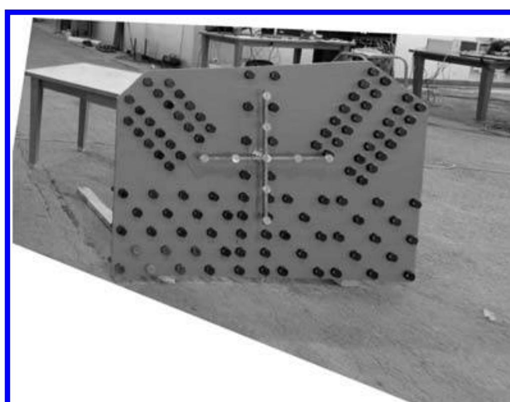

Figure 2. Orthophoto that allows metrification of connection geometry.

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The computerized cables stay inspection of the Cooper River Bridge (Arthur Ravanel, Jr)

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ABSTRACT

The Ravenel Bridge is a 4km cable stayed bridge which opened in July 2005. It connects downtown Charleston, SC to Mount Pleasant, SC. Freyssinet, LLC (FLLC), supplied and installed 128 parallel strand stay cables supporting the structure. The original contract included a warranty inspection every 2 years for 10 years after the bridge opening, the first inspection occurred in October 2007.

A main objective was to reduce time spent on the inspection process especially on report generation, Advitam's Inspection software: ScanPrint was used by FLLC for this purpose. Another important objective was to make the information easy to access and to further facilitate efficient and accurate inspections in the future.

ScanPrint was initially developed in 1999 for the Inspection and Maintenance management of the Vasco de Gama Bridge in Portugal. It is presently in use on many large cable stay bridges, large suspension bridges and several landmark structures.

The first step of the project for Advitam was setting up the inventory of the bridge and computerizing the inspection manual.

The inventory is entered by breaking down the bridge into a tree view of components. The break down is made according to the bridge structural types and in a manner consistent with the way inspections are planned to be performed. This concept is fundamental to ScanPrint. Once the inventory has been entered, no additional data entry will be required for future inspection and only a few steps are necessary to start the next inspection.

After entering the inventory, the inspection manual can be computerized. For this type of unique structure, the standard NBI inspection forms or AASTHO components do not adequately address the particular issues associated to cable stayed systems. A more detailed and specific inspection manual is required. FLLC provided a recommended inspection & maintenance manual defined for the Ravenel Bridge, however, it was written at a high comprehensive level. Advitam worked with FLLC to simplify the questions given to the inspectors. Finally, lists of defects with attributes were entered.

After brief training sessions off and on-site, each team of inspectors were given a Tablet PC loaded with ScanPrint software. They were also equipped with digital cameras and measuring tools. Following the inspection plan, the teams then visually inspected the bridge structure filling out the forms along the way; and if necessary drew defects and attached pictures directly on the Tablet PC.

Once the on-site inspection was performed, the Tablet PC databases and files were merged together. Advitam worked with FLLC to define the inspection report. Two report templates were implemented:

- Word report: Containing the information gathered on the field: AutoCAD drawings with the defects, defect information, inspection forms, results and pictures.
- Excel Report: Containing only the results of the inspection forms, it allows quick and easy comparison of results between elements.

A draft report was first generated in November allowing SCDOT to send comments prior to issuing, the final report in December.

The Ravenel Bridge inspection proved to be a perfect project to use the ScanPrint software. The visual inspection of multiple similar elements makes the data even more relevant to compare to each other. Beside the time saved during the report generation, full value of the computerized inspection will be unveiled during the second inspection, and so on. More and more data will be entered, and therefore can be compared and analyzed; each cycle is adding more value to the database.

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Load ratings of a concrete-encased steel 3-hinge arch bridge

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ABSTRACT

A concrete-encased steel arch bridge was rated using both the ACI and AISC criteria because the steel ratios are within the applicable range of both criteria. Figure 1 shows the P-M interaction strength curves and their relationships with the factored loads following both ACI and AISC procedures. The AISC procedure results in more conservative rating factors because of high dead load and live load axial force to moment ratios. The HS20 loading inventory ratings are 3.15 and 3.94 based on the AISC and ACI procedures, respectively.

A parametric study based on the steel ratios shows that ACI strength interactive curves typically have higher strength than the AISC curves in the compression zone as shown in Figure 2. When the axial force ratios drop below $P_u/\varphi_c P_n = 0.2$, the AISC procedure, however, results in higher strength.

Based on the information from the original testing data and studies used in developing the AISC composite requirements, the concrete encased arch ribs with multiple steel shapes connected by lacing bars are considered to be composite members.

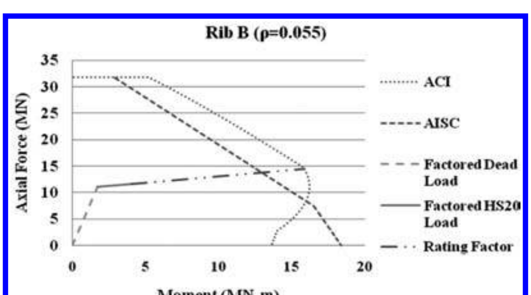

Figure 1. Rib B P-M interaction curves and load ratings.

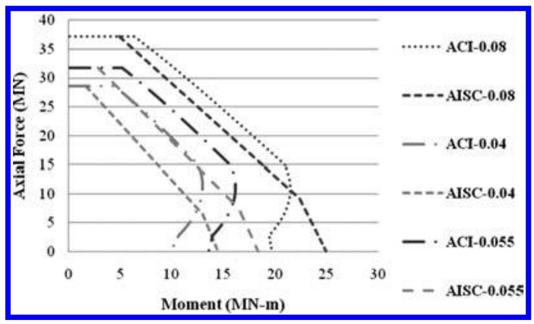

Figure 2. ACI and AISC P-M strength curves.

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Rope access bridge inspections

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ABSTRACT

Bridge Inspection has received the spotlight since the tragic collapse of the I-35W Bridge in Minneapolis, MN. Proper bridge inspection is dependent upon adequate access to the structure. Since you cannot inspect what you cannot see, visual inspections rely upon the inspector having the requisite access to all components of a bridge structure. This paper focuses on the various rope access methods currently available to bridge inspectors to supplement conventional means.

HDR Engineering, Inc. (HDR) has developed rope access techniques for evaluating structures in remote locations as well as urban settings. Engineers are challenged to evaluate structural components within arm's reach on designated fracture critical member bridges. Rope access is the application of specialized rope techniques to place inspectors in hard-to-reach locations in the vertical environment, especially when conventional man-lift, bucket truck or under bridge inspection vehicle access is prohibited due to insufficient deck width (functionally obsolete), desire not to impede normal travel lanes, or load posting of the structure (structurally deficient). Rope access is a work system using ropes and specialized hardware as the primary means of supporting inspectors. Rope access inspectors descend, ascend and traverse ropes to access the structure to perform a hands-on inspection. The support of the rope completely eliminates the likelihood of a fall. Rope access inspectors use a back-up fall protection system in the unlikely failure of their primary means of support. This redundant system is achieved by using two ropes, a working line and a safety line.

HDR's rope access bridge inspectors are certified to Level I, Worker, by the Society of Professional Rope Access Technicians (SPRAT), a member-based organization that serves the rope-access industry by developing and maintaining standards and administering an independent certification program. HDR's specialized bridge inspectors have received Rope Access I training and annual refresher training from Skala, Inc. (Skala), and have hundreds of hours of logged time on ropes, exclusively performing bridge inspections for numerous owners and maintainers. All rope access inspections are performed under the supervision of a qualified SPRAT-certified Level III, Safety Supervisor, with the training, skills, experience and qualifications necessary to assume responsibility for the entire rope access work site to include the design, analysis, evaluation and implementation of the rope access system. HDR's teaming partner, Skala, has provided safety supervision for all rope access bridge inspections completed to date.

This paper describes the planning, coordination, logistics and overall process of conducting successful rope access field inspection of fracture critical member bridges for representative owners and maintainers located throughout the United States (see Table 1).

Table 1. Representative Rope Access Bridge Inspections.

Client	Bridges
AKDOT&PF	12 Fracture Critical Highway Bridges throughout Alaska (statewide)
AKDOT&PF	36 Fracture Critical Transfer Bridges throughout Alaska (statewide)
Mn/DOT	5 Truss Bridges throughout Central Minnesota
NHDOT	Memorial Bridge over the Piscataqua River in Portsmouth, NH
NHDOT	Sarah Long Bridge over the Piscataqua River in Portsmouth, NH
ODOT	Steel Bridge over the Willamette River in Portland, OR

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California bridge management

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ABSTRACT

The Golden State of California boasts a growing, diverse population of 38 million people. For more than 50 years, California has embodied a spirit of innovation and entrepreneurship that has given birth to new technologies and industries while building the eighth largest economy in the world.

The backbone of the economic miracle that is California has been the construction, operation and maintenance of the finest highway system in the world.

Today, that system of highways and bridges safely and reliably carries millions of people and dependably delivers commerce to and from every corner of the state, the nation and the world.

The California Department of Transportation (Caltrans) is responsible for maintaining more than 50,000 lanes miles of pavement and 12,189 bridges on the state highway system.

As part of the Caltrans Division of Maintenance, the Structure Maintenance & Investigations (SM&I) unit is charged with ensuring the structural integrity of the state highway bridge inventory and 11,790 bridges owned by local government agencies.

SM&I's staff of 179 bridge engineers, structural steel technicians and support staff perform bridge inspections and engineering investigations in accordance with federal regulations on the state highway and local government agency bridges.

SM&I makes repair work recommendations, determines the safe load capacity of all bridges, reviews and approves encroachment permits and air space lease proposals involving structures, manages the bridge programs, delivers plans specifications and estimates for maintenance and rehabilitation on bridge projects and coordinates the protective coating work on more than 800 steel state highway bridges.

California has an ambitious bridge preservation program designed to extend the reliable, productive service of the state's bridges. The preservation effort is comprised of three main components: Caltrans inspection and maintenance crews, major maintenance contract projects and the State Highway Operation and Protection Program (SHOPP)

Beginning in 2004, the California Legislature recognized the need to provide more investment in preventative activities for pavement, bridges and drainage assets. Caltrans prepared the first Five Year Maintenance Plan in 2005 and has since submitted two additional plans, the most recent in 2009. This plan is required to be updated every two years. The primary goal of the plan is to reduce or defer the costly, time-consuming, major works such as rehabilitation and/or replacement projects.

Bridges like all physical assets will deteriorate over time and require maintenance, rehabilitation or replacement.

Caltrans bridge preservation approach is a three prong attack that strives to counteract deterioration:

1. Bridge crews employed by Caltrans address minor preservation very quickly to keep bridges in the green.
2. Major maintenance contracts are required when the scope of the work exceeds what the crews can do.
3. Major maintenance contracts are designed to delay or prevent the progression into the costly red rehabilitation/replacement area.

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Post-earthquake bridge inspection guidelines for New York state

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ABSTRACT

The New York State Department of Transportation (NYSDOT) employs a rigorous bridge evaluation program to assess the condition of its bridges and uses this information to formulate its inspection, maintenance and capital programs to improve safety of the bridge network.

In general, the bridge failure history in New York State shows that the greatest potential for serious bridge damage is not due to earthquakes; it is from hydraulic forces imposed on the substructure and corrosion and/or fatigue in superstructure elements. Though earthquakes are not the number one concern in NYS, the NYSDOT is taking steps to prepare for this low probability, but possibly high consequence scenario.

In the absence of nationally accepted standards for post-earthquake bridge inspection procedures, NYSDOT initiated a research project to develop the guidelines. These guidelines are under development and are briefly described in this paper. This earthquake response plan (see Figure 1) is for any earthquake of intensity equal to or greater than M3.5. The plan gives guidance on initial damage assessments, managing subsequent bridge inspections, follow-up action, and repairs that may be necessary. Though this procedure is tailored to NYSDOT needs, it may be useful to other highway transportation agencies with some modifications.

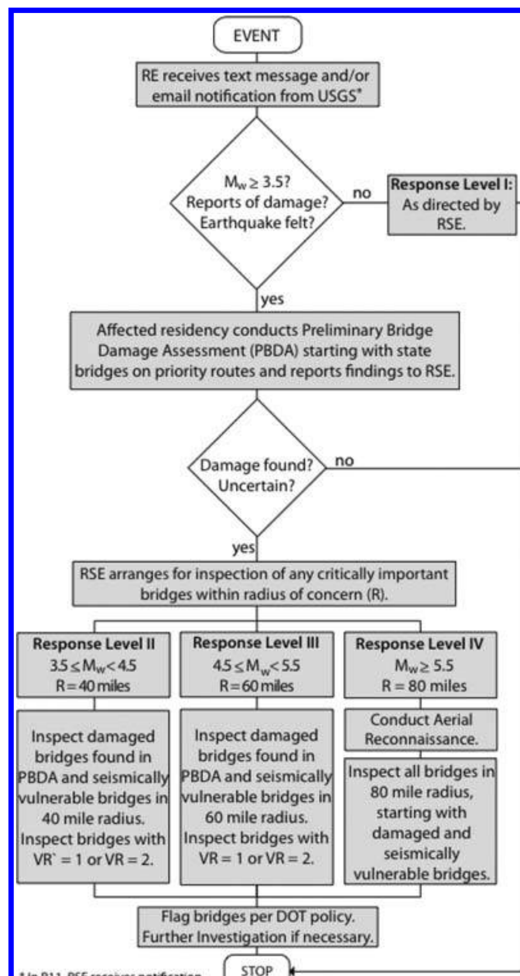

Figure 1. Proposed process flowchart for post-earthquake damage assessment program.

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A framework for evaluating the impact of structural health monitoring on bridge management

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ABSTRACT

Although structural monitoring has been recognized as a powerful information tool, bridge managers often make decisions based on their experience or on common sense, somehow regardless of the action suggested by instrumental damage detection algorithms. The first reason for this is that damage detection is normally affected by modeling errors and environmental noise: damage features are not deterministically related to the state and decision makers will weight differently the outcomes of the detection based on their prior perception of the state of the structure. The second point is that infrastructure owners are very concerned with the consequences of wrong action, and so will decide keeping in mind the possible effects of the action they can undertake. In this paper we propose a rational framework to include the impact of the mentioned issues on decision making. The former issue can be addressed formally by using Bayesian logic (MacKay 2003, Beck & Katafygiotis 1998), while the latter point suggests the problem be recast in the more general framework of decision theory (Russell & Wefald 1991). To quantitatively rate the benefit of health monitoring on bridge management, we will use the concept of Value of Information (VoI), which represents the money saved every time the manager interrogates the monitoring system after a potentially damaging event.

The methodology is applied to the Bill Emerson Memorial Bridge, a new 1206 meter long cable-stayed structure across the Mississippi River, instrumented with an 84-channel seismic instrumentation system. The approach developed for processing the monitoring data recorded on the bridge is based on two sequential neural networks (Wang *et al.* 2007), respectively trained for modeling i) the healthy response of the bridge and ii) the relation between damage and variation in the response. After the training process, the two networks are applied to evaluation of the current condition of the existing bridge. The neural network based technique can not only detect each damage scenario successfully, but also identify the location of changes correctly. Indeed, the maximum identification error for the stiffness ratios is less than 5 percent for all the conditions used in the training of the networks.

To illustrate how the procedure for estimating the VoI applies to the Bill Emerson Memorial Bridge, we consider the case of a single interrogation of the system after a seismic event. Keeping the problem as simple as possible, we arbitrarily consider in this exercise only two possible states (damaged and undamaged) and two decision options (do-nothing and inspection). Using a finite element model (FEM), we estimate the probability distributions of the response of the bridge for a possible damage scenario involving formation of plastic hinges at the intersection of tower columns and cap beams. A Monte Carlo analysis has been performed to estimate the empirical likelihood distributions in the two states: numerical simulations have been carried out on the FEM in the damaged and intact situations, introducing various sources of disturbances. The example shows how it is possible to estimate the economic benefit of a monitoring system for any event which requires a damage assessment.

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Bridge management and inspection for the County of Baltimore, Maryland

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ABSTRACT

Baltimore County owns over 350 bridges covering a wide range of types and lengths. Its prior methods of utilizing bridge data for its management and inspection processes were typical of that of many other cities and counties: largely paper based with a handful of spreadsheets and small disjoint databases. For its latest inspection cycle the county adopted an advanced bridge inspection and management system to organize all of its bridge information covering a wide variety of structures.

This paper will cover an overview of the county's needs and the solutions that have been developed to significantly improve both the inspection and management processes. In the latest inspection cycle an inspection team consisting of five different consulting companies was selected. Their experiences using the software along with how it was enabled to support multiple field teams and offices to work together will be presented. The companies used the software on a variety of hardware ranging from pen-enabled tablet computers to web-based office clients. Electronic forms were created to meet the county's requirements and allow for entry of all information from the inspection.

The county's usage and goals of its new management software which utilizes the electronic inspection data will also be discussed. Baltimore County personnel are now able to review all reports via a secure web-based browser and utilize powerful tools to instantly track changes occurring, highlight problem areas, and generate a wide range of needed reports. This new approach has allowed the county to improve its analysis and accuracy of inspection data while assisting in identifying problem early allowing a more proactive approach toward bridge management instead of a reactive approach.

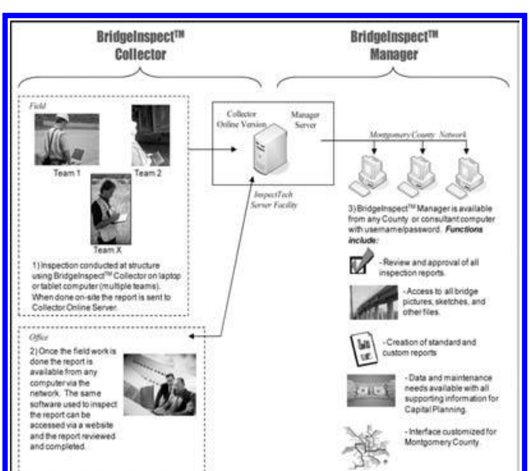

Figure 1. Software components by functions *(Montgomery Implementation)*.

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Overview of remote structural health monitoring: Focus on end user demands

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Monitoring of bridge structures has developed from a time-consuming, labour-intensive manual exercise to highly automatic systems, benefitting from the technologies of the information age. The systems are now highly automated, independent and versatile. They can be tailored to provide almost any type of information that may be required on the condition of a bridge and its environment and loadings. This paper describes some of the types of monitoring system available and the purposes they can serve:

– 'BASIC' systems with a low measurement frequency (15 minutes). This system is fully automatic and independent, offering on-line analysis and alert functions, with low initial investment and low running costs. Suitable for any applications which require slow movement observation.
– High-end and bespoke monitoring systems 'ADVANCED', offering individual and integrated system design solutions and on-line analysis for highest demands. Measurement frequencies of 100–1000 Hz are possible. Uses of such elaborate systems would include monitoring of complete bridge behavior, or any specific needs of the bridge owner which cannot be fulfilled by the simpler systems available.
– Temporary-use systems with high measurement frequency (100 Hz). This type of system offers an easy and quick health check of a bridge. It is portable and therefore can be used on more than one bridge. It offers off-line data analysis with comprehensive results. Particularly suitable for assessment of the condition of a bridge or individual bridge parts (e.g. cables).

Figure 1. 'BASIC' monitoring system ROBO®CONTROL installed at Incheon Grand Bridge in Seoul – Korea.

Figure 2. 'ADVANCED' monitoring system ROBO® CONTROL installed at Weyermannshaus Viaduct in Bern – Switzerland.

An overview of executed projects is presented, to illustrate the diversity of modern monitoring solutions. In addition end user demands are discussed, and the gained experience and challenges from practical applications are described.

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Methods of quality control and quality assurance for highway bridge inspection in the United States

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ABSTRACT

Highway bridge inspections in the United States rely primarily on visual assessments to determine and record the condition of a bridge. These assessments typically result in a numerical rating for components or elements of a bridge. The National Bridge Inspection Standards (NBIS) require that States report the condition of the bridge deck, superstructure and substructure on a scale of 0 throught 9 (CFR, 2004). These condition ratings are subjective in nature, generally describing the state of observed deterioration or damage.

States meet the requirements of the NBIS through a variety of inspection schemes and processes, including the use of element-level inspections and other approaches to meet the needs and requirements of individual states. In addition to the variability of inspection schemes, organizational structures of states vary widely. Because of the diversity in approaches to bridge inspection in the U.S., there have been different approaches implemented by states to ensure the quality of bridge inspection practices. Revisions to the NBIS in 2004 required states to implement systematic quality control (QC) and quality assurance (QA) measures. These measures are intended to address variability in inspection results, ensuring consistency and accuracy of inspection results within states. To assist States with implementing the new requirements for bridge inspection QC/QA, the FHWA provided a recommended framework for QC/QA for bridge inspection. To enhance this framework and provide further guidance to states implementing QC/QA programs, a guideline was developed that summarized the methods of QC/QA currently implemented in the U.S., and provides generalize or generic descriptions of the different approaches such that states could implement methodologies to meet their programmatic needs (Washer 2009). This guideline provides information on the typical processes and methods for bridge inspection QC/QA, methods of measurement, and characteristics of effective programs.

Methods for ensuring the quality of bridge inspections are typically described as QC, procedures intended to assure quality is maintained at a certain level, and QA, methods intended to assure the effectiveness of QC and verify or measure the overall quality of the program.

Due to the diversity of bridge inspection programs in the U.S., there has been a variety of approaches implemented to provide QA within the respective program. As a result, there was a need to develop generic models that described different approaches to QA in a generalized way. Using these models, it would then be possible to efficiently characterize the methodology used within a specific state, evaluate different options or methods that could be implemented in the future, and identify methodologies that best met individual needs. A review of the existing QA programs in the US was conducted through a comprehensive literature review, assembly of codes and guidelines from various states, and interviews with key personnel within states. Based on this review, the characteristics of various bridge inspection programs were summarized into four general models: the Independent Oversight Model (IOM), the Control Bridge Model (CBM), Collaborative Peer Review Model (CPR), and the Field Verification Model (FVM). Each of these models has distinct characteristics in terms of how QA reviews are conducted and quality evaluated. The models are described in more detail in the body of the paper. Approaches common for the implementation of QC are also briefly described.

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Studies on the use of KEEL software for intelligent analyzing of bridge load-bearing capacity

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ABSTRACT

The goal of this paper is to work out an effective computer method to assess the bridge structure condition that is expressed by means of load-bearing parameter considering typical bridge damages. In order to find the proper algorithm a survey of available regression methods within the KEEL software was made. KEEL software is used to assess evolutionary algorithms for common data mining problems. Selected methods were tested using real data that were provided by experts and obtained results were compared to results assessed by the MyBriDE expert tool. All considered algorithms base either on the neural network or fuzzy rule learning approach.

First stage of the experiment focused on selecting a group of best methods taking into account mean squared error (MSE), standard deviation (STDEV) and mean response time (MRT) for each algorithm. Verified methods use normalized input data where the considered range is the interval [0, 1]. After this initial selection RBFN, COR_GA, Fuzzy-GAP, Fuzzy-SAP and Thrift algorithms were promoted for further tests.

The aim of experiment's second stage was to compare previously selected methods to the MyBriDE expert tool. In order this comparison was possible the structure of input data sets was reorganized – to use the same datasets as the expert tool was tested on. Also root mean squared error (RMSE) and mean absolute error (MAE) are used to determine the quality of possible predictions.

The comparison between tested methods and the expert tool shows that only two out of five considered algorithms using not normalized input data give results that are similar to expert tool predictions. These methods are Fuzzy-GAP and Thrift with mean RMSE for all data sets respectively 12.4% and 7.1% whereas the reference RMSE value is 3.5% (see Table 1 for details). Mainly because of a long execution time of the single algorithm only default parameter values were applied so it can be assumed that previously mentioned Thrift and Fuzzy-GAP methods are able to provide even more accurate predictions when applying other than default algorithm's parameter values. Presented results given by KEEL software are very useful for planned implementation of intelligent tools for the Bridge Management Systems.

Table 1. RMSE values of experiment's second stage for tested methods in comparison with results of the expert tool, normalized and not normalized input data.

	Normalized data		Non-normalized data	
Methods	Avg.	Max	Avg.	Max
RBFN	50.4	84.5	50.4	84.5
COR_GA	69.1	100	19.5	54.4
Fuzzy-GAP	**26.4**	**47**	**12.4**	**12.9**
Fuzzy-SAP	24.7	43.3	9.2	19.1
Thrift	**48.4**	**67.1**	**7.1**	**8.6**
MyBriDE	3.5	3.9	3.5	3.9

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MS4: Research & applications in bridge health monitoring
Organizers: F.N. Catbas, J.R. Casas, H. Furuta & D.M. Frangopol

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Design and implementation of load cell bearings to measure dead and live load effects in an aged long span bridge

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ABSTRACT

In the spring of 2010, the expansion bearings of the Burlington-Bristol Bridge were replaced due to a poor condition rating (that caused the bridge to be classified as *structurally deficient*). The replacement of these bearings represented a unique opportunity to configure the new bearings to monitor the dead load and live load actions as well as their variation with environmental conditions. Towards that end, a series of trial designs were developed with various bearing types, load cell configurations, etc. These candidate designs were then evaluated through a series of finite element analyses and a single design was selected. To validate the selected design, a prototype was developed and tested in the laboratory under various axial forces, unidirectional and bidirectional moments, and shear forces. Once validated, 14 "smart bearings" were fabricated and installed on the Burlington-Bristol Bridge. To verify their proper operation, a series of load tests were carried out following installation. This paper and presentation will detail the development and validation activities for the "smart bearings" as well as the results from the first few months of long-term monitoring. In addition, non-technical challenges associated with developing appropriate design and performance specifications for the bearings will be discussed.

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Maintenance monitoring system design of a movable bridge

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ABSTRACT

Movable bridges are important intersection points for both highway and marine traffic. Federal agencies and movable bridge owners in the United States broadly address the need to effectively manage and maintain these complex structures. Movable bridges are unique structures due to the complex interaction between their structural, mechanical and electrical systems and mechanisms. These mechanisms provide versatility to movable bridges; however, their intricate interrelation also produces some operation and maintenance challenges. Problems related to their locations (e.g. proximity to waterways) and fatigue (stress fluctuations during the operation) also increase the maintenance cost of the movable bridges which is significantly higher than that of the fixed bridges. Operation, maintenance and repair are particular challenges for movable bridges because of their unique design. Therefore, continuous monitoring of these structures is essential to improve maintenance operation, to decrease the maintenance costs, to track and evaluate performance and to provide solution alternatives to these issues.

In this study, the authors first provide an overview of the recent experiences from monitoring design and implementation on a representative movable bridge in Florida. The paper reviews a) issues and monitoring needs for the maintenance and safety of movable bridges, b) design of the sensing, data acquisition and communications, c) field implementation and challenges, d) analysis and information generation for safety, reliability and maintenance.

The instrumentation plan is designed to monitor the most critical electrical, mechanical, and structural components. The current installation consists of an array of 162 sensors which add up to 200+ channels. The monitored structural components include main girders, floor beams, stringers and live load shoes. As for the mechanical and electrical components; electrical motor, gear box, shafts, open gear, rack and pinion and trunnions are monitored with various sensors. Figure 2 shows some examples of the monitored locations. In addition, a weather station is also installed to monitor and correlate the environmental factors.

Since many different components of the bridge are monitored, it is clear that different methodologies should be used to obtain maximum information about the safety, reliability and maintenance of the bridge. The data from structural, mechanical and electrical parts are analyzed by using different methods. For example, high speed strain and acceleration data from structural elements are analyzed and correlated with traffic video stream to detect unusual behavior (Figure 2). The final aim of the SHM system is to mitigate both problems and maintenance costs by providing useful information to the bridge owners and bridge engineers.

Figure 1. Some of the monitoring items.

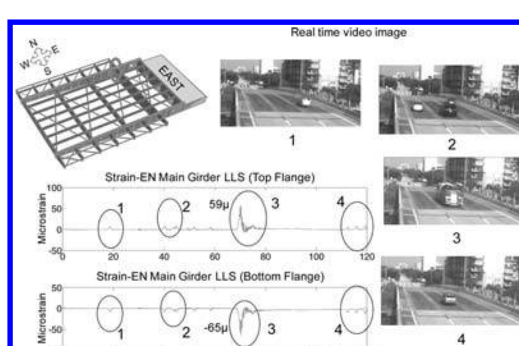

Figure 2. Example strain data from the main girder during regular traffic.

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Exploring indirect vehicle-bridge interaction for bridge SHM

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ABSTRACT

In this paper, we explore an indirect measurement approach for bridge structural health monitoring (SHM) that collects sensed information from the dynamic responses of many vehicles travelling over a bridge, and then makes extensive use of advanced signal processing techniques to determine information about the state of the bridge.

Most of the existing literature addresses direct measurement approaches, in which sensors are placed on the structural elements from which one wishes to collect information to be used for the damage identification. Yang et al. presented an indirect approach in 2004, with the sole objective of extracting bridge frequencies from the dynamic response of a moving vehicle. This was later validated in a real case example (Lin et al. 2005). We have extended the use of this indirect approach to SHM of bridges.

We expect that using an indirect approach to SHM will have a number of potential advantages. In this paper we discuss some issues such as: powering sensors, the lifespan of structures is much greater than the current reliable lifespan of most sensors, protection from environmental conditions and the threat of vandalism, traffic interruption and the potential for a broader coverage of the entire bridge population.

This approach is based on the collection of vehicle dynamic response data and the use of multiresolution image classification techniques on the collected data. In particular, we make use of a new multiresolution classifier (Chebira et al. 2007) developed by Kovaèeviæ's lab for image processing.

To test the validity of this approach, we created a numerical model of the interaction between a simple oscillator and a simple beam. We then subjected the beam to different levels of section loss at different locations. We then applied the multiresolution classification system to the simulated responses to these damaged states in order to determine how accurately the damage level could be classified. We conducted two experiments.

Experiment 1: Existence of damage classification. In this experiment, we attempted to classify the bridge into one of two categories: "Undamaged"; or "Damaged". Results indicate that this approach is able to achieve very high average accuracy of *88.4%* in determining the *existence* of damage.

Experiment 2: Severity of damage classification. This experiment considers five classes/labels: an "Undamaged" class and four "Damaged" levels. Overall, the multiresolution approach achieved an average accuracy of *71.2%* in determining the severity of the damage.

The results of these two experiments, while limited and very preliminary, seem promising. We are encouraged to further pursue the refinement and evaluation of this approach.

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Novelty detection based on symbolic data analysis applied to structural health monitoring

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ABSTRACT

Structural health monitoring is a problem that can be addressed at many levels. Generally, the objective is to determine whether damage is present or not based on the analysis of measured dynamic characteristics of a monitored system.

Dynamic measurements can easily contain over thousands of values making an analysis process extensive and prohibitive. Nevertheless, several damage detection methods exist in the literature based on signature principles, but they usually fail when making them practical. In this sense, despite the current processing power of computers, the necessary computational effort to manipulate large datasets remains a problem.

In this paper, the concept of Symbolic Data Analysis is used in order to classify different structural behaviors (Billard et al. 2006). For this purpose, raw information (acceleration measurements) but also processed information (modal data) are used for feature extraction.

This paper proposes an original approach in which the SDA is applied to three well know classification techniques: Bayesian Decision Trees, Neural Networks and Support Vector Machines (SVM). Results regarding experimental tests performed on a railway bridge in France are presented to highlight advantages and drawbacks of the described methodology. A dynamic monitoring of this railway bridge was performed to characterize and quantify the effect of a strengthening procedure (Fig. 1). In summary, three sets of dynamic tests were performed: before strengthening (15 tests represented by the letter A), during (13 tests represented by the letter R) and after (13 tests represented by the letter B). Also, the 13 tests performed during the strengthening were divided into 4 phases, as Table 1 shows.

In general, the SDA combined to the cited methods are efficient to classify and to discriminate structural modifications, either considering the vibration data or the modal parameters.

Table 2 presents some of the results obtained when SVM is used. It can be seen that reasonable results are achieved. In this case, only test TGV5R was incorrectly classified using the signals. Moreover, test TGV9R was incorrectly classed by signals and frequencies.

Figure 1. Strengthening system and procedure.

Table 1. Description of tests.

State	Phase	Tests
Before	–	TGV1A, TGV2A, . . ., TGV15A
During	1	TGV1R, TGV2R, TGV3R
During	2	TGV4R, TGV5R, TGV6R
During	3	TGV7R, TGV8R, TGV9R
During	4	TGV10R, . . ., TGV13R
After	–	TGV1B, TGV2B,. . ., TGV13B

Table 2. Classification obtained by the SVM method.

		Detected class		
Test	True class	Signals	Frequencies	Modes
TGV3R	1	1	1	1
TGV4R	1	1	1	1
TGV5R	1	2	1	1
TGV9R	2	1	1	2
TGV10R	2	2	2	2

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A temporal multi-scale analysis based bridge health divide-and-conquer monitoring and diagnosis strategy

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ABSTRACT

In present paper, a temporal multi-scale monitoring data process and interpretation scheme is proposed for bridge health monitoring and condition diagnosis. The signal time domain decomposition skills are applied to obtain the different components corresponded to different temporal scale information. For each temporal scale, the decomposed component is related and interpreted with a dominant physical or mechanical environment monitoring quantity. After that, the different features which act as a token of structure behaviors and operation conditions in different temporal scales are extracted by proposed with a divide-and-conquer algorithms collection. The proposed strategy is illustrated with two case studies on strain monitoring data process of Donghai Bridge Health Monitoring System and a freeway Bridge Health Monitoring System.

For a given civil engineering structures, both its behaviors and characteristics takes on random, time-variant and multi-scale features. As a reflection of structural behaviors and characteristics, the measuring data also takes on these features. The Physical and mechanical environment and structural response have a significant temporal multi-scale nature. This information has different variation rules on different temporal scale. The temporal multi-scale phenomenon discussed above can be illustrated in figure 1.

The long term acceleration signals monitoring from Donghai health monitoring system are interpreted in a viewpoint of temporal multi-scale. Second case study concerned of the dynamical strain data which collected from a freeway bridge in north China. One way decomposition tactics are adopted to give a divide-and-conquer treatment on these strain data.

The real monitoring case investigations shows that under the guidance of divide-and-conquer strategy, the information contained in the different time scales can be discovered one by one. And due to the decomposing, the interference among the numerous factors can be separated distinctly. More important is that by decomposing original signals into its temporal scale counterpart, high quantity, featured performance or damage index can be found and therefore a more effective monitoring, data interpreting and health status evaluation can be achieved.

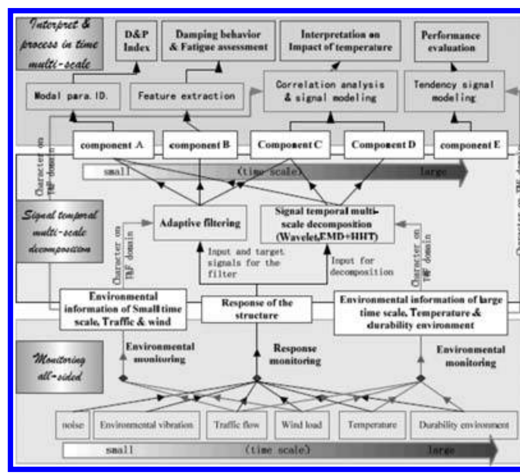

Figure 1. Divide-and-conquer strategy for interpretation of monitoring data and status assessment and diagnosis.

This paper is supported by NSFC project '50978196'; Free research project of Nation laboratory of civil engineering anti-disaster' SLDRCE09-D-01'; Provice-Ministry joint project of Guangdong Provincial Science and Technology Department & Ministry of Education, PRC,' 0812226100023'

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Validation of an SHM procedure for concrete bridges based on static strain records

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ABSTRACT

The paper is aimed at presenting the results of a research activity conducted jointly by the University of Genoa, Smartec S.A., and Autostrade per l'Italia S.p.A. The experiment consists in the continuous monitoring (for more than one year) of strains and temperatures in two pre-stressed concrete beam specimens, placed in open air and one of which has been subjected to increasing damages (Fig. 1).

Data interpretation is dedicated to find a relationship between the measured static structural response and features related to damage and/or degradation. The research focuses on the experimental validation of different damage identification algorithms; in this work Proper Orthogonal Decomposition (POD) and correlation analysis will be presented.

The first attempts to use real field measurements (Fig. 2), without any data pre-processing, to identify the already induced damages have apparently failed, producing false positives or unclear evidence of disturbances in the features of the signals associable to damage. This difference between numerical simulations and field experiments is most likely referable to one or both of the following reasons: a) the damages have been induced too early, when the reference period was not long enough and the signals were not stabilized; b) the intensity of the effect of damages was too low and hidden by the effects of irregular or anomalous temperature changes. In fact, the work shows that environmental variations have a great effect on measurements and have to be removed from the strain time histories before a reliable damage detection procedure can be successfully applied.

Further analysis aimed at removing the environmental changes has been presented. The procedure

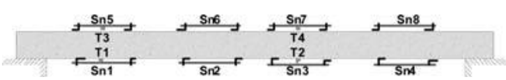

Figure 1. Sensors location on the Beam A (Sn: SOFO sensor; T: thermocouple).

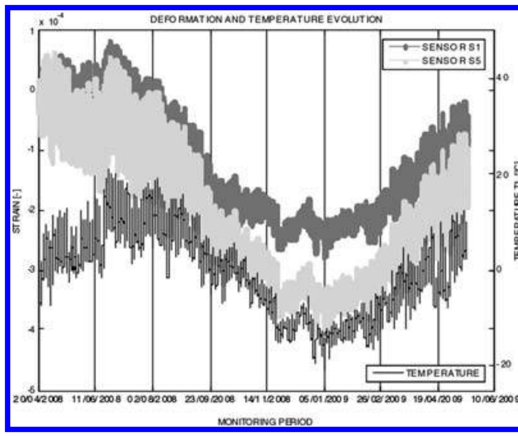

Figure 2. Complete time histories of temperature T1 and strains at sensor locations Sn1 and Sn5.

seems to be effective in reducing the strain variations and the dead load positioning can be clearly identified from all extracted eigenvectors (POD analysis). Some doubts still remain regarding the identification of damages. It's in the Author's opinion that the introduced damage cannot be detected as a punctual event due to the pretension force but they could be visible after a certain period as a degradation of the beams performance with time. The consideration related to damages introduced too early before the signal stabilization seems to be also confirmed by the eigenvectors obtained after the strain pre-processing.

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Uncertainty and reliability analysis using monitoring data and artificial neural network (ANN) calibration

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ABSTRACT

Probabilistic techniques in engineering problems provide a deeper understanding of the aleatory and epistemic uncertainties inherent to the structures being analyzed. Complex engineering structures are usually analyzed with finite element techniques to incorporate all critical details with geometric representation. The prediction of structural reliability based on a predefined limit state can be obtained with a finite element model and can be updated using Bayesian methods with the monitoring data. Another common approach is to calibrate a finite element model with the monitoring data by minimizing the error between the analysis and the measurements, which requires more time and user interaction.

The objective of this paper is to explore the comparison of the model responses and predictions between these two approaches where uncertainties are incorporated in a different manner. For this study, a test set-up which is a simplified series-parallel physical model with four structural elements (Double-H-Frame-DHF) is designed and extensively instrumented with various sensors, and monitored over time with different structural boundary conditions. A large number of simulations using the finite element model are performed under uncertainties associated with material properties, geometry, loading and boundary conditions. The boundary conditions are changed gradually and the two approaches are executed to obtain the reliability distribution at each structural state and also to predict future performance. The findings from the two approaches are compared.

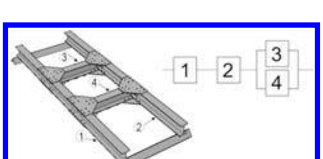

Figure 1. DHF and reliability block diagram.

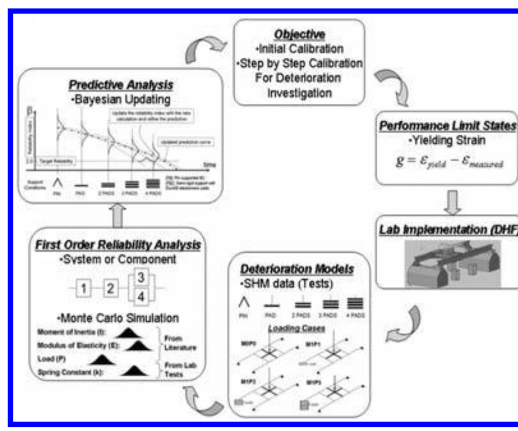

Figure 2. Framework of the proposed approach.

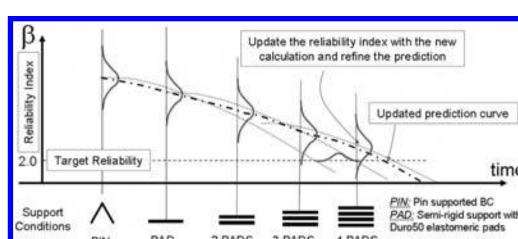

Figure 3. Reliability index change under support deteriorations.

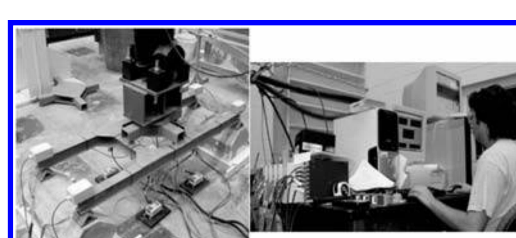

Figure 4. Test setup and data acquisition.

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Effects of environmental changes on the dynamic characteristic of reinforced concrete beams

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ABSTRACT

A modal test program comprised ten reinforced concrete beam specimens, with the dimensions of 150 mm width, 200 mm height and 2200 mm long, was conducted in the structural laboratory to study the correlation between the environmental changes with the variation of the dynamic characteristics of concrete beams. The compressive strengths of 21 MPa and 40 MPa for concrete were considered for different specimens. Tensile and compressive reinforcement ratios varied in different specimens. The dynamic response of the beams was recorded using one accelerometer. To eliminate the problem of support effects on dynamic test results, the specimens were suspended by elastic cables (Fig. 1). In the dynamic tests, the impact hammer provided the excitation input at every degree of freedom, and the frequency response functions (FRF) were derived by measuring the input force (of impact hammer) and response (of the stationary accelerometer) at each measurement point. For each point, test procedure was repeated ten times and the average of FRFs was finally stored as the input-output transfer function for that particular point.

Frequencies and damping values of specimens were recorded in the model tests. From the test results it was found that the frequencies of all specimens decreased with the age of specimens. The frequencies of the specimen kept outdoors under hot and dry conditions (during 11–39 days after casting) decreased, significantly. However, when it was brought back to the laboratory after 39 days of casting, the frequencies changed mainly with the temperature of the specimen at the test time. The average values of the frequencies of the specimens with 40 MPa concrete strength were approximately 18 percent larger than those of the specimens with 21 MPa concrete strength. The frequencies of the specimens with the larger tensile reinforcement ratio were larger than those of the specimens with the smaller one. No meaningful trend was seen in the relationship between the damping values and the age of

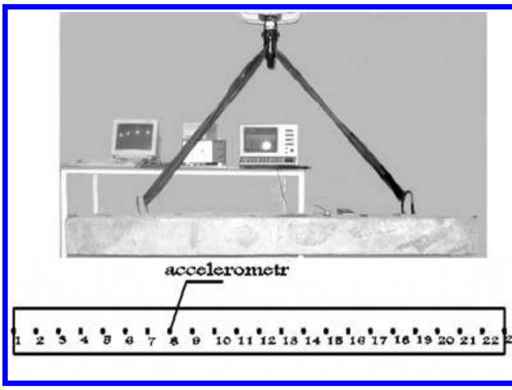

Figure 1. A suspended beam and elastic cables, and different degrees of freedom of beams.

specimens. The damping values of the specimen kept in hot and dry conditions were considerably larger than those of other specimens. It may be concluded that the damping value increases with the increase of temperature and concrete shrinkage.

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Damage detection using a novel time series methodology: Application to the Z24 Bridge data

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ABSTRACT

In this study, a novel time series analysis methodology is used for detection, localization, and quantification of damage. The methodology is based on creating ARX models (Auto-Regressive models with eXogenous input) for different sensor clusters. The output of each sensor in a cluster is used as an input to the ARX model to predict the output of the reference channel of that sensor cluster. After the ARX models for the healthy structure at each DOF are created, the same models are used for predicting the data from the damaged structure. The difference between the fit ratios is used as damage indicating feature. The methodology is applied to the experimental data coming from the Z24 Bridge. It is shown that the approach is successfully used for identification, localization, and quantification of different damage cases. The potential and advantages of the methodology are discussed along with the analysis results. The limitations and shortcomings of the methodology are also addressed.

The Z24 Bridge benchmark is one of the most commonly used benchmark problems by researchers in SHM and experimental dynamics. Therefore, the methodology is applied to the Z24 Benchmark data for verification. In this study, three levels of pier settlement are investigated. The levels of the settlements are 40 mm, 80 mm and 95 mm. The damaged pier is referred as the Koppigen pier and it is shown in Figure 2.

The Damage Features (DFs) computed for baseline and damaged cases are shown in Figure 3. It is observed that the DFs are higher for Setups 6–9 where the pier settlement occurred. This figure represents a good illustration of the increase in the DFs for the damaged locations. Furthermore, the plots also give an idea about the severity of the damage, as the DFs get higher with increased settlement. It is observed that the DFs for 95 mm settlement are higher than DFs for 80 and 40 mm settlement (except two points) and the DFs for 80 mm settlement are higher than that of 40 mm settlement.

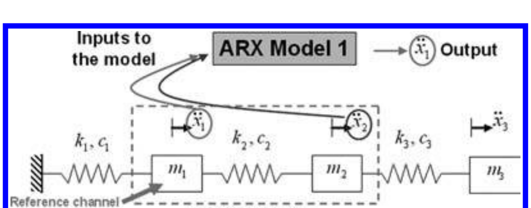

Figure 1. Creating different ARX models for each sensor cluster (first sensor cluster).

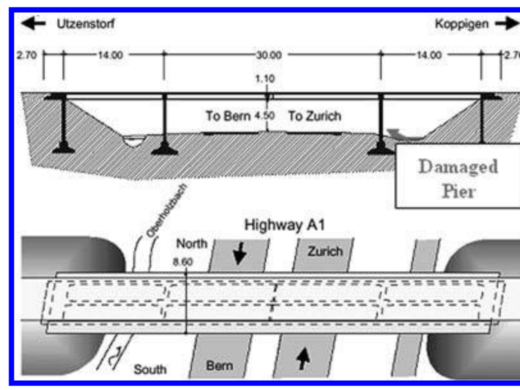

Figure 2. The geometry of the bridge; top and plan view (modified from the Z24 benchmark documents).

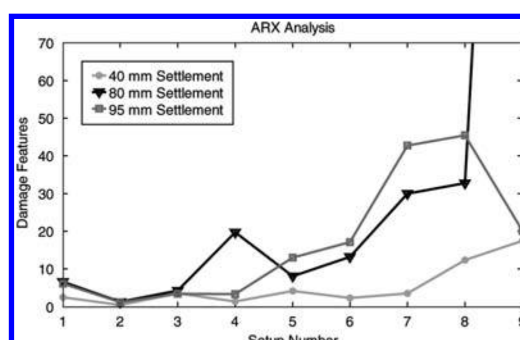

Figure 3. The damage features for each setup.

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Development of a bridge health monitoring approach using train-bridge interaction analysis and GA optimization

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1 INTRODUCTION

In the maintenance scheme of Shinkansen viaducts in Japan, the health monitoring and diagnosis of the structures becomes especially important because of the huge number of structures. Currently, the overall health condition of Shinkansen viaducts is mainly examined by visual inspections, which demand a large number of technicians and also a considerable cost. With the proceeding of a decreasing birthrate as well as an aging society in Japan, it is significant to develop more effective and economical health monitoring and diagnosis approaches to the civil infrastructures. If the dynamic bridge response induced by the running trains can be effectively used for the health monitoring and diagnosis process, it will be an economical and convenient way because the train-induced vibrations have been continually recorded.

To identify the structural characteristics of the bridge with traffic-induced vibration data, a currently conventional way is to perform the inverse analysis of vehicle-bridge interaction. However, such an approach can encounter considerable difficulties in actual applications because of numerical errors caused by the inverse analysis due to the large number of members. In this research, a structural identification approach is developed employing only direct analyses of train-induced bridge vibration by means of introducing soft computing methods to avoid the numerical error mentioned above.

In the actual railway viaducts, the possible damage patterns of the structures are comprehended by the bridge engineers based on theoretical and empirical facts. In this approach, the damage patterns of the bridge members are assumed in advance and used as the input information. Then, the train-induced dynamic responses of the bridge under a certain damage pattern are calculated by a developed analytical procedure. In the assumed damage patterns, the one identical or nearest to the actual damage condition will give the most similar dynamic responses to the recorded ones, through which the exact solution can be identified. To make this approach applicable to actual structures with enormous possible damage patterns, the soft computing methods of Neural Networks (NN) and Genetic Algorithm (GA) are introduced and applied as follows.

a) In this proposed approach, the traffic-induced dynamic responses of the structure used to compare with measured ones will be simulated by a developed vehicle-bridge interaction analysis computer program. However, for a large-scale structure even one time of such an analysis will demand considerable computational capacities, which will leads to the infeasibility in an actual identification process that needs a great number of interaction analyses. Therefore in this research, the NN techniques are planned to be used to simulate the running train-induced bridge response, which, once established, can shorten the computational time of the identification process to an acceptable degree in actual applications. To establish such a NN tool, it is impossible to use measured results to carry out the supervised learning process, thus the results from the train-bridge interaction analysis program have to be used as the sample data. Not to mention, adequate demanded accuracy for such an analytical program is indispensable in the actual applications.
b) The calculated traffic-induced bridge responses under certain damage patterns described above are then used for the identification process. However, even only the possible damage patterns based on engineering facts are assumed, the number can still be considerable large and difficult to identify. This is a typical combinatorial optimization problem and can be solved by some metaheuristic search algorithms. In this research, the GA technique is adopted to find the exact damage pattern. In the GA algorithm, the assumed damage patterns are set as the population and the difference between the calculated results and the recorded ones is defined as the object function.

The basic concepts and process are represented using simple numerical models in this paper.

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Assessment of remaining fatigue life of aging orthotropic steel deck bridges

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ABSTRACT

The maintenance cost of old steel bridges is rapidly increasing since many of them are deteriorating due to accumulative stress cycles and reaching their design life. Moreover, selection of orthotropic steel decks for long-span bridges has become more popular, considering their longitudinal stiffness and low dead weight. Despite these advantages, containing several types of welded joints, these decks are prone to fatigue. Therefore fatigue life estimation of these structures seems to be an inevitable task in order to make cost effective decisions for rehabilitation solutions. Although fatigue analysis based on specification loads and distribution factors is completely reliable for new bridge designs, it usually underestimates the remaining fatigue life of existing bridges. Instead fatigue evaluation using field measurement data under actual traffic load has proven to be more accurate for existing bridges.

This paper presents a procedure using this method for fatigue life evaluation, applied for one of aging orthotropic steel deck bridges in Iran. Hafez-Taleghani Fly-over is a 31-simple-span Orthotropic Deck Steel Bridge in Tehran. It carries three traffic lanes and was constructed in 1970. As maintained by the city urban plan, this bridge and three other identical bridges were primarily a temporary solution to growing traffic of the city, but they are still under service. After determination of stress ranges using specification loads, the fatigue life of the bridges obtained insufficient for their service life. So in order to assess the level of fatigue damages and to find out feasible mitigations measures, an extensive plan for structural health monitoring including field strain measurement, has been accomplished. The most vulnerable details in this case are the longitudinal weld between the web and bottom flange which experiences tension cycles. The in-field data which has been collected due to normal traffic and during a seven day period, processed using the procedures outlined in AASHTO specifications.

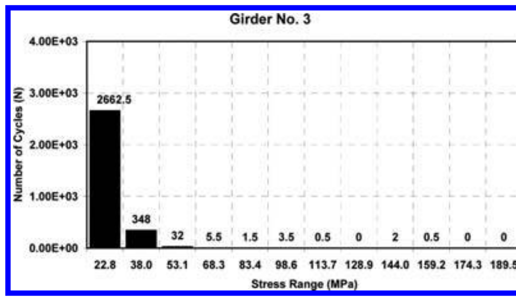

Figure 1. Stress histogram after filtering stress ranges below fatigue threshold, for girder No.3.

As a classic approach, S-N curves are used in the current study to estimate the fatigue life of the bridge. Stress cycles/ranges were extracted using a program which was developed in MATLAB based on rain flow counting algorithm proposed by ASTM (2004). Since large number of very low strains will decrease the effective stress range obtained from strain histogram, there should be a criterion for eliminating strain cycles below a defined threshold. In the current study truck weighing 10 tons was used to define the fatigue strain thresholds. Stress histogram after filtering the strain ranges below fatigue threshold for one of the girders is shown in Figure 1. Accordingly, as per AASHTO (1990) and based on the existing traffic regime the fatigue life of the girders is estimated to be infinite.

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Identifying bending stiffness change of a beam under a moving vehicle

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ABSTRACT

Usually, to define damage for bridge structures is difficult differently from other structures such as automobiles, aerial vehicles, etc. This is one reason why most precedent studies particularly addressing bridge health monitoring (BHM) has specifically examined global change of modal properties and quantities of bridge structures. Those modal parameters are usually identified using the ambient vibration (e.g. Wenzel & Pichler 2005).

For bridges with a long span, wind-induced vibrations are important dynamic sources. On the other hand, for short span bridges which are insensitive to the wind load, how to excite the bridge economically, reliably and rapidly is an important task.

This study is an attempt to use traffic-induced vibrations for the damage identification of short span bridges using an algorithm derived from a bridge-vehicle interactive system (Kim et al. 2008), because a moving vehicle is able to actuate short span bridges economically, reliably and rapidly. Feasibility investigation through a moving vehicle laboratory experiment is the major goal of this study.

In experiment three vehicle models called as VT-A, VT-B and VT-C of which the natural frequencies for the bounce motion are respectively 2.93 Hz, 3.76 Hz and 3.03 Hz are considered with six loading scenarios: SCN1 of VT-A vehicle traveling with $v = 0.93$ m/s; SCN2 of VT-A vehicle traveling with $v = 1.63$ m/s; SCN3 of VT-B vehicle traveling with $v = 0.93$ m/s; SCN4 of VT-B vehicle traveling with $v = 1.63$ m/s; SCN5 of VT-C vehicle traveling with $v = 0.93$ m/s; and SCN6: VT-C vehicle traveling with $v = 1.63$ m/s.

The experimental study demonstrates that better chance to detect damage is observed by using the vehicle with the frequency which is close to the natural frequency of the experimental bridge (2.69 Hz, 2.59 Hz and 2.54 Hz respectively for the intact and two damage scenarios.) as shown in Figure 1 as well as adopting higher vehicle speed as shown in Figure 2. It can be concluded, however, that locations and severities of damage are constantly identified without great variation according to vehicle type and speed.

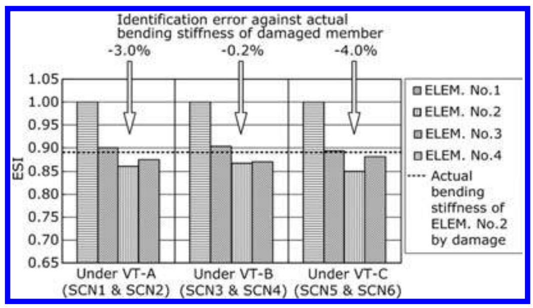

Figure 1. Identified damage location and severity of the bridge with damage at ELEM. No.2 according to vehicle type.

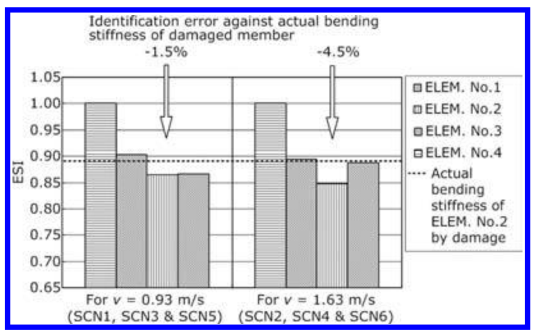

Figure 2. Identified damage location and severity of the bridge with damage at ELEM. No.2 according to vehicle speed.

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Low-cost wireless sensor node for vibration monitoring of infrastructures

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ABSTRACT

Maintaining and improving condition of civil infrastructures is critical to the structural integrity. Structural health monitoring (SHM) from vibration data of the structure is recognized as an important approach for maintaining the structures. To activate health monitoring of civil structures using vibration measurements, developing a cost-effective monitoring method and sensor system have been technical issues.

Experience with field-deployed systems has shown that the wired sensor can be costly to maintain because of environmental degradation (Lynch 2004). In addition, the deployment of such system can be challenging with a great portion of the installation time attributed to the installation of system wires and cables for large scale structures (Farrar et al. 2006). The integration of wireless communication technologies into SHM methods has been widely investigated in order to overcome the limitations of wired sensing networks. Wireless communication can remedy the cabling problems of the traditional monitoring system and significantly reduce the maintenance cost.

Considering real situation, the available wireless sensors are not necessarily designed to meet specific requirements for the vibration-based SHM such as high sensitivity, power supply, cost effectiveness, etc., and are very limited.

This paper discusses a performance experiment for a wireless sensor node (see Figure 1) equipped MEMS accelerometer and wireless device for data transmitting, which is aiming to design a low-cost wireless sensor node for monitoring civil structures. Performance of the prototype of the sensor node is investigated through vibration experiments on a newly constructed bridge and a real pedestrian bridge. In the experiments, a relay node and a high performance antenna are used to improve radio transmission. Comparable results between data taken from cabled and wireless sensors such as acceleration and FFT are obtained, even though noisy signals on the measured time history taken by wireless sensors are observed.

This study also investigates the feasibility of missing data recovery by means of the Kalman filter. Recovered signals (see Figure 2) show comparable Fourier spectra profile with those of cabled sensors. However the recovered amplitude are relatively small in comparison with cabled sensor, which is one of the next challenges of this study.

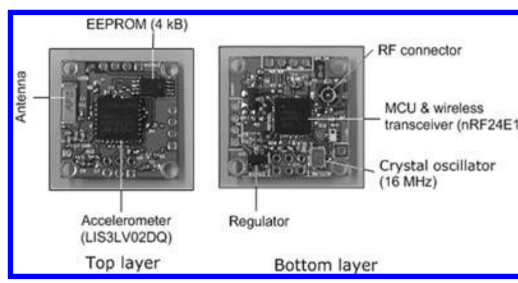

Figure 1. Wireless sensor node.

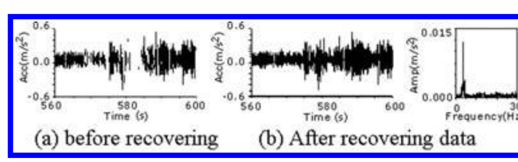

Figure 2. Acceleration and Fourier amplitude before and after recovering missing data taken from wireless sensor.

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Bridge retrofit design optimization for fatigue based on monitoring and FE analysis

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ABSTRACT

Over the past decades, aging steel tied-arch bridges, which have been built with the floorbeams connected into the tie girder with web shear connections, have been deteriorated due to fatigue cracks caused by relative movements. Several retrofit strategies have been proposed to remove structural deficiencies caused by fatigue and to preserve bridge safely up to its remaining fatigue life. As a retrofit, the softening method by partial removal of the connection angles can be employed in order to prevent stresses within the web gap from concentrating. However, it is not simple to find relevant retrofit sizes since fatigue cracks after retrofit may move into other regions or reinitiate near the retrofit regions. For this reason, optimal retrofit solutions have to be found.

For this purpose, cut-out area for retrofit is defined as objective, while the AASHTO constant amplitude fatigue limit (CAFL) is imposed as upper and lower bounds of stress constraints. Finite element analysis (FEA) is performed to investigate stress distributions in structural details associated with different cut-out areas. Fatigue reliability evaluation at potential critical locations is performed to estimate remaining fatigue life associated with the optimal retrofit solutions.

The fatigue reliability approach is based on AASHTO Specifications (2002). All necessary information is obtained from both finite element modeling (FEM) and field monitoring data from the original retrofit size. The developed FEM is validated by comparing the analytical results from FEA with field measurements. Critical locations, where potential distortion-induced fatigue cracking can occur, are identified from FEA. The optimization problem is formulated as the cut-out area minimization subjected to stress constraints associated with the CAFL corresponding to each category. The optimal solutions are computed by linking the FEM software ABAQUS (version 6.7.1, 2007) with the optimization software VisualDoc (2006). Fatigue reliability assessment is performed at the identified critical locations based on field monitoring data and FEA. The proposed approach is illustrated on an existing bridge.

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Updating the finite element model of bridge structures by an improved Taguchi updating method

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ABSTRACT

Structural finite element (FE) model updating is the core technique of vibration-based structural health monitoring (SHM) of bridge structures since this technique can provide an accurate baseline model for damage detection, structural control and safety assessment of bridge structures. In the procedure of model updating, the objective function is usually set as the weighted sum of the difference between analytical and experimental dynamic characteristics of structures (M.I. Friswell, 1995). But it is difficult to select the weighting factors since the relative importance of each parameter to updated results is not obvious but specific for different problem.

To overcome above problem, multi-objective genetic algorithm (GA) (Gyeong-Ho Kim, 2004) is introduced into FE model updating since there is no need for selecting weighting values. In practical situations, however, it is difficult to update the FE model of bridge structures by GA owing to the relative low efficiency. Taguchi updating method, deemed as an efficient and robust updating method, is a good alternative to GA for updating the FE model of large-scaled structures.

Taguchi method (Kye-Si Kwon, 2005) is a good replacer of GA to update the FE model of the large structures. On one hand, Taguchi method has more efficient than GA since orthogonal arrays (OA) (A. S. HEDAYAT, 1999) are applied to screen the main effect of parameters to objective function rather than stochastic search. On the other hand, the updated results of Taguchi method are robust against various noises since parameters are updated to maximize the signal to noise (SN) ratio. But Taguchi method is only applied to solve the single objective optimization problem of model updating. Therefore, this paper proposed improved Taguchi updating method to deal with the problem of model updating using multi-objective optimization technique. The classification of the searching population and the new objective function (the new SN ratio) are introduced into the Taguchi updating method, which makes the improved Taguchi method have the ability to search the Pareto-optimal set of FE model updating.

Finally, a parameterizing scheme for structural boundary condition is proposed, and the improved Taguchi updating method is employed to update the boundary conditions of a truss bridge structure. The updated results show that the proposed method is promising for FE model updating of bridge structures.

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High-order local vibration properties of RC Viaduct under the passing high speed train

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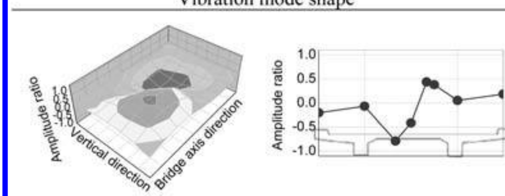
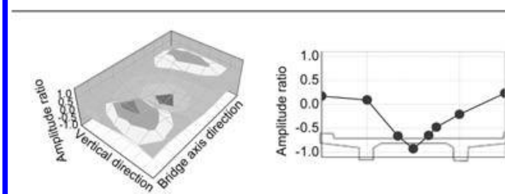
ABSTRACT

In order to sustain the development of high-speed trains, it is necessary for railway bridges to grasp the dynamic behavior of bridge members precisely, by identifying not only whole bridge vibration properties but also local ones, and high-order mode vibration properties. Actually, there are common concerns noises caused by the vibration of bridge members due to high-speed trains. However, there have been few vibration measurement experiments. In this circumstance, authors conducted the passing train experiment, measuring vibration concurrently at several points on the intermediate and projecting slabs of a RC viaduct for high-speed trains. Through the vibration measurement experiment, the authors verified the possibility of detection of high-order vibration modes of members and identified the outstanding vibration mode when a high-speed train passes such like table 1. In this table shows a result of vibration measurement experiment and identification analysis in case of central slab. As a result, the authors could observe the modes in which the vibration of each member is outstanding, and found that the vibration of the RC bridge due to a passing train is mainly caused by the vibration of the central slab, although there are low-order vibrations of the overhanging slab. It was also found that the second-order mode like table 1 contributes to outstanding components.

The future mission would be to reflect these results in actual bridges. It is considered necessary to identify the cause of such vibration problems and clarify influential modes and vibration modes, by discussing the relations with noise and ground vibration. In this study, the authors adopted the most fundamental method for identification.

Table 1. A result of identification by the experiment with a passing train (central slab).

Second-order diagonal mode of the central slab	
Vibration mode shape	
	
Contour map	Vertical direction
Equivalent natural frequency : 60.6 Hz	
	
Contour map	Vertical direction
Equivalent natural frequency : 93.7 Hz	

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Condition assessment of bridge deck truss using in-service monitoring data of strain

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ABSTRACT

Structural condition assessment is in great demand for efficient management of aging in-service infrastructure. A strategy which integrates structural condition assessment with structural health monitoring (SHM) system is expected to be viable because a SHM system can offer plentiful information on structural condition by use of various sensors and analysis techniques.

In practice, in-service monitoring of strain response plays an important role in structural condition assessment via monitoring because strain measurement can offer information about stresses experienced by the structure under operational conditions. Strain monitoring and testing in the field can be classified into two categories: long-term continuous monitoring and short-term portable testing. During temporary strain testing, such as live load testing, some long-term effect on strain response can be ignored since this kind of testing is usually completed in short time and purposed to record short-term structural behavior. On the other hand, long-term continuous monitoring of strain can capture real-time structural behavior not readily apparent from short-term testing, for example, daily and seasonal changes. Continuous monitoring of real-time streaming of dynamic strain is emerging as a critical strategy in the assessment, inspection and decision making on maintenance of bridges.

In this study, an approach for structural condition assessment of bridge deck truss making use of in-service monitoring data of strain from a SHM system is proposed and applied to the instrumented Tsing Ma Bridge (Figure 1). The proposed method consists

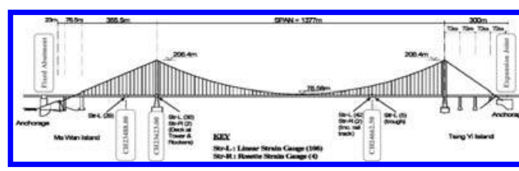

Figure 1. Tsing Ma Bridge and deck sections instrumented with strain gauges.

of structural assessment at two levels: (i) deck truss components, and (ii) deck cross-sections. Since the in-service monitoring data of strain is a result of multi-source combinations of live loads and environmental effect, wavelet-based multi-resolution analysis is first carried out to decompose the measured strain data sequences for signal extraction in recognizing that some ingredient induced by temperature is released through boundary conditions and contributes little to stress quantities. Thereafter, the time-history of stress response in deck truss components due to external load effects is derived from the extracted strain, and the peak values are counted to infer their statistical characteristics for the component-level assessment. As for the cross section-level assessment, the dynamic internal forces such as bending moments, axial forces and shear forces on the deck cross-section are evaluated by synthesizing the resultant internal forces of truss members from the derived stress-time history. Structural condition assessment of the deck cross-section is then carried out under various load conditions such as monsoon, typhoon, with and without railway traffic.

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Simultaneous monitoring of the coupled vibration between a bridge and moving trains

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ABSTRACT

The purpose of this study is to evaluate the coupled vibration between train and bridge experimentally. Moreover, numerical simulation on a simple beam and vehicles was also conducted to obtain the fundamental information of the coupled vibration.

The monitored bridge is located in Osaka city and belongs to Osaka city transportation authority. This bridge is a skew steel bridge with double decks, and the length is 39.8 m. On the basis of the actual monitoring by the bridge sensors, this bridge has, in the vertical direction, the first bending mode at 2.36 Hz, the first torsion mode at 3.89 Hz and the second torsion mode at 5.00 Hz, respectively. As for the horizontal direction, the peaks at 3.47 Hz, 3.89 Hz and 4.72 Hz were recognized. The train can have the eigenfrequency of 1.40 Hz.

As a result of the monitoring, it is found that the train eigenfrequency of 1.5 Hz can be recognized in the power spectrum of the train response, but it is difficult to find the bridge frequency in the power spectrum. However, the peak close to the bridge frequency of 2.36 Hz was found in the measured cross power spectrum between the bridge and train. This fact indicates that both vibration systems are surely correlated to each other and the train vibration must be affected by the bridge vibration. Thus this opens up possibility that the bridge vibration can be estimated in the train vibration, and the estimation theory should be developed.

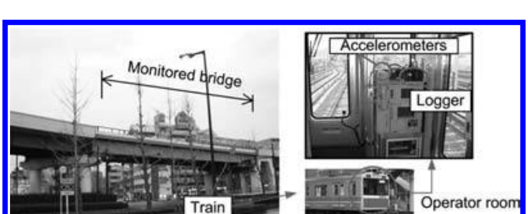

Figure 1. Picture of the monitored bridge and the train.

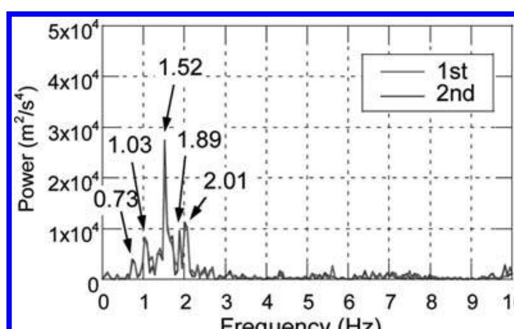

Figure 2. Power spectrum of the first car of the train.

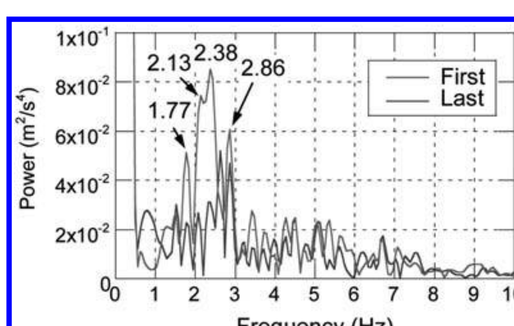

Figure 3. Cross spectrum of train and bridge.

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Challenges and uncertainty mitigation in structural identification of long span bridges

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ABSTRACT

Using full scale vibration testing as a means for structural identification (St-Id) of long span bridges represents an efficient technique for quantitatively characterizing the in-service mechanical attributes and behaviors of these complex systems. The resulting characterization can serve as an effective baseline for structural health monitoring, designing more reliable and cost-effective structural retrofits, or developing and implementing timely and efficient maintenance procedures. However, there are various uncertainties, known and unknown, involved in the experimental and identification processes that impact the reliability of St-Id and serve as a barrier to its more widespread acceptance and application in engineering practice. The prevailing excitations (wind and traffic), environmental conditions (radiation and ambient temperature), experimental hardware (sensors, cabling and data acquisition system), and the execution of the experiment (array density and distribution, data acquisition parameters, on-site quality control, etc) have significant influence on data quality. Developing analytical models of complex structural system also introduces significant uncertainty in the St-Id process. Recognizing the impact of various uncertainty mechanisms and employing appropriate techniques to quantify, bound and mitigate their impacts will greatly benefit bridge owners and engineers.

A full-scale vibration test on a long-span suspension bridge is taken as an example to illustrate a number of possible approaches for coping with the challenges presented by uncertainty in assuring practical identification of the dynamic characteristics of large-scale constructed systems. This paper examines uncertainty mechanisms in the following areas: (1) experiment design and implementation, (2) data pre-processing, (3) data post processing and modal parameter estimation. Statistical analysis of St-Id results from several datasets is also performed, which provides an effective means of investigating data reliability and its effect on St-Id. The identification results obtained for the bridge spans and towers demonstrate that the developed field testing and data processing methods may provide a reliable solution for bridge characterization.

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Bridge instrumentation for long term structural health monitoring

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ABSTRACT

Advances in structural analysis, instrumentation, data management, and reporting make it feasible to reconsider alternate approaches for bridge design. A new procedure can incorporate development of a "baseline" bridge model that can be used for structural health monitoring (SHM) of the bridge over its lifetime. Regular and effective use of SHM for bridges can provide more objective data on bridge conditions over time and lead to improved maintenance for more efficient use of limited resources. This approach has great promise at a time of aging infrastructure and limited funds for maintenance and repair.

This paper presents a new procedure for bridge design and load rating that incorporates the development of a long term SHM system which will lead to more efficient and effective bridge maintenance and management. A baseline finite element model that represents the actual 3D system behavior of the bridge is presented as a part of the designer's submittal requirement. Such a calibrated model captures the design intelligence envisioned by the original designers.

For this project, a continuous three span composite steel stringer bridge in Barre, Massachusetts was used as a pilot bridge. During construction of the Vernon Avenue Bridge, a monitoring system was installed in order to capture the locked in stresses that occur in the girders during construction, prior to being fully loaded. Once construction was completed a static load test was conducted to provide data that could be compared to the baseline model for verification. It is intended to use the collected test data during and after construction for finite element model updating using PARIS, a parameter estimation program that was developed at Tufts University.

The structural models created for the Vernon Avenue Bridge are a meaningful base for the evaluation of the collected data. Each phase of data was used to refine the structural sub-system prior to combining them for the overall system behavior. These models are more comprehensive than a structural design analytical model. Not only must the model capture the geometric and section properties, connection characteristics and boundary conditions but also all of the loading influences on the bridge. Typical methods for bridge design lead to well designed structures, however fail to accurately capture actual global structural behavior. When measured structural response is required to match a predictive model, typical structural modeling methods fall short. Changing from a design model to a condition assessment model for use with a structural health monitoring programs would provide bridge managers an in-depth understanding of structural behavior through the in-service life of the bridge with relatively little effort.

The SHM and condition assessment program being developed for the Vernon Avenue Bridge will be used by the research team as a benchmark example showing the process can provide useful information for asset allocation, and be fairly simple using an established framework. There are several benefits to the bridge owner of having a structural model that reflects the actual bridge 3D system behavior that can be used for load rating and overload permitting.

This paper presents a collaborative research project currently funded by the Partnership for Innovation and the CAREER programs at the National Science Foundation to develop a framework for bridge condition assessment integrating instrumentation and structural modeling for highway bridge decision-making and management.

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Modeling and instrumentation of the Tobin Memorial Bridge

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ABSTRACT

The Maurice J. Tobin Memorial Bridge carries US Route 1 across the Mystic River, connecting the city of Chelsea and the Charlestown section of Boston, Massachusetts. Construction on the bridge began in 1948 and it was opened to traffic in 1950. The Tobin Memorial Bridge is a critical link in the network of Boston area highways, connecting metropolitan Boston with business and residential areas to the north. The approximately 2 1/4 mile long structure includes 32 approach spans on the Chelsea side, 36 approach spans on the Boston side, the Little Mystic Truss Span, the Big Mystic Cantilever Truss Span, and the Toll Plaza.

The Massachusetts Port Authority has contracted consultant services for the structural modeling instrumentation and monitoring of the Maurice J. Tobin Memorial Bridge. The consultant team, led by Fay, Spofford, & Thorndike, LLC, and supported by Tufts University, the University of New Hampshire, and Geocomp, Inc., began work on this project in January of 2009.

This paper describes the analytical structural models created for this project, including a global structural finite element model of the Little Mystic Truss and three supporting special studies focused on the rotational stiffness of the truss connections, boundary conditions, and built-up member properties.

To date, the team has developed a set of three dimensional finite element models representing components of the bridge. The combination of a global structural model and several detailed finite element models, created through a series of special studies, more closely represents the bridge structure's true system behavior. The global structural model incorporates the geometry of the truss members, the floor system, the concrete road decks, and end supports. The models from the special studies focus on the structural truss connections that are typically assumed to function as either a frictionless pin or fully-fixed joint in conventional structural design and analysis.

An instrumentation plan that included strain gages, accelerometers, tiltmeters, and temperature sensors was developed and deployed for the Little Mystic Truss during the winter of 2009. The instrumentation plan serves the purpose of verifying the finite element models and may eventually be used as part of a long term Structural Health Monitoring System. In early fall 2009 a load test was performed to help verify the models.

Work proceeds on refining the modeling work and evaluating the data.

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Instrumentation for reinforced concrete durability monitoring of Qingdao Bay Bridge

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ABSTRACT

The Qingdao Bay Bridge with a total length of 28,047 m is a sea-crossing bridge currently under construction in Qingdao, Shandong Province, China. As illustrated in Figure 1, it comprises a cable-stayed bridge (Cangkou Bridge) with a main span of 260 m, a single-pylon cable-stayed bridge (Hongdao Bridge) with two main spans of 120 m each, a single-tower suspension bridge (Daguhe Bridge) with two main spans of 190 m and 260 m respectively, and a series of approach bridges. The construction of the Qingdao Bay Bridge will be completed in 2011. As part of a sophisticated structural health monitoring and safety evaluation system being implemented on the bridge, a durability monitoring system is specifically designed to monitor corrosion-related parameters of the reinforced concrete structures, in recognition of severe coastal environmental conditions around the bridge such as about 50-times freezing-thawing cycling per year and 29.4‰ to 32.9‰ saline concentration.

The durability monitoring system consists of three types of corrosions sensors: (i) 13 sets of Anode Ladder Systems (ALS), (ii) 12 sets of Expansion Ring Systems (ERS), and (iii) 4 sets of Embedded Corrosion Instrument-1 (ECI-1). Two categories of corrosion-related parameters are measured. The first category is about the causes to induce and affect corrosion, such as temperature, relative humidity, and concrete resistivity. The second category is electrode assemblies characterizing the corrosion extent of steel bars, such as limit values for de-passivation, potential difference, and polarization resistance.

As listed in Table 1, the three types of corrosion sensors have been embedded in different spans and heights of the bridge (mainly in tidal zone, splash zone, and atmosphere zone). The corrosion status of reinforcement at different service zones and the effect of protective coating will be investigated using the long-term monitoring data from these sensors, which will benefit the bridge owners to optimize protective measures and prioritize maintenance activities targeting to prolong service life of the bridge. The data collected from different corrosion sensing systems will be compared, and the performance of the three sensing systems will be evaluated.

Figure 1. Qingdao Bay Bridge under construction.

Table 1. Deployment of corrosion sensors on Qingdao Bay Bridge.

Construction zones	Sensor	Monitoring position altitude A	B	C
1B	ALS	$2.0^{(1)}$	$4.0^{(1)}$	/
2	ALS	$2.5^{(2)}$	$5.0^{(3)}$	$7.0^{(3)}$
	ECI-1	$2.5^{(2)}$	$5.0^{(3)}$	/
3	ALS	$0.3^{(1)}$	$4.0^{(1)}$	/
4	ALS	$2.0^{(1)}$	$4.0^{(1)}$	/
5	ERS	$2.0^{(1)}$	$4.0^{(1)}$	$7.0^{(1)}$
6	ERS	$2.0^{(2)}$	$4.0^{(3)}$	$7.0^{(3)}$
7	ERS	$2.0^{(1)}$	$4.0^{(1)}$	/
	ALS	$2.0^{(1)}$	$4.0^{(1)}$	/
	ERS	$2.0^{(1)}$	$4.0^{(1)}$	/
	ECI-1	$2.0^{(1)}$	$4.0^{(1)}$	/
8	ALS	$2.0^{(1)}$	$4.0^{(1)}$	/
10	ERS	$2.0^{(1)}$	$4.0^{(1)}$	/

$^{(1)}$ — pier column; $^{(2)}$ — cushion cap; $^{(3)}$ — bridge tower.

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Structural monitoring of Lezíria Bridge since its construction

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ABSTRACT

Currently, automatic monitoring systems are increasingly used in the civil engineering structures especially in bridges with a high socioeconomic impact. In the last decades, the technology evolution that the world has being subjected allow, nowadays, the implementation of monitoring systems capable of observe the structural behavior in a programmable, remote and real-time manner. The databases originated by the sensors records are valuable information to understand better the real behavior of the monitored structures as well to improve the numerical models used in the analysis and prediction. This work presents the Lezíria Bridge case, equipped with an integrated and automatic monitoring system. This system allows the observation of the structural behavior and the materials durability. A presentation and discussion of a set of results obtained since the bridge construction is performed. The principal considerations refer to: (i) environmental effects – temperature and humidity; (ii) rheological effects of concrete – creep and shrinkage; (iii) events related with the bridge construction, long-term behavior and short-term behavior correlated to the bridge traffic. A final discussion is presented which is focused the usefulness of the information obtained by the monitoring system for the assessment and surveillance of the structure.

In this work it is intended to present the long-term monitoring system of Lezíria Bridge by a discussion of a set of results obtained since the construction. The gathered results provide valuable information for further study and for the understanding of the real behavior of the structure. Through an adequate data treatment and processing to a higher level of information abstraction, it is intended to demonstrate the utility of this type of systems to help on the assessment and surveillance of the structure.

Figure 1. Lezíria Bridge – north viaduct (top), main bridge (middle) and south viaduct (bottom).

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Reliability assessment oriented monitoring system design for Shanghai Yangtze River Bridge

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ABSTRACT

The Shanghai Yangtze River Bridge is part of the Chongming Cross River Passage Project connecting the Pudong district and the Chongming Island in Shanghai. The main navigation channel bridge is a double pylon double cable plane cable-stayed bridge with slotted box girder. The bridge is 1430 meters long with the following span alignment 92 m + 258 m + 730 m + 258 m + 92 m. Concerning its importance, the authority made a decision to install a bridge health monitoring system on the structure. Besides that, an inspection and maintenance plan is carefully made. It will be put into use when the bridge is open to traffic.

To fulfill the requirement for structure reliability assessment, the following items should be monitored:

a) Strain at the stress concentrated points for all critical sections of the girder and cable pylons;
b) Force of critical cables in the cable net system;
c) Acceleration of key sections for structural natural frequency and mode shape estimation.

Moreover, the environment and load should be monitored to calibrate structure load model and the deformation and rotation should be monitored to calibrate structure geometrical alignment. Figure 1 shows

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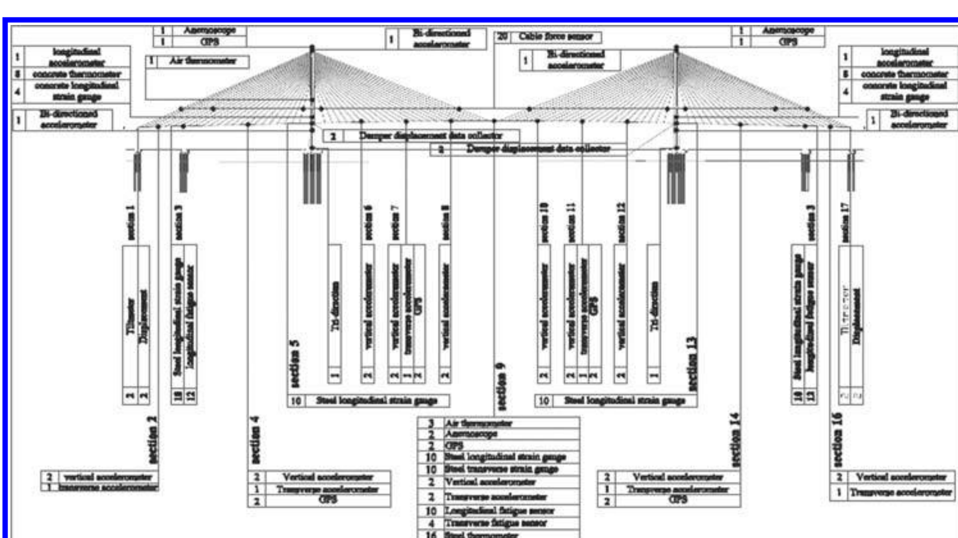

Figure 1. Sensor placement for Shanghai Yangtze River Main-navigation Channel Bridge.

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Bridge Sensor Mart: A flexible and scalable data storage and analysis framework for structural health monitoring

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ABSTRACT

This paper reports on the new sensor data collection, analysis and storage framework, Bridge Sensor Mart (BSM), for bridge health monitoring data. BSM defines a distributed data storage and data analytics infrastructure in order to collect, store, analyze and manage sensor data for Structural Health Monitoring applications (Ansari 2005).

SHM applications are gathering large amounts of data that are very hard to be managed with the current systems; thus making the dissemination and sharing of such data very cumbersome or even impractical. BSM architecture targets one of the biggest challenges in SHM applications: How to efficiently collect, store and provide the accumulated data to the end users and applications for further analysis? In this work, we present a new data storage and analysis framework, Bridge Sensor Mart (BSM), designed specifically for long term SHM.

The key features of this framework are:

1. Distributed Data Analytics: The post-processing of incoming data is usually performed on a single computer. The BSM framework allows distributing data analysis and data processing procedures across multiple machines allowing users to handle large amounts of data.
2. Distributed Data Storage: BSM also supports the distributed storage of accumulated data. This distributed architecture allows the system to be extended to handle larger scales of data by simply adding additional hardware components without changing the design or codebase.
3. Flexible Sensor Support: Current data management systems are designed to support specific sensing hardware or certain vendors. The BSM architecture is sensor- and provider-agnostic. The data import layer provides a generic interface that can be extended through a plug-in mechanism to support new sensor hardware.
4. Flexible Data Analytics Support: BSM provides a generic interface for users to plug-in their own data analysis algorithms, and to extend the analytical capabilities without requiring any changes in the system.

BSM is based on a very efficient hybrid database structure which combines and utilizes two kinds of databases: a scientific database for storing large amounts of historical sensor data and a regular row-oriented database for storing metadata. Simulations performed showed that BSM database structure outperforms traditional databases considerably for sensor related data.

BSM allows to robustly manage large amount of data, by implementing a fully distributed architecture for data gathering, analysis and dissemination. Thus, a single point of failure is avoided and the workload is distributed among several computers. This architecture promises high scalability as it is easy to extend the system by simply deploying new hardware (e.g. new server nodes) in order to lessen the pressure on highly loaded computers.

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Full-scale measurements on buffeting response of Sutong Bridge under typhoon Fung-Wong

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ABSTRACT

The field measurement of bridge buffeting response is essential to the research of bridge wind engineering. Sutong Cable-stayed Bridge (SB) is the longest cable-stayed bridge in the world. For such a long-span cable-supported bridge, the structural stiffness drops significantly with the increase of the bridge span, which makes wind-induced vibration particularly important for structural safety.

Taking into account the importance of SB, Structural Health Monitoring System (SHMS) was established to predict and assess the health condition of SB during its construction and operation stage, especially under the affects of various disasters. Typhoon Fung-Wong attacked the SB in July 2008. During the typhoon period, field measurement was conducted to record wind velocities; SHMS of the SB was used to collect the structural vibration responses.

In this paper, the measured data are studied using spectral analysis and statistical method. The study includes the analyses of wind characteristics on bridge site and the response characteristics of stay cable and the deck, the relationship of vibrations and wind speed, the comparison of upstream and downstream cable vibrations, the effectiveness of cable dampers and so forth. The spectra of the upstream cables are plotted in Figure 1.

The results validate the stability and reliability of the SHMS; indicate the acceleration RMS values become larger as the wind speed increases. The calculated dynamic characteristics agree with the measured ones well, validating the reliability of the FE model. Moreover, the comparison between the measured results and data from other studies reveals the effectiveness of dampers installed around anchor ends of the cables. The results can be employed to validate the current buffeting response calculation method, and provide references for wind-resistant design of the super-long-span cable-stayed bridges.

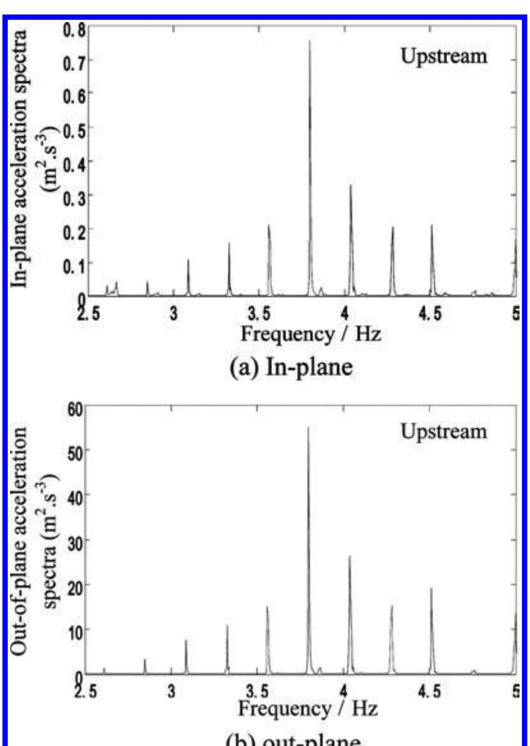

Figure 1. Acceleration spectra of upstream cables.

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Structural health monitoring for damage detection based on integration of computer imaging and sensor data

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ABSTRACT

Novel structural health monitoring strategies for better management of civil infrastructure systems (CIS) are increasingly becoming more important as CIS structural performance decreases due to reasons such as damage, over loading, severe environmental conditions, and aging due to normal continued use. Structural Health Monitoring (SHM) paradigm is designed to provide objective information for decision-making on safety and serviceability and it is expected to complement the current visual and heuristic based assessments. SHM utilizes advanced technologies to capture the critical inputs and responses of a structural system in order to understand the root causes of problems as well as to track responses to predict future behavior. Very recently, some investigators have explored the possibility of incorporating imaging and optical devices and combining them with sensing technology. In this paper, the use of computer vision, sensing, damage indices and statistical analysis methods are presented and demonstrated in the UCF 4-span bridge model for damage identification in the context of structural health monitoring. The framework of this study is given in Figure 1 and the test setup in Figure 2.

Two different remote controlled vehicles under various loading scenarios crawl over the bridge while a video camera supervises the structure providing traffic video stream. At the same time, a distributed array of sensors collects data.

Correlation between moving vehicular load and structural responses is determined as Unit Influence Lines (UIL) are extracted and used as an index for monitoring the bridge behavior. Different damage scenarios were considered and studied for the UCF 4-span bridge. These cases were chosen because they represent some of the most common issues affecting bridges, according to the Department of Transportation (DOT) engineers based on our private discussions. Rusted supports, stiffness reduction and loss of connectivity between composite sections are experimentally simulated, processed, and analyzed by means of statistical methods. After application and processing the different damage cases, influence lines were obtained. A Mahalanobis-distance-based outlier detection algorithm is used to identify changes in the structure, and results are analyzed.

Finally, a new method to more effectively identify, localize and quantify (in a relative sense) induced damage by using a new index called normalized distance "N_d" is derived and demonstrated. Results are presented and discussed.

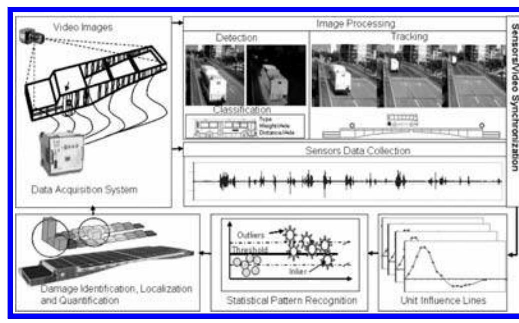

Figure 1. UCF-4-Span Bridge.

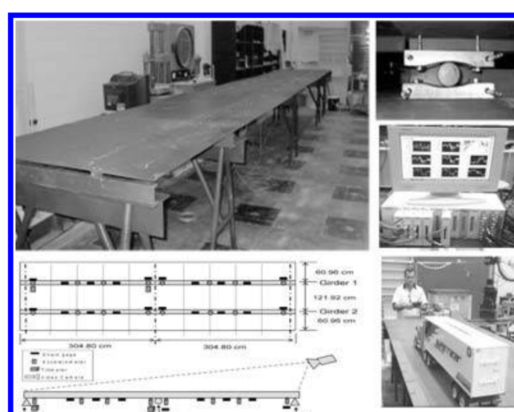

Figure 2. UCF-4-Span Bridge.

MS5: New procedures for bridge rehabilitation
Organizer: V. Popa

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Sustainable and cost effective solutions to life extension of bridges

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ABSTRACT

In order to minimise costs and disruption associated with repair it is necessary to minimise the amount of concrete removed wherever practical. This also benefits the environment, with less material going to landfill, and less fresh material being employed. Cathodic protection (CP) can be used as a repair technique for chloride contaminated concrete. The main benefit is that it is no longer necessary to remove all the chloride contamination. This minimises the extent of repairs to the replacement of spalled material and can avoid the need for costly temporary support during the repair process.

Chlorides commonly derive from de-icing salts and/or were formerly used as concrete admixture. Chloride contaminated reinforced concrete structures mainly suffer from cover concrete delamination and loss of reinforcement cross-sections. By using CP there is no need to remove all of the chloride-contaminated concrete reducing the time for access, repair and traffic management.

CP involves polarizing the reinforcement in an electrical circuit. The electrical circuit converts the reinforcement into the cathode and an inert electrode (e.g. mixed metal oxide coated titanium) forms the artificial anode which can be installed discretely or on the surface.

This paper presents an outline of a number of UK based projects where CP has formed a key part of a maintenance strategy, highlighting the decision processes involved and lessons learned for the future. These are:

- M4 Elevated Freeway, London.
- Silver Jubilee Bridge, Runcorn.
- Theale Railway Bridge, London.

Repair strategies are mainly governed by access restrictions at the structures. The system installed at the Silver Jubilee Bridge was chosen as to be the most suitable system to withstand bridge vibrations, reducing installation time and avoiding long lasting traffic management. The system designed for the M4 in London was governed by access and noise restrictions. Every structure needs to be assessed for their local restriction and the most appropriate system needs to be identified by the engineer. Conventional systems such mesh and overlay and discrete anodes can be used most of the time, however, to go one step forward and reduce the environmental footprint a little thought and research is necessary. Innovative CP systems are able to reduce installation time and the amount of materials used as well as a significant life extension of bridges. For specific site and CP system requirements traffic management, working at height and exposure to vibration and noise can be limited.

Over the years that CP systems have been designed, installed and subsequently monitored, it was observed that they have generally exceeded the performance criteria with ease, Lambert (2009). After the first 12 months of operation, cathodic protection systems stabilize and require little intervention provided the monitoring probes continue to give consistent and stable readings. The experience gained from each system has been used to refine the design of the following applications, resulting in bigger zones, less anodes and reference electrodes, and the development of hybrid protection systems to overcome local difficulties.

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Electrochemical chloride extraction and electrochemical re-alkalization Foreva Regebeton PA process

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ABSTRACT

Corrosion is a main issue of reinforced concrete structures with regards to durability. Due to the important porosity in concrete, corrosion protection is not guaranteed and can fail if atmospheric carbon dioxide (CO2), or chloride contamination penetrate the concrete.

Foreva Regebeton PA is a process dealing with both carbonation and chloride ions attack. With the same electro-chemical treatment applied to the corroded reinforced concrete, chloride ions are partially extracted from concrete and concrete is re-alkalized (means pH getting values back to 12 or 13) around the reinforcing steel in the concrete.

Foreva Regebeton PA gives back a high alkalinity environment close to reinforcing steel. The electrolyte solution contains hydroxyl ions (OH^-) that are trapped by reinforcing steel during the treatment. The increase of pH is less farther from reinforcing steel since hydroxyl ions will go close to reinforcing steel. The sketch below shows the principle of the re-alkalization treatment:

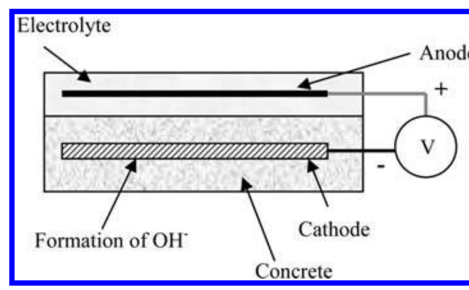

Figure 1. Re-alkalization process.

Foreva Regebeton PA can be applied if chlorides are coming from the outside and are mainly present in the cover depth. Chlorides farther than reinforcing steel from the exposed face of the structure cannot be satisfactory extracted. This treatment is therefore applied in the case of sea salt or deicing salt contamination. During the treatment, the chloride ions move toward the external anode embedded in an electrolytic solution and they are then trapped in the electrolytic solution. The sketch below shows the principle of chloride extraction treatment:

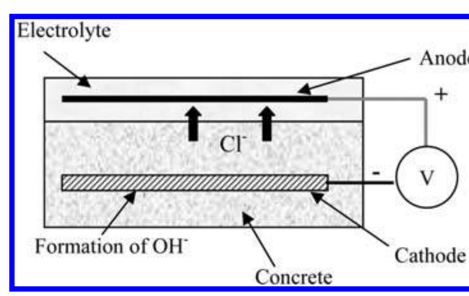

Figure 2. Chloride extraction process.

Foreva Regebeton PA can be applied even if the structure is corroded. The treatment duration is from three to four weeks. An impressed current (low amount of current) is applied to shorten the treatment duration. Lifetime of the structure is increased. In the case a protective coating is applied afterwards, lifetime is extended farther. Aesthetic is unchanged: the product is fully removed at the end of the treatment. Both effects (re-alkalization and chloride extraction) are performed with one application.

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Bridge crossings raised to provide more overhead clearance

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ABSTRACT

A frequent dilemma regarding old bridges is a choice between adaptation to new, usually higher requirements; and replacement by new structures. This choice is not only a matter of structural strength and functional fitness; it also has environmental, esthetical, heritage-related and other impacts. The Netherlands is a country of an intensive inland navigation that is considered economically and ecologically preferable to other modes of transport. Navigation is, therefore, another important issue for this choice.

In order to allow high loaded barges pass the Juliana Canal, the main waterway to Belgium and France, two old bridge crossings – Roosteren and Echt – were raised by over 3.0 m. This solution proved to be preferable to new construction in all respects mentioned above. Also its costs proved to be lower than a new construction. A raising project of that size required, however, a thorough investigation of both the condition and behaviour of the old bridges. Moreover, since the bridges had a continuous 3-span system, middle arch-span 80.0 m long, the raising had to be well synchronized in order not to generate excessive loads. An important issue was also the remaining service life. If it was not long enough, new construction might still have been a better choice. To assess such risks, load tests were performed (Fig. 1 top).

The raising was performed using the technology of hydraulic jacking-up. 12 jack-up units, 2400 kN working load each, simultaneously lifted the bridge superstructures to the desired level (Fig. 1 bottom). The full paper presents the details of this operation, followed by the substructure leveling and finishing works. The method proved not only to be very effective and reliable, but also environment-friendly. It consumed only a fraction of the materials and energy which would have been required by other solutions. Both bridges perform well in raised position.

Figure 1. Bridge load testing (top) and jacking-up the superstructure (bottom).

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Choosing the appropriate sustainable polymer concrete material for bridge preservation and maintenance

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ABSTRACT

Today engineers and owners are faced with a variety of available specialty materials for a wide range of applications on every bridge rehabilitation project. Sustainable polymer concrete products and material systems developed by industry with input from engineers have proven their ability to extend the service life of existing structures. Applications of polymer products and material systems on newly constructed structures can reduce maintenance and increase traveler safety.

Both, polymer and concrete technology are an important part in choosing the appropriate polymer product or polymer concrete material system to meet a project requirement. Material physical properties, installation procedure, construction time, material cost, weather limitations, performance requirements, and field requirements as well as safety and environmental issues are all factors that must be considered in order to select the proper material.

Polymer concrete specified and used today for the sustainability of highway structures can be broken down into several different resin types:

- Epoxy
- Modified Epoxy
- Methyl Methacrylates
- High Molecular Weight Methacrylates
- Polyesters

These basic resin types can be formulated to develop different physical properties which make them versatile when they are to be used for a variety of applications. Some of the typical areas where they are used with success are:

- Spall Repair
- Joint Headers
- Bearing Pads
- Wearing Surfaces Overlays
- Skid Resistance Surfaces
- Crack And Surface Sealing

Each of these applications has their own unique material requirements that must be considered before a decision can be made in selecting the proper material to be specified. It is important that an engineer not only consider the material physical properties requirement but they also need to consider the jobsite application conditions. Some of the key requirements to be considered when selecting the proper polymer material are:

- Compressive Strength
- Flexural Modulus
- Elongation
- Viscosity
- Temperature Limitations
- Cure Time
- Required Mixing and Installation Equipment

This paper will give engineers and owners the knowledge they need to make an informed decision on which of these products and material systems to choose, and what variables should be taken into consideration for an array of different repairs and safety concerns including but not limited to Spall Repairs, Joint Headers, Bearing Pads, Wearing Surface Overlays, and Crack and Surface Sealing.

Several successful preservation, rehabilitation and new construction installations will be discussed including spall repair, thin bridge overlay and concrete sealing projects.

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Shear resistances and strengthening of aged prestressed concrete bridges considering deterioration and fatigue effects

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ABSTRACT

The major part of European road bridges was built of prestressed concrete, about forty to twenty years ago. Compared to nowadays standards, they lack of resistances due to increased live loads, temperature effects and detailing provisions. Moreover, deterioration effects weaken the structures. Often, a pronounced diagonal shear cracking arises at the supports giving rise to subsequent strengthenings and recalculations of the present shear resistances in the focus of degradating influences as well as realistic live load scenarios during the previous and remaining live time.

In the paper an elaborated shear design approach is presented to estimate remaining shear resistances under fatigue actions. It accounts for the – at the time of bridge erection – typically used strong prestressing actions in combination with low amounts of shear reinforcements. The governing resistance of the tensile strut is summed up from the partitions of the concrete section, the stirrups and arch actions introduced by the compressive axial forces (Fig. 1). Fatigue degradations are modelled within the first two partitions by time variant weighted load-cycle factors with respect to concrete and steel strengths. The fatigue shear loading is seperately assigned to the contributions, while the stirrups are activated at last. The governing equations are derived, verified to experimental data and compared to approximations in design codes.

In the second part of the paper a shear strengthening method for multi-span bridges is presented that uses external tendons. The tendons are vertically deviated by specific steel devices near the coupling joints (up) and at the supports (down) achieving a stepwise linear course and pronounced relieving shear forces. The steel devices ensure direct couplings of horizontal deviation forces and suitable vertical load applications from underneath the bottom slab and by anchor bolts near the deck slab and the transversal girders to account for the limited amount of vertical reinforcement. The principles of the design calculations as well as details of deviation devices, tendon geometry and anchorage constructions are presented.

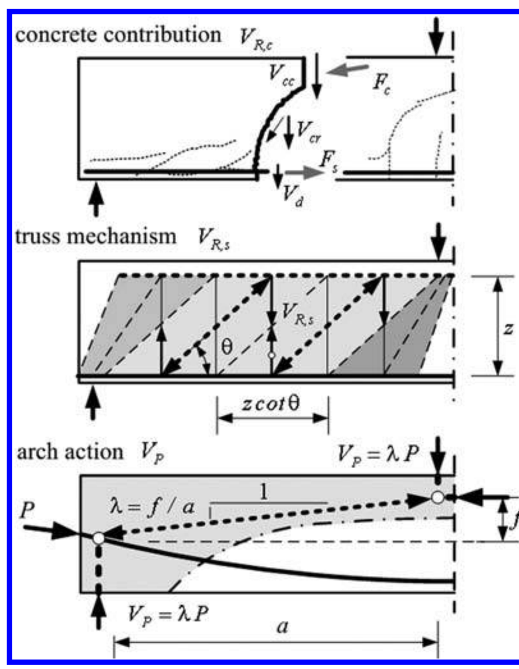

Figure 1. Shear mechanisms of members with or without shear reinforcement.

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Fatigue strength and repair methods of corroded bridge wires

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ABSTRACT

Bridge cables in suspension and cable-stayed bridges are under severe corrosion environments due to water and high temperature inside the cables. Fatigue strength of corroded galvanized steel wires was investigated.

Corroded galvanized steel wires on three corrosion levels were produced at laboratories. Then, fatigue strength of corroded wires was obtained by cyclically loading under dry and wet environments. Fatigue strength did not change when only the galvanized layer was corroded, but it significantly decreased after the steel corrosion progressed. The fatigue strength under wet conditions was lower than that under dry conditions due to corrosion fatigue.

Pit sizes were measured in the wire specimens on corrosion levels-2 and 3. The maximum depth is 5.6 mm, and both pit length and width are within 10 mm. Fatigue tests were conducted for the specimens with artificial pits of 0.6 mm deep and 3.5 mm long. They have three different pit shapes; round (group-P), triangle (group-Q) and triangle with notch (group-R). Each group has three lengths; 3.5, 6.0 and 10 mm.

The group-P specimens with round shape pits did not break until the limit cycle of one million.

The group-Q specimens with triangle shapes broke at the fewer cycles for the shorter pit length. At a pit length of 3.5 mm the wires broke at cycles between 4,800 and 36,000. At a pit length of 6.0 mm the wires broke at cycles between 49,000 and 72,000. At a pit length of 10.0 mm the wires did not break. The critical cycles increase with pit length.

The critical cycles did not depend on pit length for the group-R specimens with triangle shapes with a notch. All of the wires broke at cycles between 18,000 and 36,000.

From these results of the groups-P, Q and R it can be understood that the pits with a triangle shape and with a notch has the lowest fatigue strength. This is because stresses concentrate on a notch at the sharp edge of a triangle and initiates a fatigue crack. The triangle shape pit also decreases the fatigue strength but it depends on the ratio of pit length to width. This ratio also relates with stress concentration factor. The round shape pit is safer because stress concentration is less likely to occur.

The S-N data of the triangle pit specimens (group-Q) and the notched triangle pit specimens (group-R) are on the extension S-N line of the level-3 corrosion wires. This verifies that the pit shape is a dominant factor to lower the fatigue strength.

Cables are the crucial element in cable-supported bridges. Once cables are corroded, it is not easy to repair or replace them. However, not many studies have been conducted so far to confirm effectiveness of various repair methods and no perfect one has been established.

Seven cases were studied in evaluating effectiveness of different repair methods of corroded wires: (1) taking no measure, (2) coating only the surface wires with epoxy resin paint, (3) coating only the surface wires with zinc rich paint, (4) coating and filling the surface and inside wires with oil containing inhibitor, (5) coating and filling the surface and inside wires entire wires with epoxy resin, (6) coating the surface layer with thick paste containing zinc powders, and (7) dehumidifying the inside of a cable.

In most of the cases mass loss of the inside wires due to corrosion was much less than that of the surface wires. As for the surface wires the dehumidification method (case 1) was the most effective followed by the epoxy resin paint of surface and inside wires (case 5), the zinc powder paste (case 6), and the zinc and epoxy resin paint of surface wires (case 2 and case 3). The oil filling (case 4) was not very effective compared with other repair methods. The tests show that the dehumidification method and the epoxy resin paint and filling of surface and inside wires seem to be very effective to repair corroded wires.

MS6: Measurement systems for bridge weigh-in-motion (B-WIM)

Organizers: B. Bakht, A. Znidaric & D.K. McNeill

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Bridge capacity assessment by combined proof-loading and WIM data

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ABSTRACT

The increasing volume of European transport urgently requires an effective road and rail system in Central and Eastern countries. To bring this transport infrastructure up to modern European standards will require an immense investment (estimated by the European Commission to be about €100 billion), and therefore difficult to achieve in the medium term. New motorways will be required with many new bridges. Numerous existing bridges will need to be assessed, and a large portion of them improved or replaced.

The overall goal of the European project ARCHES (Assessment and Rehabilitation of Central European Highway Structures) was to reduce the gap in the standard of highway infrastructure between Central and Eastern European Countries (CEEC) – particularly New Member States (NMS) – and the rest of the EU. This key problem was addressed by a combined approach. As part of it, takes great importance the development of more appropriate bridge capacity assessment tools and procedures to avoid unnecessary interventions (repairs/replacements) in the existing road network (Casas 2010, ARCHES-D16 2009). In the paper, proof loading tests are presented as one of these tools. The correct application of the method should combine an accurate execution and monitoring of the loading process and a good estimate of the actual traffic in the bridge. The last is of crucial importance to decide on the target value of the load to be introduced by the test. This can be achieved by the most advanced WIM techniques available, also developed and applied within the ARCHES project (ARCHES-D08 2009).

The target proof load to be introduced in the bridge during the test can be evaluated on the basis of reliability theory as many variables involved are of random nature. The target proof load can be obtained as the nominal load in the Code multiplied by a so called proof load factor. In order to get values of the proof load factor representative for several countries from Central and Eastern Europe to facilitate the execution of such tests, actual traffic data is mandatory. The paper shows how the traffic data from 5 European countries (The Netherlands, Czcech Republic, Poland, Slovakia and Slovenia), obtained via WIM, has been used to propose a set of proof load factors applicable to the existing bridge in those countries (Gómez & Casas 2010, ARCHES-D16 2009). Evidently, in order to be general, they are based on the most heavy traffic conditions that can be encountered. Therefore, the proposed target proof load factors may become too conservative for many bridges located in local or secondary roads that will never experience such level of loading. For this reason, additionally, the paper presents a simplified method that with very common traffic data easily recordable by WIM systems may predict accurately the most critical traffic actions for any specific bridge under assessment. The simplified model has been checked by comparing its results with those derived with a complete simulation process for a long time period.

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Using weigh-in-motion data for modeling maximum live load effects on highway bridges

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ABSTRACT

This paper recommends a procedure to estimate the maximum expected load effect on a highway bridge. The procedure describes how site-specific truck weight and traffic data collected using Weigh-In-Motion (WIM) systems can be used to obtain estimates of the maximum live load for the design life of a bridge, specified to be 75 years as per the AASHTO LRFD code, or the two-year return period that may be used for the load capacity evaluation of existing bridges.

The model requires as input the WIM data collected at a site after being "scrubbed" and processed as described by Sivakumar et al (2008). The process begins by assembling the measured load effects histograms for single lane events and two-lane events. The cumulative distribution function for each load effect is used to obtain the standard deviate of the cumulative function. A plot is made of the upper 5% of the values of the normal deviate versus the load effect X as illustrated in Fig. 1.

The slope m and intercept n of the best fit regression line provide the statistics for the normal distribution that best fits the tail end of the distribution. The mean of the Normal that best fits the tail end of the distribution is obtained from $\mu_{\text{event}} = -n/m$. The standard deviation of the best fit Normal is $\sigma_{\text{event}} = 1/m$. Given a total number of events in the return period of interest, N, the most probable value, u, and the dispersion coefficient for the Gumbel distribution that models the maximum value L_{max} are:

$$u_N = \mu_{event} + \sigma_{event} \times \left[\sqrt{2\ln(N)} - \frac{\ln(\ln(N)) + \ln(4\pi)}{2\sqrt{2\ln(N)}} \right] \quad (1)$$

$$\alpha_N = \frac{\sqrt{2\ln(N)}}{\sigma_{event}} \quad (2)$$

The mean value of the maximum load effect, L_{max}, and the standard deviation are obtained from:

$$L_{max} = \mu = u_N + \frac{\gamma}{\alpha_N} \quad (3)$$

in which γ is the Euler number $\gamma = 0.577216$. The standard deviation is:

$$\sigma^2_{L\max} = \frac{\pi^2}{6\alpha^2_N} \quad (4)$$

The procedure can be used to obtain the maximum load effect for any return period to design and evaluate the safety of bridges in different jurisdictions where the AASHTO LRFD loading may not be representative of current loading due to variations in truck traffic and truck weight conditions.

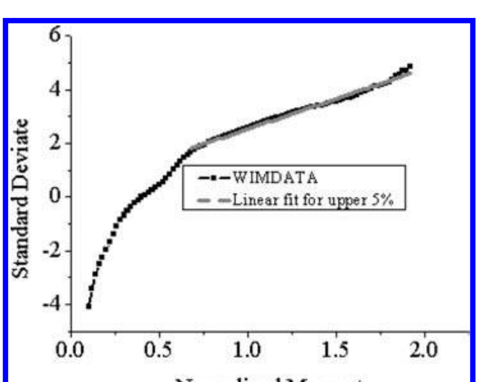

Figure 1. Linear fit of tail end of load effect histogram on Normal probability paper.

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Bridge weigh-in-motion on steel orthotropic decks – Millau viaduct and Autreville bridge

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ABSTRACT

Bridge weigh-in-motion (B-WIM) is a technique which uses an instrumented bridge to weigh heavy vehicles in motion. Several types of bridges may be used for that, but they shall be sensitive to traffic loads, above all wheel or axle loads. The longitudinal stiffeners of steel orthotropic deck bridges bend between two cross beams when wheels and axles are passing on them. The bending strains are measured by extensometers. The principle of "free of axle detector" (FAD) B-WIM, introduced by (Žnidarič et al. 1999) is used, with two transverse sections of the bridges instrumented to get the vehicle speed and axle spacings. A global optimization software introduced by (Dempsey et al. 1999) calculates axle loads and gross vehicle weights. First experimentations were carried out in the European cooperative WAVE project in the 90's, and in 2009 a large scale test was done on the Millau viaduct in France, the tallest cable stayed bridge in the world, on the motorway A75, using a commercial B-WIM system SiWIM.

The deck is a steel box orthotropic deck, 32 m in width and 4.20 m in height, which carries 2 lanes and an emergency lane in each direction. The instrumented section is close to the North end of the viaduct in the first span. 12 extensometers were stuck on the bottom of longitudinal stiffeners at mi-span between two cross beams under the slow lane, and partly under the adjacent emergency and fast lanes, for the weighing section. Four additional extensometers were stuck in the previous section 4 m upstream under the slow lane.

The SiWIM was calibrated over 1.5 day with two rented trucks, a rigid 3-axle and an articulated 5-axle truck, which made repeated runs over the viaduct at two speeds (60–80 km/h) and three transverse locations. Then 52 trucks from the traffic flow were measured by the SiWIM, identified by an observer and stopped at the toll gate 5 km downstream, to be weighed in static on an approved scale (axle load and gross vehicle weight). After a careful check of the collected data, 43 of these trucks were kept as correctly identified on both systems (B-WIM and static).

Table 1. Statistics of the relative errors of the truck sample and accuracy classes (Millau, 2009).

Criteria	Number	Mean	Standard Dev.	Class
Gross weight	43	−3.24%	5.76%	C(15)
Single axle	86	1.09%	11.18%	D + (20)
Group of axles	39	−8.01%	5.32%	C(15)
Axle of group	115	−7.93%	9.46%	C(15)

The relative errors of the axle loads and gross vehicle weights and their statistics were calculated, and then the European specifications on WIM (Jacob et al. 2002) were applied to assess the B-WIM system accuracy (Table 1).

The system meets the accuracy class C(15)/D +(20) of the European specifications. These results were compared to those of the test carried out in 1997–98 on the Autreville bridge in eastern France, another orthotropic steel deck bridge (Dempsey et al. 1999), and were found very consistent with them. However, the past results analyzed with a 2-D algorithm instead of the 1-D standard algorithm, which is implemented in the SiWIM, gave a better accuracy. Therefore it was concluded that there are perspectives to improve the SiWIM accuracy on orthotropic deck bridges by developing a new algorithm.

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Winnipeg weigh in motion

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ABSTRACT

Fatigue analysis is an important consideration for evaluating the health of the bridges. To develop a fatigue analysis, load and resistance are two essential concerns. In fact, load due to traffic vehicle differs considerably from the designed load. To estimate the remaining fatigue life more accurately, a refined evaluation load procedure is desirable. In this paper, we propose a Winnipeg method to incorporate bridge Weigh-in-Motion (WIM) technology into Structure Health Monitoring (SHM) data analysis to understand the performance of the bridges and establish fatigue life of the girders and the decks. The method chooses several corresponding strain peaks to calculate vehicle parameters, such as truck speed, axle spacing, axle weights, and gross weight. The calculated truck parameters are used as input for a program of Semi-continuum method of analysis for bridges (SECAN) to check the calculated result and to study the performance of bridges. This allows utilization of the output of SECAN for input in the fatigue analysis to estimate the fatigue life.

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The influence of correlation on the extreme traffic loading of bridges

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ABSTRACT

Accurate traffic loading models based on measured WIM data are essential for the accurate assessment of existing bridges. There are well-established methods for the Monte Carlo simulation of single lanes of heavy vehicle traffic, and this can easily be extended to model the loading on bridges with two independent streams of traffic in opposing directions. However, a typical highway bridge will have multiple lanes in the same direction, and various types of correlation are evident in measured traffic, such as groups of very heavy vehicles traveling together and heavy vehicles being overtaken by lighter ones. These traffic patterns affect the probability and magnitude of multiple presence loading events on bridges, and are significant for the maximum lifetime loading on the bridge. One example of these patterns is shown in Figure 1 which suggests that when heavier trucks are in the fast lane, they tend to be passing another truck in the slow lane.

This paper analyses traffic patterns using multi-lane WIM data collected at two European sites. It describes an approach to the Monte Carlo simulation of this traffic which seeks to replicate the observed patterns of vehicle weights, same-lane and inter-lane gaps, and vehicle speeds. A method is presented which applies variable bandwidth kernel density estimators (Scott 1992) to empirical traffic patterns of vehicle weights, gaps and speeds. Traffic scenarios are identified in the stream of measured traffic and these are used as the basis for a multi-dimensional smoothed bootstrap approach (Efron & Tibshirani 1993) which allows the observed correlation structure to be accurately simulated but also allows for unobserved patterns to be simulated. The process has been optimized so as to make it possible to simulate traffic loading on bridges over periods of 1000 years or more, and this removes much of the variability associated with extrapolating from shorter time periods to calculate the characteristic maximum lifetime loading. The effects of correlation on characteristic maximum bridge load effects are illustrated in Figure 2 for two-lane traffic on a 45 m bridge which shows that the loading event with one truck in each lane $(1+1)$ governs at the 1000-year return level, and that the modeling of correlation has a noticeable effect on all multi-truck loading events.

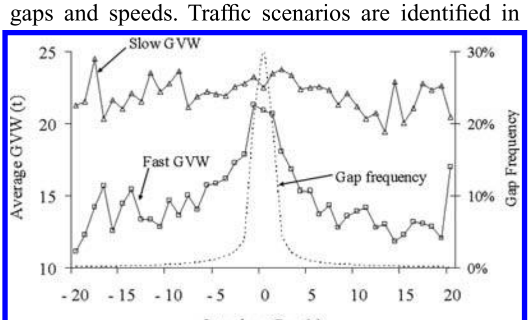

Figure 1. Inter-lane GVW correlation, the Netherlands.

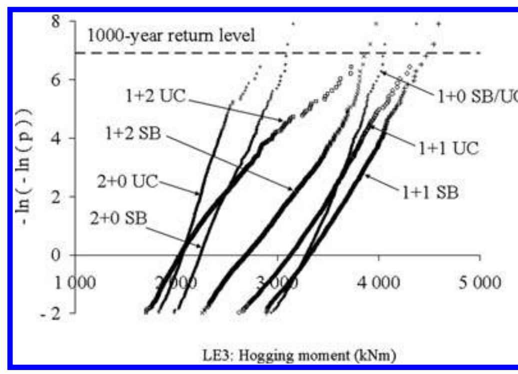

Figure 2. Annual maxima – smoothed bootstrap (SB) and uncorrelated model (UC).
Note: "$i+j$" denotes i trucks in the slow lane and j in the fast lane

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A filtered measured influence line approach to bridge weigh-in-motion

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ABSTRACT

In Bridge Weigh-in-Motion (B-WIM), an instrumented bridge is used as a scales to weigh passing trucks and their axles. The most common algorithm upon which modern B-WIM systems are based remains that developed by Moses (1979). The performance of this method is well documented; it is very good at estimating Gross Vehicle Weight, but less accurate for individual axles, particularly closely spaced axles on longer bridges. Many alternatives to Moses's original algorithm have been tested and some show the potential to improve accuracy but commercially available B-WIM systems are still based substantially on the original approach.

This paper proposes a method of altering the B-WIM algorithm to improve the accuracy of the predictions. The measured dynamic signal, to which the algorithm is applied, is first filtered to remove high frequency components of the dynamic increment of load, but filtered at a much lower threshold – about 3 Hz – than is commonly used. The objective is to remove those parts of the signal associated with the bridge first natural frequency.

The influence line used by the algorithm is also calculated differently. As previously described by OBrien *et al.* (2006) it is determined using a pre-weighed calibration truck and an algorithm to automatically convert the corresponding measured signal into a 'measured' influence line. However, for this work, the measured signal is first filtered to remove much of the high frequency dynamic components. The exact theoretical influence line is illustrated in Figure 1 and the measured filtered influence line in Figure 2. In this way, Moses's least squares fitting method is now comparing only the low frequency components of the measured and theoretical responses and produces a much more accurate fit.

The new approach is tested in numerical models and it is shown to result in a substantial improvement in accuracy.

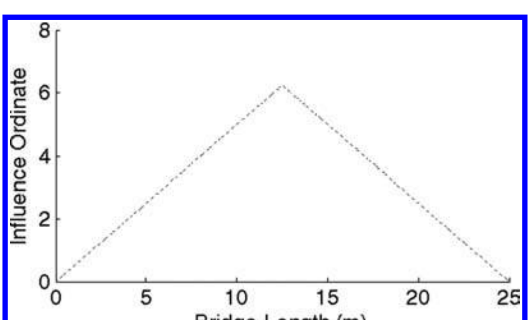

Figure 1. (Ideal) Influence Line.

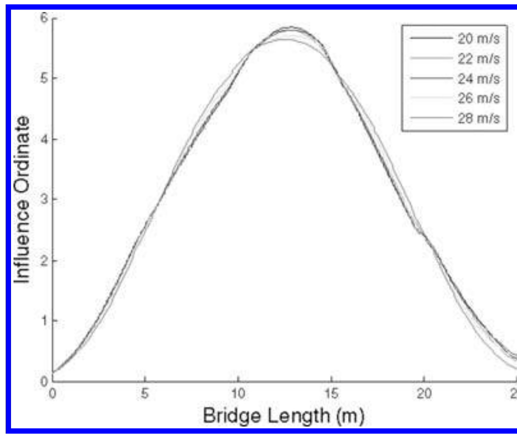

Figure 2. Filtered Measured Influence Lines for a range of vehicle speeds.

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Evaluating remaining lifetime of bridges by means of BWIM

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ABSTRACT

In the evaluation of bridges often considerably more attention is paid to the modelling of the structure than to the load – this statement mainly applies on the area of traffic loads. Considering the increasing amount of bridges with a distinct damage pattern, state evaluations will become more important, because also economic considerations depend on them since an immediate intervention is not always possible. Due to ongoing deterioration the requirements of relevant codes, which are often mounted on highway traffic, can not be fulfilled. Therefore it is necessary to gain more information concerning real load and resistance in order the estimate the failure probability of the structure and to gain decision criteria.

This paper will deal with the analysis of the remaining lifetime of bridges, focusing on a concrete example. The model that is displayed employs a stochastic format for analysis and the dominance of load assumptions in the limit state formulations will be pointed out. In consequence of that, the successful way of using realistic traffic loads will be taken from BWIM measurements, where BWIM is short for Bridge-Weigh-In-Motion. The data gained at the BWIM-measurement are used to form an axle load and traffic flow model where the Heavy Goods Vehicles (HGV) are artificially generated. To gain information on the load impact on the bridge the load step method, see [1], is applied and simulated for *n* times. Using the outcomes in a "semi probabilistic code manner", characteristic values are evaluated by means of extreme value statistics. On the other hand the distribution of extreme values can also be used in a fully probabilistic format where of course the deterioration of the bridge has to be considered.

In this paper a damaged bridge that can only be kept operational for a limited period is examined. The rated values for bending moment and shear force in critical cross-sections are predicted with help of the theory of extreme values as well as ongoing deterioration due to corrosion is considered. With a probabilistic analysis the development of the safety index is calculated and decisions for further actions are derived. The model for the consideration of further findings from following inspections is presented.

The bridge overpasses a highway and is also used as entrance and exit ramp of the highway. In 2007 this bridge already was evaluated in a similar manner, but mainly data from the literature were used and data from a highway was used as a basic to generate a load model, where trailers and semi trailers are dominant vehicle groups, see [2]. Now BWIM-measurements showed that at this bridge the local traffic is more important than the traffic from the highway. The fraction of overloads was quite high.

Within this paper failure will be described as a fault tree, consisting of elements shown in figure 1.

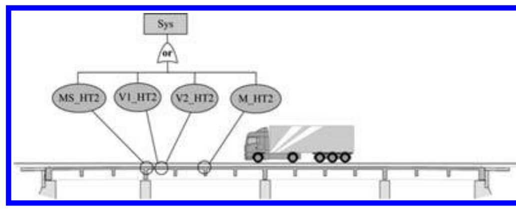

Figure 1. A fault tree applied on a bridge.

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WIM data to assess consequences of new traffic regulations

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ABSTRACT

The French traffic regulations allow common articulated trucks and road train to weigh 40 tons on 5 axles. However, there are several exceptions to these rules, with a weight limit of 44 tons for 40 ft ISO containers in a combined rail-road transport, trucks picking or delivering goods in harbors 150 km around them, or some agricultural good transport. Log trucks are allowed up to 48 tons on 5 axles and 57 tons on 6 axles. Following the law "Grenelle de l'environnement" on greenhouse gas saving in 2008, the Parliament asked the government to study the advantages and disadvantages of increasing the general weight limit to 44 tons.

To assess and compare the impacts of current traffic loads and future traffic loads with 44 ton trucks on bridges, WIM data recorded in 2009 on a heavily trafficked motorway in south of France were used by a software developed in the LCPC (Eymard & Jacob 1989) to calculate extreme load effects and fatigue damages on two composite bridges. These WIM data were then modified by a micro-simulation based on simple assumptions on the increase of some truck weights from 40 to 44 tons.

The stress influence lines of details, such as the welds between vertical stiffeners and lower flanges of steel main girders, for a 40 m single span simple supported bridge (Auxerre) and a 4 continuous span bridges (Libourne) were used. They have already been used for the calibration of the Eurocode 1 (Bruls & al. 1996, CEN EN1991-2 2003). The level crossing histograms of stresses were calculated and used, with the Rice's formula, to assess the extreme stress values for various return periods from 25 to 5000 years (Flint & Jacob, 1996). Rain-flow histograms were also calculated and used with the Miner's law and S-N curves (CEN EN1993-1-9 2005) to assess the detail lifetime in fatigue (Jacob and Kretz 1996).

The modified traffic with 44 ton trucks induced maximum stresses 6.5 to 8.5% higher than the recorded traffic. The detail lifetime are decreased, up to 20%, with these increased traffic loads (Table 1).

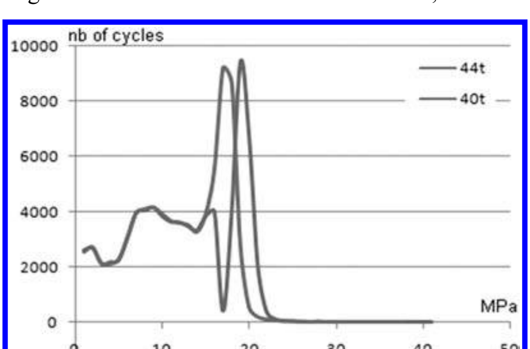

Figure 1. Rain-flow histogram of the stresses in Auxerre bridge at mid-span, lower flange, both traffics.

Table 1. Auxerre & Libourne bridge lifetime for A9 traffic, 40 or 44 t gross weight limit.

Auxerre Fatigue class	Lifetimes (in years) 40 tons	44 tons
Class 56	10.1	8.5
Class 63	14.3	12.0
Class 71	20.1	16.9
Libourne	40 tons	44 tons
Class 50	34.6	28.9
Class 56	48.9	40.8
Class 63	69.0	58.8

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Vehicle loading and effect on the Tsing Ma Bridge using WIM data

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ABSTRACT

Many innovative long span bridges made of new materials and implemented by new technologies have been built in recent years throughout the world. These long span bridges surrounded by harsher environment than ever before engender many challenges to professionals on how to ensure these structures function properly during their long service lives and how to prevent them from sudden failure. Recently-developed structural health monitoring technology provides a better solution for some challenges (ISIS Canada, 2001). In Hong Kong and Mainland China, structural health monitoring systems have been installed in more than 40 long span bridges.

The Tsing Ma Bridge in Hong Kong is a suspension bridge with a main span of 1,377 m. The bridge deck carries a dual three-lane highway on the upper level of the deck and two railway tracks and two carriageways on the lower level within the bridge deck. The Hong Kong Highways Department installed a comprehensive Wind And Structural Health Monitoring System (WASHMS) in the Tsing Ma Bridge in 1997 (Wong et al. 2001). This paper takes the Tsing Ma Bridge as an example to manifest how the WASHMS installed in the bridge is used for monitoring highway traffic condition, vehicle loading, and vehicle loading effects on the bridge. The configuration and the relevant sensors in the WASHMS of the Tsing Ma Bridge are briefly introduced. By using the traffic data collected by weigh-in-motion (WIM) stations of the WASHMS, the highway traffic condition and vehicle loading on the bridge are analysed in terms of vehicle traffic volume, vehicle traffic composition, axle load spectrum and gross vehicle weight spectrum. The measured axle load spectrum and gross vehicle weight spectrum are compared with the design spectrum. The strain data collected by 110 strain gauges installed at three sections of the bridge deck are analysed. A practical method in conjunction with the method proposed in BS5400: Part 10 is used to estimate the fatigue life of the bridge due to highway loading and railway loading.

The statistical data on highway traffic composition in 2006 revealed that a total of 8.5 million vehicles run through the Tsing Ma Bridge for the Airport bound way and 8.8 million vehicles for the Kowloon bound way. The percentage of heavy goods vehicles, including rigid heavy goods vehicles and articulated heavy goods vehicles, was about 5% to 6% of the total vehicles. Most of heavy goods vehicles used the slow lane for both the Airport and Kowloon bound ways. The statistical data on axle load distribution in 2006 manifested that only 12.5% of the total vehicle axles had an axle load more than 5 tonnes. The statistical data on GVW distribution in 2006 showed that only 7.5% of the total vehicles had an GVW more than 16 tonnes. The axle load spectra and GVW spectrum from the measured WIM data in 2006 and based on BS5400 both were on the safer side compared with the design spectrum.

The results of fatigue life evaluation demonstrated that the fatigue life estimated based on the daily data length was almost the same as that based on the monthly data length. The fatigue life estimated based on the connection class "F" was much longer than that based on the connection class "F2". The fatigue damage to the steel connections of the Tsing Ma Bridge was mainly induced by moving trains. The contribution of road vehicles to fatigue damage was rather small. The fatigue life of the bridge is very long for some connections, and all the instrumented components of the bridge are in good conditions.

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Latest practical developments in the bridge WIM technology

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ABSTRACT

The European Commission research project WAVE (2002) that ended almost 10 years ago, introduced some substantial improvements of the bridge WIM (B-WIM) technology. Since then developments continued and today the B-WIM systems are accepted in many countries around the world as an important player on the WIM market. They can be equipped with solar or fuel cell power units, traffic cameras and other sensors and are collecting data for various applications, from traffic studies, design and reconstruction of pavements, pre-selection of overloaded vehicles and bridge applications.

Bridge WIM systems have several advantages over the pavement systems. One is their full portability, which allows rotating of the same equipment around several locations. The free-of-axle detector (FAD) technology (Žnidarič, Lavrič, & Kalin, 2005) does not require any actions on the pavement during installation and life-time of the system. Results on some types of bridges, especially on shorter spans, are very accurate and, last but not least, they can in parallel measure the structural parameters that are essential for optimized bridge assessment.

In the last few years the B-WIM developments focused on one hand on new algorithms that would further improve accuracy of the results, and on the other hand on new or improved measuring techniques, that would allow them to be used on bridges that were traditionally less or not appropriate for B-WIM measurements. Some of these developments are discussed in the paper.

Generation of the experimental influence lines that are based on the measured strain responses of the instrumented structural elements has proven crucial for the high accuracy of B-WIM results, especially the axle loads. The influence lines used for calculations are obtained from a number of strain responses of random vehicles passing the bridge.

On bridges longer than 20 meters the traditional B-WIM algorithm is not able to properly divide the total measured loading between several vehicles and

Figure 1. Typical B-WIM installation.

their axles, if they are detected on the bridge at the same time. In the B-WIM practice this problem, known as multiple-presence of vehicles, is being mitigated by the *strips* method, which takes into account distribution of traffic loading from one lane to another. Applying the measured lane factors can be seen as an intermediate step towards implementation of a 2-dimensional influence surface, which is foreseen as the next major improvement of B-WIM algorithms but would, contrary to strips, require by far more comprehensive calibration procedure.

Recent studies also demonstrated how accuracy of results is heavily affected by the road roughness and by the type of calibration. Smooth (resurfaced) pavement and employing several calibration factors (WIM calibration by vehicle type) can considerably increase accuracy of measurements.

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Applications of B-WIM technology to bridge assessment

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ABSTRACT

Weigh-in-motion (WIM) data has been for many years used for bridge applications, primarily for calibration of bridge design and sometimes of bridge assessment codes. The data used was primarily obtained from the pavement WIM systems that did provide the necessary information about realistic traffic loading but could not give any additional information about the behavior of the structures. These can be obtained by a bridge WIM (B-WIM) system if it is installed on the bridge being assessed (Žnidarič, Lavrič, & Kalin, 2010).

Influence lines are the key factor for quality B-WIM measurements. They should be directly derived from the measured data on the site.

Load testing, especially the diagnostic one, efficiently optimizes structural models used for assessment. To avoid high costs of traditional load tests, the *soft load testing* was proposed. With the same objective to optimize the structural model, it uses a B-WIM and random traffic rather than some pre-weighed vehicles, to monitor the real structural behavior of the bridge under traffic loading. Due to the lower level of loading the results are more conservative than those from the traditional diagnostic load test, but can still result in large optimizations of results. At the same time the procedure is considerably cheaper than a traditional load test and does not require closing the traffic. To this stage it has been applied to shorter span bridges (up to 40 m), but this does include the majority, even over 90% of the entire bridge stock in most of the countries.

The B-WIM systems have also been used for measurements of static loading of traffic and dynamic amplification factors. The results of convolution, to obtain the expected maximum static loading effects, and the measured Dynamic amplification factors compared very well with the analytical results (ARCHES, 2009). For the DAF, the results of thousands of measured loading events demonstrate an inverse proportionality between vehicle weight and dynamic amplification and also a reduction of standard deviation with increasing weight. However, as results at this stage are still limited, the ARCHES project recommended using, with individual vehicles on the bridge, DAF equal to 1.15 to 1.20, depending on the smoothness of the pavement.

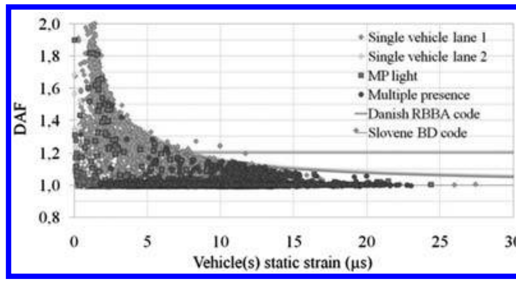

Figure 1. Typical relation between the maximum static strains of loading events and DAF.

In order to fully verify them, the new DAF measurement and evaluation procedures are being further developed.

The evident goal of the described methods, which include monitoring of the structural behavior (soft load testing) and traffic loading, is to optimize bridge assessment. If used, these results will prevent from spending the always limited financial resources for unnecessary expensive interventions on bridges, such a strengthening and replacements.

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A methodology to estimate risk related to road links, due to latent processes

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ABSTRACT

A methodology is presented that has potential to be used in infrastructure management systems to determine risk related to road links, due to latent processes. The methodology was developed taking into consideration the current infrastructure management systems and the availability of data in Switzerland. It is particularly relevant for Switzerland, but in general is applicable for all road infrastructures. The methodology was developed with the following conceptual view of a road link: A road link is composed of objects that are both clustered based on topographical characteristics and composed of object sections in the longitudinal direction. Objects are clustered hierarchically based on the topographical characteristics of their surroundings since natural hazards, in many cases, are governed by these topographical characteristics.

The risk related to a link is estimated by aggregating the risk of each item (object section, object or cluster) on the lowest hierarchical level to which they belong to give the risk of the item on the next highest hierarchical level, until the risk related to the link is estimated using appropriate modification factors. A graphical depiction of this aggregation for the flood hazard is given in Figure 1.

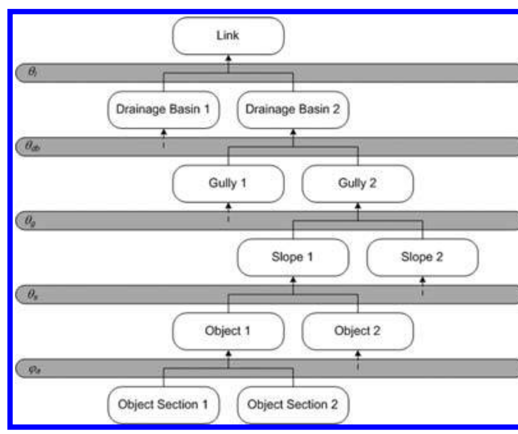

Figure 1. Graphical depiction of the aggregation of risk using modification factors.

The estimation of risk using the methodology requires the estimation of the probabilities of occurrence of hazard parameter intensities, the determination of the failure modes of the objects for each hazard parameter intensity, and the determination of intervention and other costs incurred for each failure mode. It is proposed that:

– the probability of occurrence of hazard parameter intensities be determined by conducting geographic coincident analyses between the investigated hazard and the infrastructure objects.
– the failure mode for each hazard parameter intensity be determined by conducting a failure assessment of the object section using predetermined guidelines, and
– continuous functions be developed from the discrete data to assess the consequences of failure for failure modes at each hazard parameter intensity.

The methodology proposed has potential to be used in Switzerland for the evaluation of risk related to road links with respect to one hazard that may result in multiple simultaneous failures on one link within one period. More research, however, is required to verify the assumptions made.

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A framework for comprehensive estimation of user costs for bridge management: A synopsis of existing practices and discussion of new considerations

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ABSTRACT

Bridge agencies strive to incorporate stakeholder perspectives in their decision-making processes. One way of doing this is to include user costs as a key performance criterion in investment evaluation. While agencies seek to manage their bridges to meet user expectations of performance, they are often stymied by lack of a comprehensive and consistent framework for assessing the several different user cost types and components. This paper tries to establish a framework for comprehensively estimating user costs for bridge management.

The paper first identifies the factors of bridge user cost incurrence and the various categories of user cost (Table 1). This is done on the basis of a synthesis of the existing state of practice and some new considerations.

For purposes of this presentation, the bridge user cost is categorized into detour cost, delay cost and accident cost. Also, it is recognized that the bridge life-cycle user cost consists of the cost incurred during the period of normal operations of the bridge and those incurred during the work zone period. The literature review showed that relatively few studies had addressed the latter category of user cost. The paper provides a refined and detailed procedure to calculate bridge detour cost and delay user cost. In addressing key considerations, the paper recognizes that for certain bridges, the user may detour for more than one reason, and not accounting for this situation may lead to double counting of user costs. This issue of double counting has not been explicitly considered in the literature. By using principle of set theory, the paper shows how the estimation of user costs of detouring could be carried out in a manner that avoids double counting. In addition, the paper improves the methodology for calculating bridge delay user cost by updating the speed-volume function.

The applicability of the framework is then demonstrated using a case study that calculated a broad array of bridge user cost categories for a steel bridge.

The results show that when the double counting issue is not taken into account, the calculated user cost is higher. Specific results for this case study suggest that the calculated workzone user cost (whether the double counting issue is addressed or otherwise) can be extremely high when the bridge is closed for rehabilitation and that the user cost of delay reduces drastically after the bridge widening intervention.

Summing up, the case study results suggest that it is prudent to appreciate the problem of double counting and to address this issue duly. The results also suggest that work zone user cost is a critical and dominant component of bridge user costs and needs to be considered in any bridge investment evaluation problem.

Table 1. User cost factors and possible user costs.

Traffic	Causes	Detour Cost	Delay Cost	Accident cost
On bridge	Load Capacity Limit	Yes		
	Vertical clearance limit over bridge	Yes		Yes
	Horiz. clearance limit over bridge	Yes		Yes
	Poor alignment	Yes		Yes
	Low capacity		Yes	
	Work zone	Yes	Yes	Yes
Under bridge	Vert. Clearance limit under bridge	Yes		Yes
	Horiz. clearance limit under bridge	Yes		Yes
	Work zone	Yes	Yes	Yes

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Bridge management technique: Implementation of a deterioration model to different highway networks

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ABSTRACT

Sineco performs from several years detailed inspections on bridge structures of some motorway networks in northern Italy, in order to perform maintenance programs as an integrated method between the inspection results database created by Sineco since 1995 and the Bms tools provided by the software Pontis. Such ask data analysis defines optimal preservation, policy identification and recommendation; bridge needs and performance measure forecasts and development of projects are included in agencies' economical planning.

Simultaneous use of the two systems consent, by one side to preserve the high level of detail of different data recorded during inspections, that are referred to any single defect, and to exploit also that historical data collected before the introduction of Pontis system, and by the other side, to develop cost and deterioration forecasting models necessary to achieve life cycle cost analysis and long term programming.

Deterioration forecasting models can be developed from different source of information. Sineco first step in defining forecasting models has been compiling expert elicitation data. This elicitation process is used to create deterioration models solely based on expert judgment. This allows reasonable models to be developed before having actual data, which take several years to be collected.

Expert transition probabilities are updated using historical inspection data collected by Sineco for several years, accumulating all possible pairs of successive registered condition state of the elements, subject to certain constraints (e.g., there should not have been a spontaneous improvement in the element condition between inspections).

It is generally possible to create deterioration models very customized and reliable for different bridge networks using the amount of historical inspection data. Nevertheless, in some particular cases, despite of a great amount of data available, it is not possible to draw up a deterioration model for a medium-long time forecast.

To overcome this problem, a theoretical method has been developed to draw up deterioration curves using available tests on concrete carbonatation on different elements of the bridges of Savona-Ventimiglia motorway (a forty years old motorway in which all the structures are in quite good conditions). In this case both the deterioration models determined by real inspection data and created by the experience or literature on similar motorways are unable to describe and simulate the real condition of analyzed bridges.

To calculate the carbonatation thickness also on those elements for which direct tests are not available, have been used diagrams that correlates carbonation thicknesses s and cubic concrete strength R_{ck}.

So it has been possible to determinate the whole service life of structural elements as the sum between the corrosion initiation time t_i (carbonatation time of concrete cover) and the propagation time t_p (ratio between limit penetration and corrosion speed). Then the service life of each structural element has been divided in the different condition states, and finally the transition probabilities to one condition state to the next one is calculated by means of the expression $P_j = 1 - 1/n$, where n is the time of staying in one condition state.

The resulting forecasting model is more conservative than the previous expert one, but it is useful to obtain significant results and to manage reliable cost-benefit analysis, and so to draw up a realistic and functional maintenance plan.

This method, that can be repeated for any other bridges group in which inspection data are not enough or not useful, provide a realistic deterioration model that allows to simulate future deterioration and obtain a priority list of maintenance works in a medium-long term scenario.

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Structural monitoring with wireless sensor networks: Lessons learned from field deployments

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ABSTRACT

In the last years, wireless sensor networks have emerged as a promising technology that is inducing a deep innovation in the field of structural monitoring. The main advantages of wireless sensor networks compared to conventional monitoring technologies are fast deployment, little interference and self-organization. However, since wireless sensor network nodes are battery powered, in long term monitoring applications the power management influences significantly the operation of a wireless sensor network. In data intensive applications, e.g. vibration based monitoring, low power hardware, duty cycling and efficient communication policies are not sufficient for achieving a sustainable system life-time. Since data communication is the most energy consuming task, a significant data reduction has to be attained in the nodes to achieve sustainable system life times. This data reduction is a challenging task, since it has to be performed with very limited computational and memory resources and in competition with the basic network functionality.

The objective of the paper is to present our experience over the past three years with data intensive structural monitoring using wireless sensor networks. Specific aspects of sensing, data quality, stability, availability, and system life-time are presented and analyzed. These results stem mainly from a still ongoing field test started in November 2006 on a cable stay bridge. The goal was to monitor the natural frequencies of stay-cables by using ambient vibration records (Figure 1).

These 3 years of field test experience demonstrate that operating a wireless networks reliably over a period of months or years was a non-trivial task. The problems relied basically on balancing the requirements of a data intensive application with the requirements of minimizing power consumption for a achieving a sufficiently long battery lifetime. The omnipresent goal to save power generated problems like sensor signal corruption by duty cycling or switch-on operations. Data reduction, a powerful method to save power, destabilized seriously the wireless sensor network by preventing the basic operations like time synchronization and routing.

Nevertheless, the progress made during these years demonstrates that data intensive structural monitoring with wireless sensor networks is feasible. The problems could be substantially solved with several hard- or software improvements. Despite the severe hard- and software limitations, the generated information complies with the quality requirements in civil engineering, which usually do not require high precision information. The hardware limitations, however, require a tight specialization to the monitoring task. This implies a detailed analysis and specification of the monitoring goals.

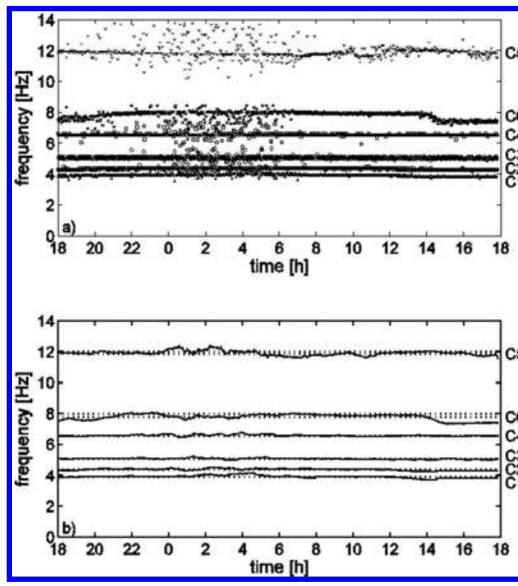

Figure 1. a) Natural frequencies of monitored stay-cables. b) Monitored natural frequencies smoothed with a moving average filter.

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An improved model for predicting NBI condition ratings

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ABSTRACT

The Federal Highway Administration (FHWA) National Bridge Inventory (NBI) Translator predicts the NBI condition ratings for deck, superstructure, substructure and culvert given the element-level inspection data. The American Association of State Highway and Transportation Officials (AASHTO) Pontis Bridge Management System (BMS) uses the NBI Translator as part of the system's program simulation to predict future NBI condition ratings and structurally deficient or functionally obsolete status. In using Pontis, the Hawaii Department of Transportation (HDOT) determined that the NBI Translator overestimates the poor ratings for NBI components and the number of structurally deficient bridges. The Translator logic accentuates the small quantities in the worst condition state, and this leads to an overprediction of bridges with poor NBI ratings.

This paper describes an approach developed by the authors for using the health index to translate element-level inspection data to NBI condition ratings. The approach was developed for HDOT to provide more accurate predictions of future bridge conditions and to support its bridge programming. The approach consists of devising a conversion table that translates the health index ranges into NBI ratings and applying the converted NBI ratings to Pontis simulation results.

The review of the HDOT inspection data showed that the average health index of each component group and the NBI ratings assigned by the inspectors were well correlated. This finding was also true when the same comparison was performed using data from Arkansas, Idaho, Oklahoma, and Wyoming. Based on this review, a conversion table was developed which is presented in Table 1. The table assigns values for NBI ratings of 4–8, which are the most prevalent in the HDOT data set. The health index ranges in Table 1 were used to assign simulation NBI ratings based on the predicted health index.

Table 1. NBI to health index conversion table.

	NBI Ratings				
NBI Item	4	5	6	7	8
Deck	<25	25–80	80–89	89–98	<98
Superstructure	<72	72–81	81–89	89–98	<98
Substructure	<72	72–81	81–89	89–98	<98

With the health index approach, the impact on the components' health index is minor when small quantities are predicted for the worst condition state, unlike the large impact predicted by the NBI Translator. This approach generates predicted NBI rating values that are well correlated with the inspector assigned NBI ratings and consistent with the health index. HDOT has implemented this approach for recalculation of the condition ratings generated by Pontis simulation. Future versions of Pontis should consider including the health index based prediction model.

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Bridge maintenance and practical bridge management systems in Japan

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1 INTRODUCTION

In Japan, maintenance of existing bridges is an important and urgent issue, because there are 150,000 bridges with more than 15 m span should be maintained in good conditions. Furthermore, Japan has severe environments surrounding bridge maintenance as follows: 1) Reduction of investment for infrastructures, 2) Increase of renewal investment, 3) Increase of maintenance cost, 4) Increase of stock of infrastructures.

In order to establish a rational maintenance program and provide a basis for accountability, several Bridge Management Systems (BMS) have been developed. In this paper, basic ideas and frameworks of bridge maintenance and representative five BMSs are introduced, which have been practically applied by local governments in Japan, such as Osaka prefectural government, Aomori prefectural government, Osaka municipal government, Nippon Expressway Company Limited (NEXCO), Honshu-Shikoku Expressway, and Hanshin Expressway. The characteristics are described and compared to each other, through which their differences are made clear.

2 BRIDGE MANAGEMENT SYSTEMS IN JAPAN

In recent years, many Bridge Management Systems (BMS) or Asset Management Systems (AMS) have been developed in Japan. Although the basic concept and procedure used in the systems do not significantly differ from each other, some details are different by reflecting the peculiar characteristics of bridge owners. Namely, damage states and damage causes are different so that the focusing or emphasizing points are not the same.

Expressway bridges require higher safety level than the ordinal road bridges. This means that the bridge owner of expressway bridges should perform more frequent inspection and repair to maintain the bridges in better conditions. Here two groups of bridge owners are selected; one is the local governments and the other is expressway corporations.

As local governments Osaka prefectural government, Aomori prefectural government and Osaka municipal government are selected, and three expressway corporations such as NEXCO, Hanshin Expressway and Honshu-Shikoku Expressway are considered. Osaka prefectural government, Aomori prefectural government and Osaka municipal governments are maintaining old bridges based on the concept of LCC. NEXCO built BMS in 2003 for the preventive bridge maintenance and the system has been applied to bridge management in NEXCO since then to keep the safety and security of the expressway. Honshu-Shikoku expressway has focused on the preventive maintenance so far, because they have to mainly manage long spanned bridges. Hanshin expressway has been paying attention to risk evaluation and financial analysis for road maintenance on urban expressway based on H-BMS (Hanshin Expressway Bridge Management System).

3 AOMORI'S BMS AND H-BMS

In this paper, as representative BMS for local government and expressway corporation, Aomori's BMS and H-BMS are introduced in detail. Aomori Prefectural Government has developed the Aomori Bridge Management System (ABMS) in 2004 and 2005, and began its bridge management in 2006 by using ABMS. H-BMS is a maintenance management system which has functions to calculate optimal repair policies by minimizing life cycle costs, to simulate future condition state and repair costs and to determine repair priority.

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Contribution of non-destructive defect detection to bridge management

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ABSTRACT

Regular inspection in structural and bridge engineering may result in doubts about the integrity of structures or critical structural details. In some cases, information is not available or not detailed enough for the planning of further maintenance decisions. Bridge owners and providers need additional information about the integrity of the structure or about the success of strengthening measures in order to assess the safe and reliable further service of the structure. Often, according to the possibilities of the structure's owner, only destructive testing (as e.g. coring) can provide the required information of the inner structure of reinforced concrete structures, about the current materials condition or about the quality of repair.

After the enhancing development of methods and data analysis within the last decades, Non-Destructive Testing is recommended in special inspection to investigate the inner structure. Images from evaluated data answer questions of interest and provide information on further details. The German Road Bridge Management System *SIB-Bauwerke* proposes to call in object-related damage analysis of a bridge structure (special inspection) in any case of doubts that are arising either from regular inspection, heavy deterioration or aging, from impact of accidents resp. to identify the execution quality of repair.

Latest steps in research and development for the application of acoustic non-destructive testing to reinforced concrete structures resulted in advanced sensors and scanning tools, allowing reliable data acquisition for the condition appraisal. The data processing as well as data fusion of two or more different measurements using different sensors or even different measurement techniques increases the accuracy of the obtained details.

The paper describes the opportunities for detection of near surface delamination between non-metallic materials using acoustic sensors. A dry coupling point contact transducers causes a mechanical wave in layers thinner than the wave length. The excitation is received by a second sensor. The signal is an information about the degree of the bond integrity between any non-magnetisable thin layered materials. The method was applied in laboratory and in field. Bond defects may be deteriorating and separating surface layers, insufficient repair quality or debonding of layered strengthening material as CFRP due to degradation or load effects. to physical characteristics, material-related properties and sensor reliability.

The lack of a reliable technique for the investigation of delamination between thin non-metallic layers resulted first in the confection of an appropriate sensor with piezo-ceramic point contact transducers working with low frequencies. The prototype was used during first applications.

Besides laboratory investigation, an extended field study with application of further non-destructive testing methods was carried out. Task of the study was i.e. bond control using ultrasonic method.

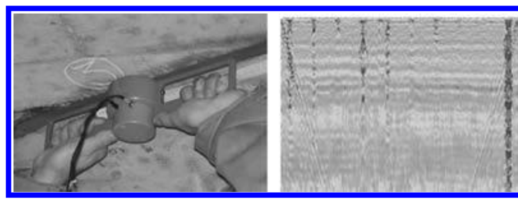

Figure 1. Sensor prototype and image with detected defects.

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Bridge inspection: Are we getting it right?

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ABSTRACT

For many bridge management authorities, the overarching inspection principles are to detect defects likely to cause unacceptable serviceability risks, safety to the public, structure or the environment. Inspections provide information used in the planning and management of bridges. However, there are still noticeable increases in costs associated with bridge maintenance. Catastrophic failures in service and increasing backlog to the management programs still make headline news even with improvements in training, certification and use of experienced and competent inspection staff.

In previous paper presentation to IABMAS06 & 08 I have advocated the issue of training and certification of bridge inspectors. Unfortunately, current practice still relies on prescriptive guidance such as inspection work being carried out by appropriately qualified, trained and experienced personnel. In the UK, for example, this is prescribed as, "a programme of continuing professional development (CPD) and training for bridge managers, engineers, inspectors and other staff should be provided to enable them to understand and implement the process described in the code"

This paper, argues that this prescriptive approach falls short of what is required. For example, it lacks emphasis on what skills or experience that the inspectors should be getting training in or experience. An evaluation by the author on the current practice of inspection and bridge management, found that many authorities still rely on methods where an inspector assesses defects on site, provide them a ranking and using a bridge management program a grading for each structure is made.

As in previous presentations, the author believes this approach will still lead to wasted resources and continued backlog to the maintenance programme. For example, an inspector may find a crack in a bridge. However, presence of a crack does not necessarily imply a significant risk. Further understanding of the type, characteristics, location, and effect of the crack on the structural and serviceability behaviour of the bridge is required.

This paper, in conclusion, proposes a radical approach to the training and certification of bridge inspector.

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Maximizing return on investment utilizing a bridge depreciation model

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ABSTRACT

The challenge facing bridge managers is how to maximize investment in their infrastructure given very limited budgets. There are few tools available to demonstrate that scarce monetary resources are effectively utilized.

An adaptation of basic accounting principles is employed to rationalize bridge expenditures. The model is that of a very basic business model that seeks to maximize value by obtaining the most yield from strategic investments.

The fundamental premise is that of maintaining the asset value of a bridge population at the highest possible level. Depreciation continually erodes the book value of a jurisdiction's structures. Rehabilitation and renewal of bridge components increases the measured asset value of the bridge fleet. The accounting and engineering challenge is to identify those capital investments that yield the highest return as measured by the increase in value of the bridge assets. The measure of yield is termed "the efficacy of investment."

The asset value of a bridge is determined in two steps; the undepreciated value and the depreciated value. The undepreciated value is obtained by breaking a bridge into its components and assigning a value to each component based on its geometric and material properties, and assigning a base new value on a unit rate basis. In this manner the asset value of a single bridge and a bridge cohort is assessed.

The depreciated value of a bridge is calculated by applying decay functions to the value of each bridge component. Each bridge component is given a deemed service life. The decay function depreciates the component value by comparing its age against its deemed service life. Both straight-line and parabolic decay functions are tested.

It is shown that for a normally age distributed bridge population and assuming straight-line depreciation, the depreciated value of all of the assets should be maintained at above 50% of the new value. Similarly for a parabolic decay function, the assets should retain 67% of their new value under ideal circumstances.

Capital investment in a bridge either renews a component or extends the life of a component. When a component is renewed, its depreciated value is reset to the new value. If a component is rehabilitated, the original service life of the component is extended.

The efficacy of investment is measured as the ratio of improvement of the depreciated value of the bridge to the amount of capital investment required.

There are significant overhead burdens that influence the capital cost of bridge rehabilitation or renewal. These overheads are real but do not contribute to improving the asset value of the bridge fleet. Hence the model gives credence to the notion of "Get in, get out, and stay out."

Calculating the efficacy of investment provides a rational basis for optimizing investment in bridges where all other considerations are relatively equal.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Trends in development of bridge management systems

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ABSTRACT

This paper focuses on the evolution of how latent processes are being considered, evolution of the modeling of deterioration, evolution of the optimization techniques being used to find the optimal intervention strategies in different bridge management systems.

The early generation of most BMSs started as a database to handle bridge inventory. The capability of BMS becomes modernized to help manage bridges over operational criteria satisfying safety and serviceability restrictions (Cho *et al.* 2007). As the roll of bridges becomes an essential component for the physical distribution and the transportation, the functional requirement of modern BMS has been changed also (Frangopol *et al.* 2001). In these days, the concept of the lifecycle cost (LCC) has become wide spread and the long term performance of bridges has been considered to assess the value of bridges as a civil infra- asset. Therefore, advanced BMSs have to have a decision making function to help bridge owners to obtain the most effective budget distribution and corresponding optimum management scenario (Kong and Frangopol 2003).

The reliability and usability of the bridge management system significantly depends on the fundamental data such as costs of inspection, maintenance, and failure; condition profiles based on inspection data; and reliability profiles considering the deterioration model. In this paper, trend of BMSs has been discussed based on some basic features of BMS such as Direction of BMS, Bridge Inventory and Inspection, Inspection Information, Condition and Safety Measure, Deterioration Prediction Model, Life-Cycle Performance and Cost Evaluation, Optimum maintenance scenario, Optimization in system level. Except these there are more issues to be considered for developing modern BMS such as network level optimization (Liu and Frangopol, 2004).

For the characteristics of inspection measure, it can be said that the current trend in inspection measure is based on subjective condition even though some safety related measures are added to cover the weakness of quantitative but subjective condition based on visual inspection.

To construct a bridge management system with better performance, an effort needs to be made to obtain reliable data even though this would be a time consuming procedure. Also, the bridge management system always needs to be used with inspection results and suitable engineering judgments to avoid making an inappropriate decision because of uncertainties included in the cost, condition, and reliability models.

The optimal solution depends on the condition and safety measures in addition to life-cycle cost of management scenarios. However, currently, each BMS system developed by independent authorities uses different methods resulting in loss of opportunity of communication for saving funds and man powers. Therefore, it might be worth of trying to prepare international fundamental standard and guideline of bridge management by an international society.

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The bridge management system of the NYCDOT

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ABSTRACT

The New York City Bridge Inspection/Research and Development Unit (BIR&D) is tasked with inspections and management of approximately 800 of the more than 2100 bridges in New York City. The BIR&D unit is continually striving to improve inspection methods in order to meet their own, internal requirements as well as those imposed by the NY State Bridge Data Management System, and the National Bridge Inspection Standards of the Federal Highway Administration.

In 2003 members of the BIR&D unit attended a meeting where Advitam was demonstrating the functionality of its ScanPrint® Inspection Software. Shortly thereafter an inquiry was made as to whether or not the software could be customized to fit the needs of the Bridge Inspection Unit. The application development process began in October 2004 with two Joint Application Design (JAD) sessions to identify shortcomings in existing systems and desired features of the new software.

As a result of the JAD sessions and other interviews with the user community regarding past experiences with NYCDOT applications, a functional requirements document was created to list the specific tasks expected of the BDS system.

The BDS is an integrated solution consisting of 3 main components:

1. An extensive inventory which can be accessed by a standard application or via a web based portal,
2. A Windows based application to manage the inventory, the inspection process and perform software administration,
3. An offline system designed to work with disconnected laptops and PDAs during the field inspections.

Using these 3 components, BDS is able to perform all of the daily tasks typically associated with bridge management.

Beta testing of version 1.0 of the application commenced in July 2006 with a small group of Key Users within the Inspection Unit. Full deployment to all of the inspection staff was initiated at the end of July 2006. Since its initiation the unit has completed 65 biennial inspections, 94 emergency inspections and 490 monitoring inspections. BDS version 1.6, the present version installed at the DOT, addresses 55 of the 57 functional requirements identified during the initial stages of the application development, and in addition addresses 15 more that were identified during the project. A key feature added during the project was BDS-Web, an interface that allows access to bridge information and inspection data without having the application installed on your PC.

Through the customization of an existing bridge inspection and management tool, the NYCDOT has been able to develop a comprehensive Bridge Data System suited specifically to their needs. The utilization of current information and data warehousing technologies provides an excellent platform for the development of future upgrades to address advancements in Asset Management Techniques and Information Technology. Most importantly, the BDS will facilitate the inspection processes and provide valuable decision support information to assist the Department in managing its inventory of critical and aging infrastructure.

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Integrated bridge inspection and management software for New Jersey Turnpike Authority

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ABSTRACT

The New Jersey Turnpike Authority is one of the United States' largest toll-road systems and operates both the New Jersey Turnpike and the Garden State Parkway. In the past these two major roadways operated as separate entities and had developed different bridge inspection programs for the nearly 1,000 combined structures (over 20' in length). The Authority's bridge inspection program is designed to meet the needs of a variety of critical stakeholders. The first and foremost priority is ensuring the safety of the users of the roadway. The inspection data is also used internally by the Maintenance Department to plan repairs and by the Engineering Department for capital improvements and overall management. Externally, information is provided to NJDOT and FHWA.

A number of different consultants are engaged by the Authority to perform bridge inspections each year. Organizing all of this information into paper and computer formats for the various stakeholders has proven to be a difficult challenge. The New Jersey Turnpike Authority has recently completed a program to standardize the format of the reports and information for all of its structures (bridges, culverts, ancillary). This is being accomplished by implementing a computerized inspection and management program that is capable of handling the wide variety of structure types, various consultants, and output needs. This paper will examine a brief history of the Turnpike and Parkway and their respective bridge inventories (over 1000 bridges) and inspection programs. It will present the challenges faced as well as the software solution that is being delivered to solve these issues and provide a greatly enhanced inspection and management process.

Figure 1. The New Jersey Turnpike varies from two 2-lane roadways to six 3-lane roadways by Newark Airport, with dual-dual roadways over a 32 mile length through the busiest zone.

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Integration of bridge management systems (BMS) and pavement management systems (PMS)

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ABSTRACT

Transportation agencies are interested in finding ways to better manage pavement and bridge assets together using an intelligent and integrated asset management approach. More than simply storing asset information and condition in management systems without providing analytical capabilities, or simply reporting information from separate management systems, the challenge is to find ways to better use inspection and testing information, recommended work, prioritization and timing of the integrated work program.

The problem of an integrated analysis is complex and few solutions exist. One of the earliest integrated MR&R management systems was the Kuwait Infrastructure Maintenance Management System (Al-Kulaib et al (1997)). Developed for the Kuwait Ministry of Public Works between 1997–99, the system allows the Ministry to manage their capital and maintenance planning functions more cost effectively for roadway, bridge, storm and sanitary sewer networks, as well as right-of-way features such as traffic signs, pavement markings, traffic signals and sidewalks.

Technology has evolved in BMS, PMS, and information technology to the extent that many new tools are available. Computing speeds, network and internet access speeds, and storage availability has made certain processes viable that were previously marginal. Yet there has been little progress in the development of integration tools for these two major transportation assets and very little progress in integrated prioritization. In 2007, Stantec embarked on a research project to develop a concept design for the integration of inventory, condition, and analysis results from bridge management and pavement management systems. This included a new integration tool and new analysis integration optimization tool. The resulting tool is a web-based application with GIS interface which can display all information and analysis results for integrated assets.

This paper presents the findings of this research and development project and a description of the prototype tool which integrates BMS and PMS in a single interface. The tool was developed to use Stantec's Pavement Management System (HPMA) and Bridge Management Systems (Thompson et al (2003), Ellis et al (2008)), but can be used for any bridge management system and any pavement management system.

The Phase 1 prototype includes the ability to display a variety of inventory and condition data on several types of Microsoft Vitual Earth maps. In Phase 2 (currently underway), an analysis module is being developed based on multi-objective optimization methodology of the type described in NCHRP Report 590 (Patidar et al (2007)). The final product will also be able to display integrated results on the mapping interface as well in tabular and report form.

It is hoped that this research will further advance concepts of integrated asset management, and benefit agencies who are already implementing integrated asset management solutions, or who are considering similar projects.

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Development of a bridge network life cycle cost model

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ABSTRACT

As bridges age and volumes of traffic continue to increase unabatedly, bridge maintenance has become a considerable challenge for governments around the world who are managing ageing infrastructure. A Bridge Management System (BMS) is defined as a rational and systematic approach to organising and carrying out all the activities related to managing a network of bridges. Two of the main objectives of a BMS are the prediction of the future state of structures, and the prioritisation of maintenance and rehabilitation. It is used to optimise the inspection, maintenance and rehabilitation of a network of structures with respect to minimising the total cost, and achieve the best performance from the network of structures.

Ireland has over 20,000 bridges on its road network with over 2,000 of these on national roads. This stock comprises a range of structural forms, located in different environments and with varying ages. In 2001, the EIRSPAN Bridge Management System was introduced to coordinate and integrate activities such as bridge inspection, repairs and rehabilitation work in order to ensure optimal management of this national road structure stock. The system includes the essential components of a BMS, considering the inter-related activities of inventory gathering, principal inspections, routine maintenance and rehabilitation works required for bridges. To date however, the system has been primarily used as an inventory database and prioritisation decisions on maintenance and repair have been taken on the basis of visual defects highlighted in the Principal Inspection report using a simple ranking system which identifies the bridges with the worst condition rating carrying the highest traffic volumes. As such, whole-life costing has been available on a bridge specific level but not as yet on a network level. To this end, in 2009 the National Roads Authority of Ireland (NRA) launched a research project with the primary objective of developing a bridge network lifecycle cost model whereby "The model would be developed in conjunction with the NRA's EIRSPAN Bridge Management System".

Consequently the developed procedures have a Markovian Basis and will achieve the specific objectives of (i) incorporating deterioration parameters from the full range of materials including, Reinforced Concrete, steel and masonry structures, (ii) considering bridges of all ages and exposure conditions as well as other parameters specific to Irish conditions, (iii) identifying budget restrictions going forward as well as highlighting the future financial impact of investment restrictions (i.e. % increase in structures of a particular material, age etc. deteriorating to lower condition), (iv) highlighting the future financial impact of various investment options (i.e. % increase in structures of particular material, age etc. deteriorating to lower condition rating as a consequence of alternative investment options for repair) and (v) ultimately providing a consistent basis for improving the way in which Irelands bridge network is managed.

The principal tasks to be completed in achieving these objectives have been identified as (1) the development of Markov transition matrices for different materials at global structural level, (ii) development of non-stationary transition matrices to model correlation between components within structures, (iii) incorporation of assessment and repair performance uncertainties, (iv) allocating maintenance costs to the condition ratings assigned within the EIRSPAN system and (v) Bayesian updating of condition ratings based upon assessments logged within the EIRSPAN system.

The development of this model then is the basis of this paper. It is calibrated specifically for the Irish network in conjunction with the information provided in the EIRSPAN database.

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Implementation of a multi-period bridge investment optimization approach utilizing Pontis results and additional constraints in three agencies

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ABSTRACT

One use for a bridge management system (BMS) is to optimize allocation of funds and predict future conditions given a set of funding constraints. The Virginia Department of Transportation (VDOT), Oklahoma Department of Transportation (ODOT) and Hawaii Department of Transportation (HDOT) all use the American Association of State Highway and Transportation Officials (AASHTO) Pontis BMS for this purpose. In recent years staff in each of these agencies have invested considerable time refining the models in Pontis to improve their ability to obtain realistic results from the system. A critical limitation they have encountered in using Pontis is that the system offers no functionality for specifying the budget by work type when performing a program simulation. Instead, when one runs a program simulation in Pontis, one specifies an overall budget constraint, and the system attempts to optimize the work recommended in any one year without regard to work type. The lack of functionality for specifying budget constraints by type of work has been perceived by some to be a significant shortcoming in the system that limits the value of the system's results.

This paper describes an approach developed by the authors for using results generated by Pontis to optimize bridge investments over a ten-year period considering budget constraints by year for each of five work types. The approach was originally developed for VDOT to support VDOT's approach to bridge programming, which involves determining the allocation of funds by district with funding constraints by work type. The resulting allocation is used to determine what bridges on which to perform work, given work on a bridge typically is performed no more than once every ten years. Formulated as an optimization problem, the objective of the problem is to select the set of alternatives that maximizes benefit subject to a set of budget constraints specified by funding period and work type. Each alternative has assigned to it a benefit or utility and set of costs by work type and period. Conceptually, the alternative could be a single project or life cycle profile that includes a set of actions over time. It is assumed that one and only one alternative can be selected per bridge for the analysis period.

Solving the optimization entails performing a series of Pontis simulation runs as a one-time or occasional step, and then performing a secondary optimization with work type constraints. The preliminary Pontis simulations are used to specify alternative life cycle profiles for each bridge in a network, including a "do nothing" for each bridge. Once the simulations are completed, statistics are compiled for each bridge, including the costs, benefits and predicted performance for each candidate life cycle profile (also referred to as alternatives or project alternatives in this paper). The optimization step selects the set of life-cycle profiles that maximizes agency and user benefits, subject to budget constraints for each year and work type. Two systems have been developed to support the approach described here: the VDOT Pontis Robot for automating Pontis scenario runs, and the VDOT Post-Pontis Optimizer for performing the optimization. Following initial implementation of the modeling approach for VDOT described previously, subsequent changes detailed in the paper were made based on testing performed for ODOT and HDOT.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

BrIM based bridge operations and management

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ABSTRACT

The design, construction, subsequent service life operations and management phases in a bridge life cycle have become increasingly dependent upon information management. Information to be managed is of various types: engineering analysis and design, methods/techniques of construction, contracting methods used for project delivery, day-to-day operational practices such as inspections, load rating, permitting and routing, project and network level planning and programming to meet financial, personnel, equipment, and contractual needs. "Bridge Information Modeling" (acronym "BrIM"), first introduced in 2005 by co-authors Shirole' and Chen, facilitates an integrated approach to managing all phases of bridge life. This paper describes and illustrates just such an approach to demonstrate how recently developed software linkages to existing commercial software utilize a BrIM "Data Pool" to accomplish a variety of operational activities, such as load rating, routing and permitting. It also demonstrates the leveraging of this Data Pool for project and network level bridge inspection, reporting, and planning and programming of maintenance, repair, rehabilitation and replacement activities.

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Updating bridge deterioration models with irregular inspection intervals

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ABSTRACT

Many transportation agencies are using an element level condition state inspection methodology to inspect their bridges. The Provinces of Ontario, Nova Scotia, Prince Edward Island, Manitoba, and Saskatchewan, and many municipalities, have adopted the Ontario Structure Inspection Manual (OSIM), while Québec has implemented a similar approach. The methodology requires the inspector to quantitatively record the severity and extent of defects, appropriate to the material and the specific component of the bridge. This is as opposed to methods in which the inspector assigns a numeric rating (e.g. 0 to 9) to represent the status of an entire element or component of the structure and thus masks the quantities and types of defects occurring in the element.

Like many element-based bridge management systems, Québec's Strategic Planning Module (MPS) uses Markovian deterioration models to forecast future element condition state distributions. A benefit of these models is the ability to update them from historical condition data without a long time series of inspections.

The 5-year period of the Canadian deterioration model is unusual in bridge management systems but has some distinct advantages. In a 1-year model, there is a small but non-zero probability of each transition each year. With four condition states, there is a small probability of the worst condition state after just four years, something that would be considered unrealistic in practice. Because of the small fraction that quickly arrives in the worst state, bridge management systems have to take steps to suppress small needs or to postpone the more advanced transitions. This adds complexity to the analysis. With the longer transition periods used in the Canadian systems (Thompson et al. 2003, Ellis et al. 2008), these extra steps have not been necessary, making the model outputs simpler and easier to understand, and especially suitable for smaller agencies such as municipalities wanting a less elaborate solution.

The analysis module, referred to as the Strategic Planning Module or MPS includes a module to perform the regression analysis and Bayesian updating process to generate new deterioration models. An innovation in the procedure is that it takes advantage of irregular inspection intervals from one to five years, rather than being tied to a fixed cycle length. The method takes advantage of the structure of Markovian transition probability matrices to arrive at a closed-form solution that is readily implemented in software. This improves the utilization rate of inspection records.

The model was demonstrated in a user-friendly Excel spreadsheet, which has been useful for training purposes. It was also built into the MPS software for routine maintenance of the Québec bridge deterioration models over time.

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Estimation of enhanced Pontis deterioration models in Florida

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ABSTRACT

The Florida Department of Transportation (FDOT) has conducted a series of projects to enhance its implementation of the Pontis Bridge Management System, which is developed by the American Association of State Highway and Transportation Officials (AASHTO). These enhancements have included a Project Level Analysis Tool to present graphically the scoping and timing alternatives on a bridge; and a network analysis tool to analyze funding vs performance tradeoffs. These tools have proved to be a valuable platform for implementing new improved planning models that can't be built into Pontis itself, but operate on Pontis data.

Taking advantage of 14 years (1995–2009) of history with 884,678 individual element inspection records, the agency has amassed sufficient data to develop statistically sound deterioration models for its entire bridge inventory, including specialized elements for non-bridge structures, such as sign structures and retaining walls, and moveable bridge equipment.

A problem noted in previous research is that the Markovian models used in Pontis have fairly rapid initial deterioration. This creates a serious problem for multi-year programming models, because it is difficult to configure such models to maintain a realistically high network condition level.

In order to address this problem, Florida in 2009 developed an enhanced form of the Markovian analysis that features a Weibull survival probability model for the initial transition from the best to second-best condition states. This model has a shaping parameter that can describe the slower deterioration rates typical of new elements. This same model form is planned for inclusion in the next major release of Pontis in 2012.

Figure 1 illustrates some of the results of the research. It was found that decks and expansion joints do behave in a manner consistent with the Markovian model, without a statistically-verifiable age dependency. However, superstructure and substructure elements evidenced delayed onset of deterioration that could be described by a Weibull model.

Further research is underway to improve the screening of data points for maintenance activity, and to apply the new models in Florida's decision support tools to see how they affect the outputs.

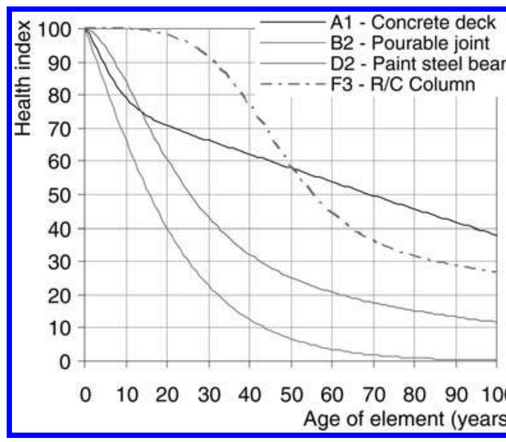

Figure 1. Comparison of selected element types.

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Advanced dynamic testing and bridge management system

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ABSTRACT

Transportation network of each developed country is one of the most exploited and lively systems. Bridges are the most critical points in this network and due to their peculiar structure and extensive use they are exposed to accelerated deterioration. Results of dynamic bridge tests offer information important for the management process covering areas such as damage detection, data for fatigue analysis and for life-cycle forecasting as well as for long-term bridge structure health monitoring.

The purpose of this paper is an overview of advanced dynamic tests of bridges as well as discussion of their applications in the Bridge Management Systems. Main attention is paid to various techniques of structure excitation: tests of vibration excited by normal traffic, special vehicles, vibration exciters or free vibration tests as well as to advanced testing equipment and efficient data processing procedures.

Data of bridge structures required for use in Bridge Management Systems (BMS) can be collected during bridge dynamic tests which are focused on two areas: parameters of the vibrating bridge structure itself (modal tests) and vibration parameters of the structure subjected to a specific load (operational tests). The two philosophies of testing are different in terms of applied methods of vibration excitation, data processing techniques and field of results applications in advanced BMS. General classification of dynamic tests of bridges, taking into account excitation methods, analyzed parameters and range of application in BMS, is presented in the paper.

Dynamic parameters of the bridge structure can be identified on the basis of a single test as well as can be collected either by permanently installed measuring systems or by systematically performed tests. Conception of the bridge condition monitoring system based on structure modal parameters changes, proposed by Zwolski (2007), is presented and discussed in the paper.

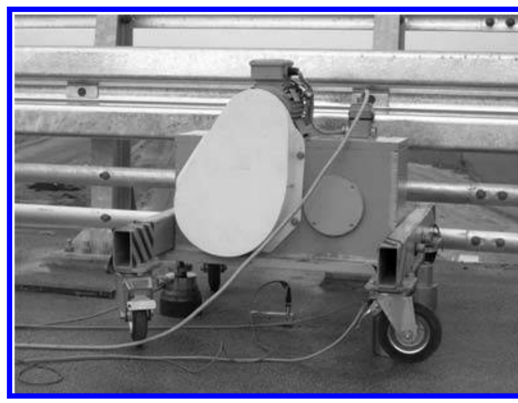

Figure 1. Rotational eccentric mass exciter applied in bridge tests (Zwolski 2007).

Considered technologies of the bridge dynamic tests are illustrated by example of practical application for identification of dynamic parameters of the structure and damage detection. Presented tests were performed on a steel railway bridge with damage caused by dismantling of riveted connections between the girder stiffeners. The structure was modally tested by means of the rotational eccentric mass exciter (Bień et al. 2004) presented in Figure 1. For damage detection, based on analysis of dynamic data, the tool called UNCOMAC was applied (Zwolski 2007).

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MS8: Uncertainty in bridge damageability modelling
Organizer: M. Shinozuka

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Failure estimation of highway bridges under combined effects of scouring and earthquake

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ABSTRACT

Bridge scour is one of the most common causes of bridge failure in the United States (HEC-18 2001). In order to prevent the failure of bridges spanning waterways, it is essential to consider the effects of scour on the structural performance of bridge and to design the bridge so that it can resist any structural degradation due to scour in addition to other natural hazards. According to AASHTO LRFD (2007), scour is not considered as a load parameter, but its effects on changing the structural properties of bridge should be evaluated to make sure that the structure is not vulnerable to extreme loads, such as wind and earthquake. Towards this goal, the current paper studies the behavior of reinforced concrete (RC) highway bridges under the combined effects of scouring and earthquake.

Scour is defined as the water-induced erosion of soil around the foundation of bridge and can be identified mainly in three forms: 1) long-term aggradation and degradation of the river bed due to erosion and deposition of material, 2) general scour which may result from contraction of the flow, and 3) local scour caused by an acceleration of water flow at the piers or abutments (HEC-18 2001). In the latter form, the presence of obstruction to the flow increases the speed of stream and creates vortices along the horizontal and vertical axes which result in higher scouring depths. Among the mentioned three forms of scour, it has been proved that the local scour is generally the most significant form. Hence, this paper focuses on the local scour and studies the uncertainties associated with its modeling around the bridge pier. Then, a set of two- and three-span RC bridges are selected to assess the extent of structural degradation as a function of scour depth. The seismic performance of case study bridges are evaluated at the next step using fragility parameters and effects of local scouring on the response of RC bridges are studied during nonlinear time-history analysis. This provides a multi-hazard framework which can be used predict the probability of occurrence of various seismic damage states for a bridge under scouring effects.

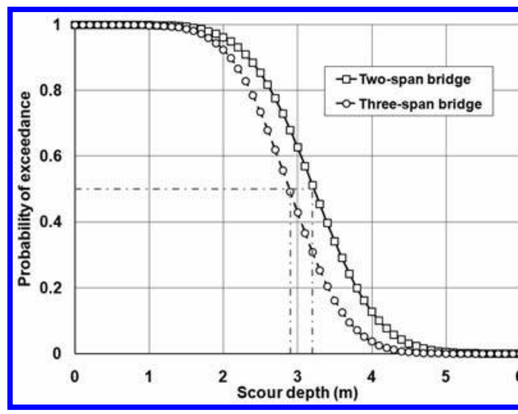

Figure 1. Developed risk curves for the two- and three-span bridges under different scour depths.

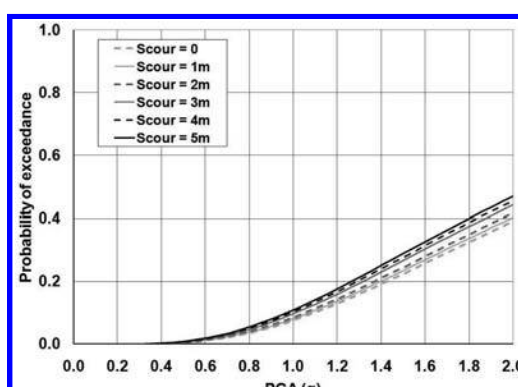

Figure 2. Developed fragility curves for the three-span bridge under different scour depths (moderate damage).

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Deterministic and probabilistic evaluation of time to corrosion initiation for bridges located in coastal areas

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ABSTRACT

A Chloride-induced corrosion in reinforced concrete (RC) bridges is a mechanism caused by the intrusion of chloride ions into concrete. This mode of corrosion is more probable when RC bridges are located in coastal regions and exposed to aggressive environmental conditions. Because of the penetration of chloride ions in structural members, the chloride content of concrete gradually increases and when the concentration of chloride ions in the pore solution on the vicinity of reinforcing bar reaches a threshold value, the chloride-induced corrosion initiates.

Chloride transport mechanism in concrete is a complex phenomenon that may occur in several forms, such as ionic diffusion, capillary suction, and permeation. The rate of this mechanism depends on the characteristics of concrete, degree of pore saturation, chloride binding capacity, free chloride content, and exposure conditions. By increasing the duration of time through which a bridge is exposed to aggressive conditions, the deterioration process of reinforcing bars can get relatively fast. This results in cracking or spalling of RC members and may lead to severe reduction in serviceability of bridges.

The time between corrosion initiation and serviceability failure is usually smaller than the required time for corrosion to initiate. Therefore, the realistic estimation of corrosion initiation time has a significant role in the accurate performance prediction of RC bridges over the time. In the present paper, an integrated computational framework is proposed to simulate the penetration of chloride ions into concrete. Towards this goal, the effects of various parameters, such as water to cement ratio, ambient temperature, relative humidity, age of concrete, free chloride content, and chloride

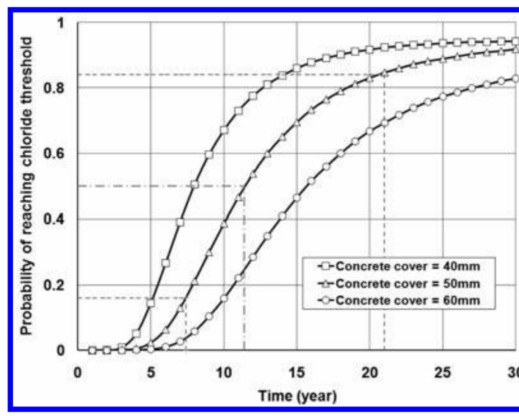

Figure 1. Cumulative distribution function of corrosion initiation time for three concrete depths.

binding capacity, are considered to calculate the chloride content in different time intervals. This leads to a deterministic approach for the estimation of corrosion initiation.

The developed computational framework is then improved by taking into account the effects of uncertainties of influential parameters in the deterioration process. Through a probabilistic approach, the probability distribution function of corrosion initiation time is determined and the results are compared with the estimated initiation time obtained from the deterministic approach. Finally, the structural performance of a set of short-, medium-, and long-span bridges are evaluated using pushover analysis over a life cycle of 30 years and the effects of variation in corrosion initiation time on the prediction of remained structural capacity are studied.

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Uncertainty in bridge fragility curves and its effect on the seismic risk evaluation of a highway transportation network

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ABSTRACT

Seismic vulnerability of bridges, commonly expressed in the form of fragility curve, is a key component of seismic performance evaluation of a highway transportation network. Two-parameter lognormal distribution functions are generally used to represent fragility curves. The fragility parameters are estimated using maximum likelihood method. However, these parameters may have some statistical variation, which in turn introduces uncertainties in the fragility curves. Such uncertainties may play a critical role when the bridge seismic vulnerability model (in terms of fragility curves) is used for the performance evaluation of highway networks under regional seismic hazards. The present study evaluates the statistical uncertainty associated with such a bridge seismic vulnerability model. Monte Carlo simulations are performed to develop 90% confidence intervals (within 5% and 95% confidence levels) of bridge fragility curves. These confidence intervals are taken as a measure of the uncertainty involved in the bridge vulnerability model. This vulnerability model is used then in the seismic performance evaluation of a regional highway transportation network. The network is analyzed under a set of USGS hazard consistent scenario earthquakes. Monte Carlo simulations are performed to predict the degraded performance of the network when all constituent bridges are damaged under the scenario earthquakes considered herein. The expected loss due to network degradation is evaluated in terms of total social cost. Seismic risk curves are developed that represent the annual exceedance probabilities of various levels of expected loss in the network under future seismic events. Results indicate that the developed risk curves have variation due to the use of bridge fragility curves with different confidence levels (e.g. 5%, 50% and 95%). Hence, the quantification of uncertainty involved in the bridge fragility curves is extremely important for reliable evaluation of seismic risk of highway networks.

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Sequential bridge fragility updating using long-term monitoring data

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ABSTRACT

The present study evaluates the performance of a real-life bridge under a set of earthquake ground motions. A three-span continuous cast-in-place pre-stressed post-tensioned box-girder bridge (known as "Jamboree Road Overcrossing (JRO)"), located in Orange County, CA, is considered for this purpose. The bridge is instrumented by California Department of Transportation (Caltrans) with 13 servo-type accelerometer. These sensors are recording bridge vibration response under ambient and traffic loading conditions over past seven years. Additionally, the bridge response under several earthquakes was also recorded.

In this study, the natural frequencies of the Jamboree Bridge are calculated from the database containing recorded bridge response. Following the flow-chart in figure 1 a finite element (FE) model of the bridge is developed using the computer code "OpenSees" and including non-linear characteristic like plastic hinge, gap element, bearing, piles and soil modeling. The model is initially built using the design data and successively updated using the long-term monitoring data from the installed sensor. A dataset of Inelastic Time History Analysis is performed after each model updating and the damageability of the bridge, in term of fragility curves is updated. The comparison of the damageability of the bridge over the time is studied, in figure 2 the comparison of fragility curves obtained from the initial model built from the design data and the updated model using field data is made. The outcome of this research provides useful insights for analyzing RC bridges under seismic ground motions.

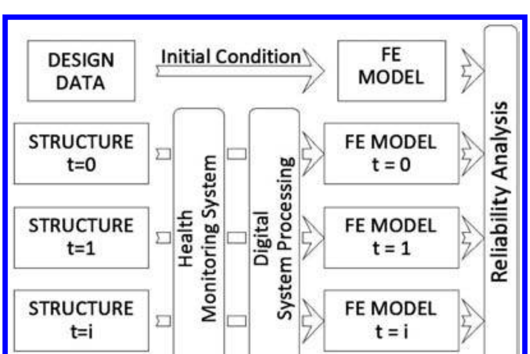

Figure 1. Flow chart of the study.

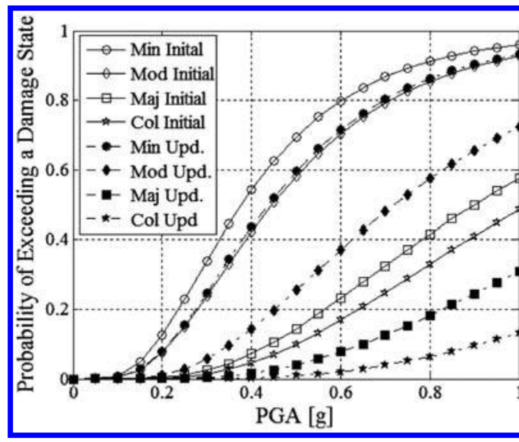

Figure 2. Fragility curves using model updating.

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MS9: Performance-based asset & risk management of the highway infrastructure system

Organizer: A.E. Aktan

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Mitigating infrastructure performance failures through risk-based asset management

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ABSTRACT

There is a pressing need for a more conceptual and generic definition of risk in the context of infrastructure performance. The objective of this paper is to propose such an expanded definition of risk appropriate for underpinning infrastructure decisions. This paper presents a brief history of risk and discusses the current limitations of convention risk definitions related to infrastructure decision-making. Of particular interest is the need to include both psychological and cultural weighing factors within the risk formulation. This allows both the public perception related to the objectional nature of the specific risk and the cultural expectations for infrastructures to be explicitly defined and considered. The paper concludes with a description of an eight step procedure for constructing an integrated asset management system, based on the proposed risk definition, to effectively underpin infrastructure decision-making.

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"One Team" bridge maintenance strategy – MACs in the UK

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ABSTRACT

The UK's strategic road network is managed through a series of innovative contracts called MAC (Managed Agent Contractor) contracts whereby a fully integrated, team consisting of a maintenance contractor and consultant, effectively take on the management of large sections of highway including bridge structures. The team plans, manages and executes routine, preventative and rehabilitation works (to a defined value limit). Driven by government policy and independent reports recommending an industry "re-think", the MAC contracts have been developed over a long period of time and have resulted in interesting developments in the bridge maintenance strategy adopted for the network.

With overarching strategic goals (safe, reliable journeys, informed travelers) and aligned incentives, the out-come-based, performance contracts and specifications have allowed the private sector to innovate and work with the UK's Highways Agency to develop a bridge management and maintenance process that is considered cost-effective and sustainable. The process includes a risk-based approach that guides the maintenance activities conducted and pro-actively encourages a culture of continuous improvement.

All aspects of service delivery are based on the Highway Agency's strategic goals – safe, reliable journeys with in-formed travelers. The success of the MAC has required an intelligent and pro-active client. Success has required partnership between the client and the contractors who take ownership and pride in their Network Areas with an optimal allocation of risk and focus on value for money.

Future contracts will need to demonstrate and actively manage sustainability issues. Continued efficiencies are required to eliminate waste particularly in construction, network operations, design and implementation of processes.

The current proposed US Transportation Bill is expected to create a greater focus on performance and to include funding mechanism changes. The current US State budget shortages an increasing need to improve the management of existing infrastructure are strong drivers for change that may provide an opportunity to re-think bridge maintenance contracting. An option is to bundle existing contracts and create longer term performance based contracts connecting bridge management policy with maintenance activity to gain the efficiencies and innovations necessary for the future. The MAC contract offers a proven model that can be adapted to the US market.

A private sector's view of the bridge maintenance strategies adopted, the reasons for success and the issues arising is also provided. The paper illustrates how bridge maintenance activities are driven by policy and the effects of strategic goal alignment and risk-based approaches. It also draws parallels with current bridge maintenance outsourcing activities and maintenance strategies in the US.

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TEAMS: Florida's Turnpike Enterprise's answer to asset management

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ABSTRACT

Florida's Turnpike Enterprise has an application called the Turnpike Enterprise Asset Management System, or TEAMS as it is commonly referred. The mission of TEAMS is to provide an improved approach to management and preservation of infrastructure assets, to protect bondholder investment, and sustain system performance.

TEAMS was developed as a web-based computerized means to catalogue Florida's Turnpike Enterprise's assets, evaluate current conditions, predict time to renewal and replacement, and develop appropriate periodic maintenance budgets. The system resides in centralized servers and users access it via the Florida's Turnpike Enterprise Intranet using a graphical web browser interface. The system is comprehensive, customizable, user friendly and compatible with other legacy systems currently in place. The asset management system gives Florida's Turnpike Enterprise a highly efficient means of identifying, tracking, and maintaining the performance history of right-of-way assets, generates lists of detailed periodic maintenance needs, and develops renewal and replacement budget forecasts at the user's desktop.

This paper focuses on how asset management principles have been implemented at Florida's Turnpike Enterprise in the form of the TEAMS application. Discussion will start with the history of Florida's Turnpike Enterprise, lead to the early stages of asset management at the agency, and then to the development of the asset management system. Afterwards the current state of TEAMS and an analysis of its future at Florida's Turnpike Enterprise will be discussed.

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Optimal resource allocation for seismic retrofitting of bridges in transportation networks

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ABSTRACT

In this study bridges' fragility is evaluated considering the effect caused to the transportation network and to the economy of the region affected by the earthquake. This is a multi-disciplinary approach since the procedure is composed by several modules: geological, structural, transportation, economic. The procedure is based on a stock of bridges included in a database and starts from a set of possible seismic scenarios. By using fragility curves of bridges the damage state of the network links, in which bridges are included, can be obtained. By making a series of hypothesis on how a bridge damage state is correlated with a link damage state and how a link damage state can influence its functionality, has been created a set of damaged networks. In this set of damaged networks has been assigned the transportation demand and the fluxes have been calculated. The total network time has been calculated in order to evaluate the network delay that the damage causes. At the end of the process the network risk curve (probability of the seismic action vs. total delay) is derived. Some possible applications of the procedure are the evaluation of the order of bridge retrofitting and the analysis of the emergency response. The procedure developed in general terms has been applied to a test network near Treviso in the north-eastern part of Italy.

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Inspection process of Bulgarian Bridges

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The Road Administration of Bulgaria and Advitam set up jointly, in 2003, a national scale solution to all these questions. Rehabilitation is not a matter of deciding how much money to deploy, but shall/can fit to available budgets. The project mainly stands on three steps:

– A global audit including regulations in effect as well as the infrastructure network. Practical methods and techniques allowing it within limited time and controlled budget from the beginning
– A global infrastructure network evaluation, using objective indicators through synthesis tools
– The setting and application of a progressive director scheme, from partial rehabilitation projects towards a global preventive maintenance management.

To support this project, Advitam was in charge of developing a Bridge Management System (BMS) based on its software suite: ScanPrint. From mid of October to mid of December 2003, inspectors traveled around the country and gathered information and pictures about the 1312 bridges. Advitam provided technical assistance in the various regions to finalize the training.

By the end of the project, the REA has been provided with the following information system:

– Standard procedures for bridge inventory and inspection
– Database with information about all bridges. This information may be accessed by many means and be easily transmitted to local administrations:
  • ScanPrint interface
  • Excel tables resuming information
  • Web interface containing inspection results
– Software and trained teams to periodically inspect the bridges condition. The correct following of the inspection calendar defined in the Bulgarian norms is now facilitated.
– Software to help the decision about actions of maintenance. The systematic link between defects and respective actions to undertake can now be done automatically.
– Preliminary repair projects for 15 critical bridges. These projects will serve as basis to launch repair works and can be used as reference in terms of actions and costs for future similar projects, including:
  • Defect total quantities from detailed inspections, which allowed to state directly on the necessary light repair budget for the 15 bridges
  • Recommendations for heavy repairs (replacement of equipments as bearings or expansion joints, structural reinforcement, etc…)
  • Recommendations for further investigations (durability tests, monitoring, etc…)
– A global maintenance master plan on the 1312 Bridges, stating when to re-inspect the bridges, according to their technical and final condition rating, and to the criteria selection. A global decision tree has been worked out with the REA in order to step towards a global preventive maintenance management plan of all bridges over 20 m in 10 years.

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Current state of highway transportation asset management in Germany

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ABSTRACT

In Germany the federal road network carries the main load of the transit traffic by reason of its central position in Europe. Currently it contains about 53,000 km of motorways and trunk roads including about 38,300 bridges.

The main part of the network was built in the 60th up to the 80th of the last century. Over the years a very high quality of construction methods was achieved. Nevertheless maintenance needs today are in the range of about 1.7 Bio € per year. Maintenance costs have to be spent in a way to obtain the greatest possible benefit. This task is supported by an comprehensive Asset Management.

Asset Management can be considered to be a systematic process of maintaining, upgrading and operating assets. It provides tools to facilitate a more organized approach to making the decisions necessary to achieve the public's expectations. In Germany the Asset Management procedures include all the processes, tools, data and policies necessary for the effective management of the federal road network.

In Germany the system is essentially influenced by the federal constitution. The Federal Government is the owner of the federal road network however the states administer these roads under their own responsibility.

In the frame of the current German Asset Management firstly the Federal Ministry of Transport is supported with an overview of the current situation of the road network and with statements concerning financial requirements as well as possible strategies for realizing long-term objectives and fulfilling basic conditions for maintenance routines. Secondly, the states and authorities are supplied with recommendations for performing improvements in compliance with strategies, long-term objectives, basic conditions and budgetary restrictions.

The required consulting service includes for example network-related analysis and evaluation. For this purpose, a methodological basis was developed. Cornerstones are a federal database, which covers all the required information about the road network and management systems for pavements and structures.

Whereas operative tasks are performed by the state administrations, the federal Ministry of Transport has the significant mission to define specifications and to supervise and control the implementation and achievement of objectives. Aims of the Asset Management are:

– a sustainable and efficient securing of substance and function of the network
– a reliable assessment of the development and extension needs and
– the efficient operation of the federal network

However, the complexity of these system continues to increase and an holistic approach is necessary to understand the effect of technological, environmental, economical, social and political interactions on life cycle aspects. In order to accomplish this, methods have to be developed to systematically analyze the whole lifecycle, and models have to be formulated for evaluating and comparing the risks and benefits associated with various alternatives on a probabilistic basis.

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Network level deterioration modeling: A case study on masonry arch bridges

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ABSTRACT

Engineering systems comprise a series of interconnected elements, which behave in a complex but integrated manner to perform their intended function. Being exposed to harsh environmental conditions, and owing to their critical position within the transport infrastructure, effective management of bridges is vital for the reliability of overall transport system. Deterioration of bridge elements and sub-elements may lead to a reduced level of service derived from the transport network. Traditionally, element level management approaches have been extrapolated in managing system performance, especially in dealing with possible changes due to wear and deterioration.

Predicting deterioration is a vital component of any modern bridge management system. It is generally established for individual bridge elements, through inspection records, thus leading to an approximate prediction of bridge performance over time. Several methods are available in literature to account for such elements vs system interactions. BBN's are capable of handling complex relationships between the elements and the system by means of conditional probabilities, and can also address the variation of performance with time and the updating of performance with new information, both important considerations in a bridge management context.

The development of a network level deterioration model using Bayesian Belief Networks is presented in this paper. Conditional probabilities between bridge elements and initial element conditions are assumed on the basis the inspection methodology and inspection results from a sample group of UK masonry arch railway bridges. A BBN model representing the network level deterioration of masonry arch bridges is shown in Fig. 1. Three condition states are considered for each element in the BBN. 'Poor', 'Fair' and 'Good' states are represented by SCMI score range of 0–45, 45–80 and 80–100 respectively (see full paper for details). The probability of an element in these states is derived

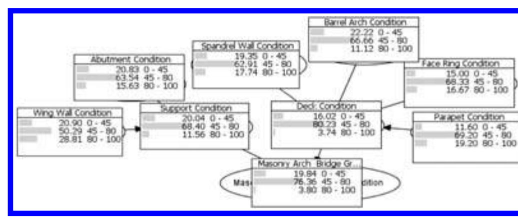

Figure 1. BBN model of network level deterioration for masonry arch bridges.

from the sample bridges and is shown in Fig. 1. The overall condition of the network is predicted using the BBN. The figure shows that a majority of the elements are in 'Fair' condition, leading to a higher probability for the overall group being in the 'Fair' condition.

A desirable feature of the BBN models is their ability to analyze 'what-if' scenarios. This is particularly helpful for the prioritization of assessment and maintenance intervention activities. This feature has been demonstrated though a sensitivity analysis to obtain the relative importance of each minor element on the overall bridge group condition.

It is evident from the results that initial element conditions are crucial in determining not only the network condition but also in understanding the maintenance/intervention priorities. Hence, for practical applications these should be obtained as accurately as possible. This, in turn, leads to the question of what inspection techniques should be used, how often and with what priority. In answering those questions, the usefulness of the BBN model lies in its ability to deal with a wide range of scenarios both readily and transparently.

The BBN model can be used to compare various inspection techniques and to estimate appropriate inspection intervals with the objective of maintaining a consistent risk/reliability target within any particular bridge network.

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Road infrastructure asset management – a holistic approach to road infrastructure supply

J.M. Jansen
Danish Road Directorate, Operation & Maintenance Division

ABSTRACT

This paper gives a view on the concept of road infrastructure asset management in context with the concept of modern road administrations as corporate businesses, which mission is delivering of a contemporary transport network and traffic management to support societal prosperity and growth.

Management of the road infrastructure asset encompasses a holistic management of all road infrastructure elements in the corridor, both capital works on pavements, structures and installations and routine maintenance works on areas and equipment.

Based on work in PIARC committee on Asset Management and national research on asset management approach, this presentation will paint a picture of the multidiscipline complexity that has to be accomplished to execute effective asset management.

Asset management is not only to provide best practice maintenance, but also to deliver perceived outcomes to stakeholders' value. To do this, much more knowledge than just engineering has to be incorporated in the process of road infrastructure asset management.

With introduction of road infrastructure asset management also internal processes in the road administrations has to be reengineered, as the holistic approach will change the basis of decision-making from sub-asset decisions to total asset decision-making, which means that old habits has to be reconsidered, and the traditional budget competition between road network engineering disciplines will be replaced by objective measurement in stakeholder surveys on perception of delivered asset management outcome. Different stakeholder groups have their own perception of outcome value, and this is the true driver of effective asset management decisions.

This paper presents the complexity of asset management for modern road administrations and give examples of how this process has been approached; – in view of nobody has a complete asset management in operation.

Major learning from asset management is to adjust the choice of enablers for intervention to secure stakeholders expectations for perceived outcome.

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A holistic approach to asset management in The Netherlands

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ABSTRACT

The Directorate General for Public Works and Water Management is responsible for the management of the national roads and water infrastructures in the Netherlands. This agency, further to be referred as Rijkswaterstaat, is an executive agency within the Dutch Ministry of Transport.

Approximately 9.000 fte are working on 3.300 km (~2,000 miles) strategic highways, 1.700 km (~1.000 miles) strategic waterways and 65.250 km^2 (~25.000 square miles) water systems (such as dunes and surface water). The annual budget for maintenance, operations, renewal and expansions of those networks is approximately € 5 billion.

A comprehensive approach to infrastructure management needs integration in the decision making process and therefore reliable asset data, a long term performance based maintenance program, clear steering objectives and transparent procurement strategies. The introduction of asset management is used as the integrating principle. The goal is: "to deliver best service to the public at lowest life cycle cost, given public acceptable risk".

Within the asset management process three roles can be distinguished: the asset owner, the asset manager and the service provider (table 1).

Rijkswaterstaat decided to implement asset management in 2008. The program should be completed at the end of 2011. In this program four sub-programs are defined:

1. Reliable and accurate asset data
2. Stable long term maintenance programs
3. Clear objectives and transparent requirements
4. Transparent procurement procedures.

Asset data management calls for a network oriented approach. In this approach a decomposition of all object classes can be handled effectively.

Development of stable long term maintenance programs (including prioritization) is another important development at Rijkswaterstaat. The existing object class oriented programs contain elements for strategic and operational planning, but they were not integral and not based on strategic goals. (Klatter & van der Vusse,2004).

Clear objectives and transparent requirements are important for justifying the maintenance programs. Each hierarchal level in the decomposition needs a specific description. Also requirements should be connected throughout the hierarchical levels. System Engineering principles are used to connect all these requirements and to assure compliance within the different levels.

The policy in the Netherlands is to outsource activities to private parties to an extend as large as possible. All maintenance and construction works are contracted out to private companies, contractors and engineering firms. This means that 100% of the final output and outcome is realized through these contracts. The asset management program will be completed within two years. The full implementation and management of the system thereafter should not be underestimated. Today de results of the program can already be seen in the organization, in a grown insight in the asset quality, maintenance needs and effects on performance and risks.

Table 1. The asset management roles.

Asset owner	Asset manager	Service provider
Responsibilities and tasks		
- Overall network policy - Targets for performance and condition on a network level - Target for acceptable risk profile	- Investment strategies - Maintenance concepts - Technological standards - Risk management - Network management	- Project delivery - Maintenance execution and services - asset data management - project management

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Towards sustainable life cycle design of highway bridges

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ABSTRACT

Highway bridges provide a core public service and are needed for a very long time to support the sustainable development of neighboring communities. They are critical links in Canada's transportation network that should be kept safe and functional during their service lives to enable personal mobility and transport of goods to support the economy and ensure high quality of life. The average service lives of these assets vary from 50 years up to 100 years, which are continually extended by using different management strategies that include different combinations of preservation, rehabilitation, strengthening, replacement, mitigation and adaptation actions. The growing concerns for environmental protection and the shift towards achieving sustainable transportation infrastructure are now requiring the use of approaches that can achieve an adequate balance between social, economic and environmental performance over the entire life cycle of the bridge.

Relevant and practical performance measures are needed to support the decision-making at different levels of hierarchy and to ensure the social, economic and environmental sustainability of bridges, as well as the sustainability of transportation and the communities they serve.

This paper discusses some performance indicators, such as safety, serviceability, costs, traffic disruption, greenhouse gas emissions, which can be used for life cycle design of highway bridges. An example, taken from the North American context, illustrates how different design and rehabilitation approaches can contribute to achieve the desirable balance between social, economic and environmental sustainability criteria. The example illustrates the benefits of implementing a life cycle-based design approach through the construction of high performance concrete highway bridge decks that could yield reduced environmental footprints and lower social pressure.

The long-term efficiency of high performance concrete (HPC) containing supplementary cementing materials (SCM) is evaluated and compared to that of normal performance concrete (NPC) in terms of service life, life cycle cost, environmental impacts and social impacts. A limited number of performance indicators are chosen to illustrate the proposed approach to sustainable design, which include:

- Economy: life cycle agency costs and life cycle user costs.
- Environment: CO_2 emissions and volume of construction waste materials.
- Social: Accident cost and traffic congestion.

The use of HPC materials results in bridge decks with extended service lives, reduced life cycle costs, and better environmental and social profiles when compared to conventional normal concrete bridge decks.

In this particular example, all the performance measures used indicated that the HPC deck alternative would be a better choice. There is no need to trade off or reach an equitable balance between environment, social and economical. Only on initial construction cost would the NPC bridge deck option perform better, but initial construction cost is not a suitable indicator for sustainability.

In life cycle performance analysis, the reliable estimation of service life and associated schedule and extent of MR&R activities is critical. Beyond their significant influence on economic performance, they also drive the results of the social and environmental impacts analysis. For bridges with high level of traffic, the social and environmental "costs" of frequent and/or extended interventions should be taken into consideration to move towards a sustainable approach for the design and management of highway bridges.

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Green stormwater infrastructure: A prototypical multi-domain, complex infrastructure system

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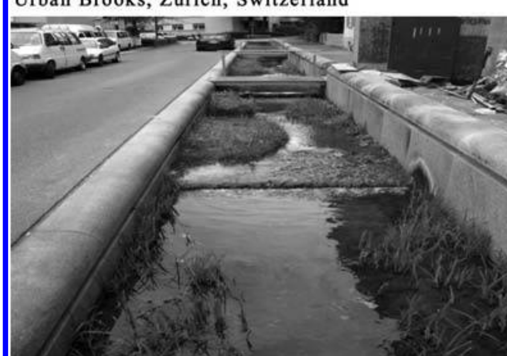
ABSTRACT

Runoff generated from impervious surfaces in urbanized watersheds is an extensive liability for water utilities which must meet increasingly stringent federal regulations governing its discharge. Traditional "grey" infrastructure solutions to this problem are costly because they involve expansion of existing collections systems and/or creation of in-line storage. A new suite of "green" infrastructure (GI) approaches would reduce the quantity of runoff by addressing its source, while also addressing many urban sustainability goals. However, to be effective, such approaches (1) would need to be implemented widely in urban watersheds, (2) would require coordination/collaboration between multiple stakeholders, (3) would need to be implemented relatively rapidly and efficiently, and (4) would require the development of new decentralized maintenance and quality control measures.

Through description of these four challenges, this paper presents green infrastructure as a case study in multi-domain, complex infrastructure planning. For green infrastructure to become part of urban stormwater management plans, many challenges need to be overcome. These include overcoming spatial constraints, and engendering meaningful levels of participation, in reasonable timeframes, with adequate quality assurance. To be effective, green infrastructure projects also need to be implemented in both the public right of way, and on private lots. While the challenges are formidable, they must be considered as the only alternative to investing billions of dollars in concrete tanks and tunnels that, while they may control urban runoff, provide few other benefits to urban residents.

Indeed, the future of the management of water infrastructure assets will be informed both by participatory processes and lessons learned from the fields of urban, transportation, and watershed planning. It will also require new vision. We need to view highways as more than simply conduits for the passage of cars and trucks. Streets and highways can be habitat corridors and infiltration parks, or be used to daylight historic streams, or create new ones. We need to view buildings as more than simply places for us to live and work. Buildings can also cool our cities, and reduce our consumption of energy and natural resources.

Urban Brooks, Zurich, Switzerland



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Managing catastrophic risk to Istanbul's housing infrastructure

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ABSTRACT

This study conducted a comparative analysis of mitigation alternatives for a destructive scale earthquake (magnitude 7.68) in the Zeytinburnu District in the City of Istanbul, Turkey. Zeytinburnu is particularly vulnerable to earthquakes due to the district's close proximity to the North Anatolian Fault, alluvial soil, and seismically weak residential building stock. The mitigation strategies investigated within the scope included seismic retrofit to reduce the probability of building collapse during an earthquake, relocation of the family to a seismically-stable dwelling, or to take no action at all.

The study considers economic loss for the most vulnerable residential building groups including poor quality, mid-rise reinforced concrete (RC) frame buildings and weak masonry structures. The focus on these two building types is driven by the fact that 86% of the residents of Zeytinburnu live in mid-rise, clustered, apartment houses, 69% of which are considered unsafe. Masonry dwelling units in Zeytinburnu are generally of poor quality and would not likely provide the lateral resistance to withstand a major seismic event. Viable retrofitting measures appropriate to these types of buildings were identified in the study.

Decision trees were developed in order to compare expected values of the three earthquake mitigation alternatives. Separate evaluations were conducted for both building types. The RC units were analyzed at three levels of damage, while two probabilities were considered for masonry structures; destructive scale damage and no destructive scale damage. The total loss calculated for each outcome was the sum of property loss, mortality loss (in high damage scenario) and cost of the selected action. The expected property loss was a function of the damage ratio and value of the housing unit. The expected costs for the RC scenario were $9,372 for the retrofit option, $9,760 for relocation, and $17,612 if no action were taken. The expected costs for the masonry building scenario was $18,137 for the retrofit option, $18,301 for relocation, and $23,157 if no action were taken. While there does not appear to be substantial benefit of adopting retrofitting measures over relocating to a seismically-safe home, mitigation clearly is preferred over taking no action at all. A sensitivity analysis indicated that the mitigation alternatives were preferred over the no action alternative over the full range of parameters values. While this is a preliminary study, that required many assumptions and is subject to substantial uncertainty, the expense of the mitigation efforts appears justified. While future efforts to improve the risk assessment are justified, efforts to improve risk communication and develop public policies to reduce this risk.

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System-identification of multi-domain (human, natural, and engineered) infrastructure systems

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ABSTRACT

Population growth and urbanization trends pose challenges for existing civil infrastructure systems. Unfortunately planning for such infrastructure systems has usually not considered system-level impact, which has led to a wide range of unintended impacts. Construction of highways resulted in low density sprawling development and increased the extent of impervious surfaces, creating storm water runoff which adversely affected aquatic ecosystems.

While the importance of the interactions between infrastructure systems is becoming increasingly evident, models that can be used to simulate, visualize, and predict these interactions are still rare. Even within infrastructure systems, existing asset management approaches are fragmented, generally internalizing the opportunities and constraints of only a single asset class within a given system, such as bridges or pavement. During an era of such critical planning for the future, there is a need to devise new integrated, multi-domain decision-making models that directly incorporate the important interactions that exist between layered infrastructure systems, and the human and natural communities that are impacted by them. In this paper we propose a process for systematically identifying the human, natural, and engineered systems comprising infrastructure systems and developing an asset management system that manages these assets in an integrated fashion. The proposed approach consists of five steps that begin with system identification and culminate in the development of software system for integrated asset management. The five steps of system development are as follows: 1) Identification of Information Flows, Institutional Knowledge Repositories, and Management Metrics 2) Assessment of Forecasting Tools and Needs 3) Development of Asset Management System Architecture, 4) Database Development, and 5) Database Interface Development and Refinement.

The process should be based on what software developers call an iterative and incremental development model, rather than the traditional waterfall management model of software development by successive execution of pre-defined tasks. The iterative and incremental approach allows the software developer to implement only a small subset of software requirements and iteratively enhance a sequence of versions with additional functional requirements until the full system is implemented. The need for such an approach is derived from the uncertain nature of large, layered infrastructure systems with dynamic boundaries used and managed by multiple stakeholders. A key management objective in such systems is to be able to accommodate changing performance goals in response to the needs of multiple stakeholders with influence over multiple time horizons and scales.

As future research, the authors plan to develop a prototype asset management system based on the proposed five step approach and validate the prototype system with real case studies.

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A systems approach to assessing the sustainability of the Grand Canal of China

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ABSTRACT

The prevailing approach to infrastructure sustainability studies today is to implement changes in existing facilities, monitor these changes, and compare the results to the initial conditions (Bell and Morse, 2003, 2008). One of the most obvious drawbacks of this forward approach is that people tend to draw conclusions too quickly. The uniqueness of this study is that of looking back by studying the historical materials of the Grand Canal of China, a centuries-old critical infrastructure system.

The Grand Canal of China started in Beijing in the north and ended in Hangzhou in the south, with a total length of about 1,250 miles. For more than six centuries (1289–1905), the grain supplied to the personnel of the imperial court, as well as to the troops stationed in Beijing and along the Great Wall, was transported via the canal (Li, 2008; Wu, 1999; Yao, 1998). The activities of the canal were recorded in detail throughout its life cycle. Many of these documents have survived and are in libraries in the US, China, and Taiwan (Liao, 2006).

The temporal focus of this study is from 1770 to 1870, which is the period during which China as well as the canal declined rapidly. Although his early years saw the continuing of an era of prosperity in China, historians generally agree that later in his reign China began declining. Spatially, this study focuses on the canal at Huai'an, where it crossed the Yellow River until 1855

Sustainability indicators such as economic, institutional, and environmental as well as indices relevant to these indicators can be examined in the historical-geographical context of the Grand Canal of China. In this systems approach (Clayton and Radcliffe, 1996), an Infrastructure Sustainability Indicator (ISI) is developed, based on the three sustainability measures: economic, environmental and social.

Each of these sustainability measures is then considered and examined in terms of factors that might influence them. From this preliminary list, a subset of indices are selected that take into account limitations on the availability of data. These indices are then normalized so that they can be combined into each sustainability measure, using weighting functions. The three sustainability measures (economic, environmental and social) are then combined into the Infrastructure Sustainability Indicator (ISI), again using weighting functions.

This brief paper is dedicated to the background of the approach and the selection of appropriate indices. This process examines the issues that were important over the 100-year study period, and utilizes the advantage of hindsight to relate these factors to available data.

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Strategies of investment in the management of urban bridges

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ABSTRACT

Decisions on maintenance and rehabilitation strategies for a portfolio of bridges should be based on life cycle cost and benefit analysis (LCC). With a limited source of funding, however, further strategy is needed to determine the priority and timing of expenditure. The communication and presentation of programs to the public at large are extremely important.

This paper discusses and broadens the perspectives of bridge management, providing a life cycle cost savings and investment tool to guide decisions regarding bridge maintenance and rehabilitation. A spreadsheet computer program is developed to incorporate user supplied costs and benefits, and to rank different projects by priority, using the savings to investment ratio (SIR), a measure of cost-effectiveness.

The program is based on a probabilistic approach, with the SIR involving the initial investment; annual operation, maintenance and repair/rehabilitation costs; and failure costs. These future costs are linked to probable events, such as natural hazards, vehicle and boat crashes, and user time delay. Life cycle cost analysis can be sensitive to the selection of an appropriate time period, and this paper introduces the concept of infinite lifetime for bridge maintenance. Consideration is also given to aesthetic considerations, which for a city such as Paris has economic consequences.

An example of use of the program is shown, so that the developed method is not viewed as either too theoretical or vague. The chosen example is Paris, France. Crossed by 37 bridges on a 13 km-long path, the Seine River has exposed the French capital to floods for millennia. The data are mainly provided by the Office of the Mayor of the City of Paris, and are complemented by assumptions when required. This application is especially interesting since Paris has a very diverse portfolio of bridges, with a wide variation in dates of construction, material and their dimensions.

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Eco-audit of seven green infrastructure practices

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ABSTRACT

This paper describes an Eco Audit of seven different urban green infrastructure practices. It analyses the amount of energy embodied in the materials used in the construction of the BMPs and the CO2 emissions footprint associated with their production. Different BMP scenarios are applied to both a typical one-way and a typical two-way New York City Street and compared.

For the 152 m long, one-way street, the following scenarios were compared:

- No BMP
- BMP scenario 1: sidewalk permeable paving, 0.1 ha
- BMP scenario 2: sidewalk reservoirs with street trees, 0.1 ha
- BMP scenario 3: parking lane permeable paving, 0.1 ha

Both BMP scenario 1, the sidewalk with permeable paving, and BMP scenario 3, the parking lane with permeable paving that receives runoff from sidewalk and street, lead to a significant reduction (9–23% and 2–9%, respectively) in embodied energy and CO2 footprint. In contrast, BMP scenario 2, the sidewalk reservoirs with street trees, leads to a significant increase (58–75%) in the environmental impact.

For the 152 m long, two-way street, the following scenarios were compared:

- No BMP (Table A5)
- BMP scenario 4: sidewalk permeable paving, 0.1 ha (Table A6)
- BMP scenario 5: sidewalk reservoirs with street trees, 0.1 ha (Table A7)
- BMP scenario 6: sidewalk biofiltration, 0.1 ha (Table A8)

The picture that emerges from the analysis of the two way street is similar to that of the one way street. BMP scenario 4, the sidewalk with permeable paving, leads to a significant reduction (7–19%) in embodied energy; BMP scenario 6, the sidewalk with biofiltration, has an embodied energy and CO_2 footprint that is very similar to the "do nothing" case. In contrast, BMP scenario 5, the sidewalk reservoirs with street trees, leads to a significant increase (73–83%) in the environmental impact.

The design of environmentally benign BMP scenarios has many aspects; one is eco aware materials selection, the choosing of construction materials with low embodied energies and CO_2 footprints. The Eco Audits reported above are quick, approximate assessments of energy consumption and CO_2 emissions that are associated with the construction materials used in alternative BMP scenarios applied to a typical one-way and a typical two-way New York City street. They reveal which components of the design dominate the environmental impact and provide guidance for the decision-making process and the exploration of alternative scenarios.

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Determination of a performance baseline for lifecycle consideration of bridges

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ABSTRACT

There is global agreement that bridge rating based on subjective visual inspection needs to be improved. Shrinking budgets of our transportation infrastructure are contrary to the growing demand identified for our aging structures. It is a fact that not all deficiencies identified can be mitigated. Therefore risk management is a must in order to determine priorities and a ranking of necessary investments.

The European approach has been focused on ambient vibration monitoring since 15 years. Considerable progress has been made and prototypes of bridge management systems providing decision support based on case based reasoning routines have been built. In the large European Research Project IRIS (CP-IP213968-2) this procedure is further developed into a tool, which, in combination with a reduced visual inspection, has demonstrated that our current rating practice produces in average too low ratings.

In the European bridge stock (about 1 million bridges operated by some national authorities) 12% are rated deficient. It is anticipated that only 2% deserve special attention. The proposed procedure therefore knows 2 levels of approach:

– A quick and cheap basic assessment campaign to produce a ranking of the structures
– An elaborate risk monitoring and assessment routine which goes deeper into detail and produces recommendations for rehabilitation

The methodology has been applied to a large bridge stock in Europe and is able to show first results. For structurally deficient bridges the ranking is improved in average by over 25%.

Lifecycle Modeling has 2 aspects:

– The safety and reliability aspect covering the bridge owners responsibility to provide a save and reliable infrastructure

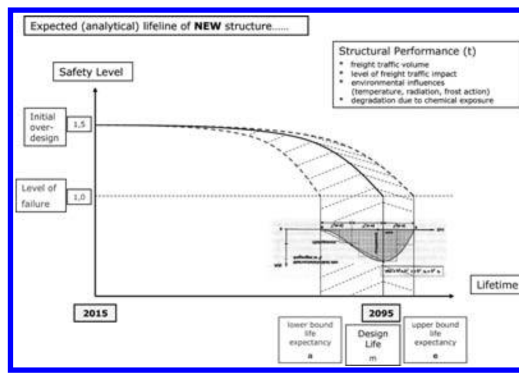

Figure 1. Expected (analytical) lifeline of new structures.

– The operation and maintenance aspect covering mainly the optimization of the financial demand for preventive maintenance and budget planning

Conventional lifecycle models are based on the information provided by the respective databases. In order to introduce objective values for assessment monitoring results are utilized. In the following description this process is elaborated.

The presented approach has been developed for Central European conditions and the related database and information structure. Nevertheless the principle can be applied globally when the respective interfaces are fitted for the specific purpose.

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Study on evacuation simulation system for urban expressways using multi-agent system

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ABSTRACT

Recently, the occurrence of the large-scale earthquake will be expected with the high probability in Japan within 30 years in the future. Therefore, performance securing of infrastructure facilities and the user's life securing are strongly demanded when disaster occurs, the disaster prevention and disaster mitigation measures are matters of great urgency. Since Great Hanshin Earthquake, introduction of new earthquake-proof and seismic isolation technology that accompanies review of earthquake resistant design code of structure and earthquake strengthening of structure made from design standard before Great Hanshin Earthquake are executed. As a result, the protection in the hardware side is steadily promoted. However, to do prompt restoration and restart of business after suffering, it is not only measures of the hardware side, and it is important to examine disaster prevention and the disaster mitigation measures on a soft side concerning a prompt evacuation conduct from collapsed structure. Especially, early restoration is demanded as for the expressway. Because it is an important infrastructure for transportation of the rescue supply and prompt disaster recovery.

In this study, simulation system for Hanshin Expressway that is urban expressway is developed to examine disaster prevention and disaster mitigation measures from soft measures. And, car and person's evacuation guides in the expressway are examined. Specifically, the evacuation simulation system is developed that constructs behavior model of evacuation car and evacuee when disaster occurs using MAS (Multi-Agent System). This system made the simulation process visible by using JAVA applet, and it can change the simulation condition in real time. Therefore, using the system, different persons concerned with disasters such as inhabitants, the persons concerned with fire fighting, the expressway manager, the administrations, the experts and so on can exchange opinions about the evacuation and reflect and plan a different of view.

Moreover, by using this developed system and reproducing the disaster situation (Impassable by destruction of structure, Collision of car to structure, Collision of car to structure, Fires, etc.) on the expressway that can happen when the large-scale earthquake occurs, the method that evacuates car and person on the expressway from the situation promptly and safely is examined. In this paper, "Influence that transmission of disaster information and roadway securing instruction for emergency car from expressway manager gives to evacuation time of car" and "Influence that number of evacuation gates gives to evacuation time of evacuee" when the disaster occurs are examined. And, the effectiveness of the evacuation guide is examined.

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MS10: Bridging the data gaps for effective management

Organizer: H. Ghasemi

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Defining and implementing data collection goals for the Long Term Bridge Performance program

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ABSTRACT

The Federal Highway Administration has initiated the Long Term Bridge Performance (LTBP) program as authorized in the "Safe, Accountable, Flexible, and Efficient Transportation Equity Act: A Legacy for Users (SAFETEA-LU)", signed into law in August of 2005. The LTBP Program is a 20-year, multi-faceted research effort that is strategic in nature and has both specific short-term and long-range goals. It is anticipated that the LTBP program will provide a better understanding of bridge deterioration, focusing on its numerous causes including corrosion, fatigue, environment and loads. The program will collect information regarding the effectiveness of current maintenance and improvement strategies, and information on the operational performance of bridges, focusing on congestion, delay and accidents. It is envisioned that the quantitative database developed under the LTBP program will be used to solve a variety of bridge condition assessment and management problems, to develop new tools, and to advance the state-of-the-knowledge of bridge design, maintenance and preservation.

An interview process was adopted to assist the team in achieving the following objectives:

1. Develop an understanding of how representative states manage and track bridge performance.
2. Identify the most common concerns and the most costly activities of the representative states in maintaining, repairing, rehabilitating and replacing bridges.
3. Determine what data the states currently collect and use for their decision-making processes and what gaps they see in their currently available data.
4. Identify the aspects of bridge performance on which the states would like the program to focus.
5. Develop and support a community of practice for the Long Term Bridge Performance program.

Based on the comments and recommendations provided by state highway agencies, the LTBP team identified a list of research topics related to various aspects of bridge performance. The list was compiled, and each topic was supplemented with supporting background information and literature review, definition of general scope. The compiled list was then shared with a select group of experts from government, academia and industry who provided input regarding the perceived urgency and importance of each of the proposed study topics.

Each of the research topics have been developed by outlining known background information, identifying knowledge gaps or limitations in practice, and then posing a series of research questions, with associated hypotheses and data requirements, to arrive at specific types of data needed to address the topics. These data requirements will be further refined into protocols for the gathering of specific data and methods for conducting associated tests or field evaluations.

Using the methodology outlined herein, the LTBP team has established priorities for the program, determined specific study objectives and identified data required to fulfill those objectives.

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Bridging the data gaps for effective management, session 3: performance – LTBP data infrastructure and data integration

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ABSTRACT

The Long Term Bridge Performance (LTBP) program aims at an in-depth understanding of the underlying causes for bridge structures performance issues, such as deterioration processes for example. A better understanding will ultimately lead to a better way of managing the bridge infrastructure, minimizing life-cycle costs and improving the current asset management strategies. In order to achieve this goal, during the LTBP program massive amounts of data will be collected from different sources, such as retrospective inventory data, environmental information, visual inspection reports or instrumentation data, to mention a few. The main hypothesis is that putting together all these pieces of information will help in a better understanding performance issues and generate more accurate predictive maintenance strategies, optimized life-cycle cost models and overall improve maintenance policies. The central piece of this undertaking is a sound data management framework that facilitates the integration of information and the mining of knowledge nuggets commonly referred to as data mining. This publication provides an overview of the data integration and management framework build as part of the LTBP program.

Three requirements and their solutions will be described in more detail. Namely, a software architecture to ensure extensibility, a data integration framework to allow for the seamless integration of additional data sources, and a data security concept to enforce data safety.

A hybrid data integration framework is proposed taking advantage of the two main data integration approaches, the *fused data* and the *federated data* approach. The introduced framework allows the integration of multiple heterogeneous data through a centralized data warehouse as well as through a service oriented architecture (SOA) that links data from decentralized locations. This framework is highly scalable and extensible, providing the necessary foundation for the Long Term Bridge Performance Program.

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LTBP pilot study methodology and preliminary results

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ABSTRACT

The Federal Highway Administration's (FHWA) Long Term Bridge Performance (LTBP) Program is a 20-year, flagship research program aiming to obtain scientific quality data which can be used to, among other things, better understand bridge performance and drive data-based bridge management methods.

The Pilot Study Phase of the LTBP Program has begun, seeking to test data collection methodologies and obtain useful data early in the life of the Program. Seven bridges scattered throughout the United States are involved in the Pilot Study, each representative of typical environment conditions and structure types common in the Nation's broader bridge population. These bridges provide a platform for researchers to validate, streamline, and field test the inspection, testing, and instrumentation procedures prescribed earlier in the development phase of the Program, and ensure the necessary components are in place to achieve the Program's objectives once it is expanded to a larger sample of the population.

To achieve the objectives of the LTBP Program, researchers are using a complementary package of both traditional and state-of-the-art data collection techniques. At the time of this paper, researchers have conducted initial testing at four of the pilot bridges to establish a baseline condition for comparison during long-term monitoring.

A detailed visual inspection, the industry standard for monitoring deterioration, has been completed. More detailed than a typical biennial inspection, LTBP Program inspections feature the use of ScanPrint computer software, allowing inspectors to accurately record the size and location of a defect electronically onto a CADD bridge plan while in the field.

In conjunction with the visual inspection, LTBP Program Pilot Study bridges are also evaluated using a suite of non-destructive testing and evaluation (NDT/NDE) techniques. These tools are the next generation of inspection technology, allowing researchers to evaluate corrosion in a bridge deck throughout its lifecycle, from detecting corrosion potential at its earliest stages to characterizing ongoing delamination. The LTBP Program is using half-cell corrosion potential surveys, ground penetrating radar, impact echo, and ultrasonic testing at all pilot bridges. Utilizing this full-range of NDE tools creates a comprehensive understanding of the current health of the bridge and allows an early diagnosis of underlying problems, enabling researchers to trace the progression of deterioration before it can be detected at the surface.

Various concrete testing techniques are also being used in the Program to obtain more detailed information on material properties, such as chloride content, and to serve as ground truth verification of NDE testing.

Live load and dynamic testing are being used at each pilot bridge to analyze the structural behavior of the bridge system. The baseline structural conditions currently being recorded will allow researchers to track changes in structural behavior and critical detail behavior over time, and compare it to the material deterioration data obtained in inspection, NDE, and concrete testing.

Close coordination of all field activities is vital to the success of the pilot program. To minimize disruption to traffic flow, multiple tasks are consolidated and completed during a single lane closure whenever possible. This requires numerous LTBP teams, including the visual inspectors, NDE/NDT operators, and deck monitoring crew, to share space on the bridges with limited periods of time to conduct their work. To ensure activities are carried out efficiently, safely, and to the satisfaction of the bridge owners, tremendous amounts of coordination and communication are required.

The preliminary results of this array of tools are now being analyzed and correlated to begin to answer critical questions regarding bridge performance.

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Enhancing bridge performance – defining/measuring/improving bridge performance

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ABSTRACT

The transportation user community in the United States expects and deserves a highway system that routinely provides high quality service in terms of safety, efficiency and economy while having little or no negative impact on the local or global environment. Optimal (i.e., safe, efficient and economical) operation of public highway systems is dependent on many factors. Bridges are critical nodes in the highway infrastructure and poor performance of any highway bridge can reduce the operating condition of the highway system of which it is part. Four aspects of bridge performance are critical to owners and highway users alike. These are: the structural condition and durability of the bridge, the functional capacities of the bridge, the integrity of the bridge in relation to potential failures and the overall costs of maintaining a bridge in service over its lifespan.

In the United States, there exists a current knowledge base of inventory information and condition data on bridges that is by far the best in the world. Current programs and methods of bridge inspection and the tools used for managing bridge programs are also unmatched anywhere. Yet the level of understanding of how bridges perform and how to satisfactorily measure their performance falls well short of desirable. This is because the performance of any single bridge is dependent on complex interactions of multiple and often interrelated factors which include: the original design parameters and specifications (bridge type, materials, geometries, load capacities); the initial quality of materials and of the as-built construction; varying environmental conditions of climate, air quality, etc; corrosion and other deterioration processes; traffic volumes and types, weights and volumes of truck traffic; and the type, timing and effectiveness of preventive maintenance, of minor and major rehabilitation actions and ultimately of replacement actions. All of these factors combine to impact the condition and operational capacities of the bridge and its various structural elements at any given point in the life of the bridge. This session will provide a clear and complete perspective on the concept of bridge performance and present the path to a better understanding of what bridge performance means and how to reliably measure it and improve it.

Table 1. High priority bridge performance issues.

Issue
Performance of untreated concrete bridge decks
Performance of bridge deck treatments (membranes, overlays, coatings, sealers)
Performance of precast reinforced concrete deck systems
Performance of alternative reinforcing steels
Influence of cracking on the serviceability of high-performance concrete decks
Performance and repair of bridge deck joints
Performance of jointless structures
Performance of coatings for steel bridge elements
Performance of weathering steels
Performance of bare or coated/sealed concrete superstructures and substructures (splash zone, soils, or exposed to deicer runoff)
Performance of embedded or ducted prestressing wires and post-tensioning tendons
Performance of prestressed concrete girders
Performance of bridge bearings
Direct, reliable, timely methods to measure scour
Performance of scour countermeasures
Performance of structure foundation types
Performance of innovative materials and designs
Risk-based management approach
Operational performance of functionally obsolete bridges

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NDE condition assessment of reinforced concrete bridge decks within the LTBP program

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ABSTRACT

Reinforced concrete (RC) bridge decks are exposed to several types of deterioration processes: corrosion, alkali-silica reaction (ASR), carbonation, shrinkage, freeze-thaw actions, etc. The most commonly found problem is corrosion induced concrete deterioration, which is initiated by rebar corrosion and afterwards extends into concrete cracking, delamination and ultimately concrete spalling. Previous studies have shown that surveys of bridge relying on a single nondestructive evaluation (NDE) technology provide only limited information about condition of reinforced concrete elements. To overcome limitations of individual technologies, a complementary approach using several NDE technologies should be used in RC evaluation.

The presented approach utilizes a suite of NDE technologies, namely: impact echo (IE), ultrasonic echo (UE), ultrasonic surface waves (USW), ground penetrating radar (GPR), half-cell potential and electrical resistivity. The application of listed technologies is illustrated on material characterization and defect/deterioration detection of RC bridge decks. The NDE evaluation was complemented by "ground-truth" measurement on the cores extracted from nine bridge decks.

The condition assessment using the six NDE technologies has clearly shown their advantages and limitations. For example, the GPR survey provides assessment of concrete deterioration at relatively high speeds of data collection. On the other hand, the IE and UE provide high accuracy in detection and characterization of delaminations in the deck, but at a lower testing speed. Half-cell potential test provides accurate assessment of likelihood of corrosion, while the USW test provided accurate assessment of the effects of deterioration processes and defects on mechanical properties, primarily the elastic modulus degradation. Most importantly, the surveys have shown advantages of the use of multi-modal NDE surveys in the comprehensiveness of condition assessment of concrete bridge decks and RC elements in general.

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Performance indicators for highway bridges and their integration in bridge management

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ABSTRACT

Performance indicators are key and a crucial decision tool. The uncertainties in the available information for computing the performance indicators and their changes over time impose a great challenge in the decision making process.

The assessment of reliability of highway bridge structures based on component reliability leads to either over-conservatism in evaluation of redundant structural systems or to overestimation of the actual load carrying capacity of non-redundant structural systems (Hendawi and Frangopol 1994). However, the performance of a highway bridge structure can be evaluated by assessment of its structural system reliability (Estes and Frangopol 1999).

To compute the system reliability, Estes and Frangopol (1999) and Akgül and Frangopol (2004a,b) used AASHTO guidelines to formulate limit state functions for the components of various bridge system against different failure modes. The system reliability was then computed by establishing a series-parallel model for an assumed failure scenario for the structural systems. The shortcoming in this approach is clearly the need to assume a model for system failure.

Incremental nonlinear finite element analyses (INL-FEA) has recently been used in the computation of system reliability of bridge structures. Using INL-FEA accurately allows for the prediction of the resistance of the entire system. In fact using this approach, serviceability as well as ultimate limit states can be considered accordingly.

An interaction among several computer softwares is an integral part of the success of the process. Figure 1 shows a diagram that illustrates this interaction. The connectors in the diagram show the directions of interaction among the software programs (ABAQUS, MATLAB, VisualDOC, RELSYS, and RELTSYS) and the numbers show the steps that each interaction is established to achieve. Some steps are in fact performed on a single program. These steps are: (1) build the finite element model, (2) generate the response envelope,

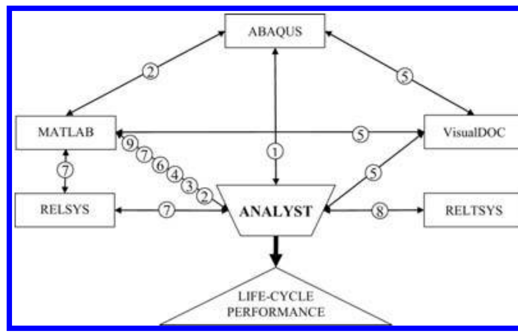

Figure 1. Interaction among programs and the analyst for computing the life-cycle performance.

(3) forecast the average daily traffic, (4) perform statistics of extremes for finding the live load multiplier, (5) perform the response surface analysis, (6) perform the Latin hypercube sampling for resistance computations, (7) compute the point-in-time reliability, (8) compute the cumulative-time failure probability, and (9) compute the life-cycle performance.

In this paper, various life-cycle performance indicators are reviewed, and their integration in bridge management is discussed.

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Parameter identification of a reinforced concrete T-beam bridge

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ABSTRACT

Parameter identification based on model updating is one of the most important links in Structural Identification. The benefits of combining finite element (FE) analysis with on-site measurement through model updating are significant and growing as more reliable experimental, modeling and model correlation approaches become available. The authors have successfully applied Structural Identification to numerous bridges over the past decade. However, manual model updating of structural parameters has significantly limited the application of model updating on large infrastructures. The methodology proposed within aims to use the Application Programming Interface (API) function in the Strand7 FE software package to automatically update selected parameters using Matlab. The proposed methodology was applied to a three span, simply supported T-beam bridge. Both the elastic modulus for the global structure and the crack height of the primary girders were utilized as updateable parameters in two separate cases. The results show that the average crack height parameter reproduced the static measurements with a high degree of accuracy and is a reasonable parameter choice for this class of structure.

MS12: Computational prediction & in field validation of bridge performance

Organizers: F. Biondini, F. Bontempi & P.G. Malerba

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Probabilistic performance prediction modeling for bridges considering maintenance effects within a combined computation, visualization and programming environment

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ABSTRACT

Various models have been developed in recent past for lifetime performance prediction of bridges. Studies for developing new theoretical models and improvement of the existing ones are part of ongoing research in bridge deterioration and management. However, improvement of such theoretical models so that they can be practically implemented in bridge management systems is a challenging task. In order to achieve such task, newly developed models must be tested and further improved to achieve practicality.

Lifetime performance prediction for bridges can be performed using either a safety or condition criteria. In a well designed bridge management system, it is well established that both of these criteria should be implemented and monitored at the same time. The Markovian decision process-based models, linear or polynomial curve-based techniques and profile-based probabilistic methods are the methods to conduct condition or safety-based performance prediction and maintenance optimization. Linear or curve-based techniques use linear or polynomial functions to predict lifetime deterioration based on the deterioration rate and time as the parameters. Profile-based robabilistic methods are based on formulas defining the profiles and effects of maintenance actions which contain random variables having probability distributions.In this study, an investigation and improvement of an existing piecewise linear lifetime condition and safety prediction model is performed.

A bridge performance prediction model developed by researchers in this field has been implemented in Matlab environment for further improvement. Computation, visualization and programming tasks are performed as part of this application specific solution using the toolboxes. For the simulation of the random variables, Latin Hypercube Sampling technique is used which is available in the computational environment. Random variables may be assigned normal distributions. A function is also developed to generate samplings from other distributions such as triangular distribution. Instead of using superposition of the effects of maintenance actions on no-maintenance profiles, the newly developed algorithm consists of a simulation loop which contains a time loop embedded in it. Such an algorithm enables the generation of the performance profile over the whole lifetime at once for a single simulation of all random variables involved. This approach may prove to be a faster algorithm than the one previously developed. Mean value and standard deviation profiles for the condition and safety indices as well as the probability density distributions of these indices over the bridge lifetime are graphically generated using the integrated modeling and simulation environment.

An overview of the condition prediction or deterioration models, ranging from the simplest linear model to more complex probabilistic piecewise linear model based on simulation, is also presented. Using the developed environment, effect of various maintenance-repair-replacement scenarios on condition and safety profiles of bridges is investigated. The computational platform may further be developed into a deterioration model analysis tool for infrastructure systems such that the desired model type, whether linear, polynomial, Markovian or piecewise-linear probabilistic, can also be selected from a list of deterioration model options.

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Structural maintenance and rehabilitation of a 1900's iron bridge

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ABSTRACT

The "Tre Martiri" iron bridge, across the Oglio river near the city of Cremona (in the north of Italy), was built by the National Society "Savigliano" and opened in 1900 as a road bridge. The original project, drawn up in 1899, was based on the three-moment equation proposed some years before by Clapeyron.

The bridge structure is constituted by a continuous beam supported by six metallic piers between two masonry abutments. The two end spans are 8.10 m long, while the five central spans are 18.0 m long, for a total length of 106.20 m. The bridge is 5 m wide from curb to curb. Its horizontal bearing structure is constituted by two main continuous iron trusses, with parallel chords, braced with St. Andrew's cross bracing system. The floor system consists of iron floor-beams and stringers, with St. Andrew's cross bracing system. The bridge deck has been realized by using iron Zorès profiles, with a concrete topping characterized by a variable thickness (around 50 mm at lateral sides and 100 mm in the central part).

Even if the existing structure is basically the same originally built, the bridge has undergone considerable repairs in 1946–1947, since it was seriously damaged during the 2nd World War. These restoration works, carried out by the Ansaldo Company of Genoa, interested different parts of the bridge and particularly its deck, iron trusses and foundations. Nowadays, after about one century from its construction, the bridge shows signs of major deterioration, mainly due to weather and environmental factors as well as to an inadequate routine maintenance.

The principal factor leading to deterioration of structural members and their joints is surface corrosion, which has caused a uniform destruction of relatively large surface of iron, so determining the reduction of cross-sections in the structural elements. Damages due to corrosion are mainly located in the bottom chord of the iron truss, near the abutments, where the vegetation growth causes insufficient ventilation and dewatering. Material losses are mainly localized in the horizontal flange of iron angles, where moisture stagnation is more significant. Other signs of deterioration can be observed in the diagonal elements of the lateral bracing system, which have been realized by the coupling of two angle profiles through the riveting of their flanges. Water seepage and stagnation between the angles have caused the formation of a variable thickness layer of rust, with consequent swelling of the iron elements.

After the survey of the main factors leading to bridge deterioration, the bridge structure has been critically analyzed in its actual state and a numerical model has been performed so to reproduce the theoretical state of stress and verify the ULS conditions of the bridge bearing elements.

Some hypotheses for a restoration project have then been proposed and discussed. Even if the most deteriorated elements should be removed and substituted by new ones, repairs should be done with sensitivity, so that the historic character of the bridge and its aesthetic aspect are not jeopardized in the process. Consequently, the selection of proper materials and structural solutions in the bridge rehabilitation is crucial in order to minimize the visual impact of restoration works and provide at the same time an adequate safety level.

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Credibility indicator for bridge service life prediction

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ABSTRACT

The assessment of the life-cycle performance of deteriorating bridge structures can be formulated as a reliability problem where a significant loss of performance results in failure. Therefore, when a failure is reached, the system passes from the current state into another state characterized by a lower level of performance. On the other hand, structural performance can also be improved by maintenance and/or rehabilitation interventions. In this case the system may move from the current state into another state characterized by a higher level of performance. In both cases the process may be defined as a transition process through different states. Since the problem is affected by several sources of uncertainty, the assessment of the life-cycle performance must be based on a suitable modeling of damage and maintenance process (Biondini et al. 2008).

The probabilistic modeling of the deterioration process is affected by uncertainty. In this paper the influence of the epistemic uncertainty on the service life prediction of deteriorating structures is investigated by means of a relative criterion of validation introduced by Grandori *et al*. (1998, 2003). This criterion is based on a credibility index able to compare two models and decide which one is the most reliable (Grandori *et al.* 1998, 2003, Guagenti *et al.* 2003).

The effectiveness of this approach is shown with reference to an existing steel bridge. Two probabilistic distributions, Gamma and Exponential, are put in competition in the modeling of the failure times, and their credibility evaluated (Fig. 1). The failure processes are simulated using Monte Carlo techniques and the credibility index is built on the basis of a large set of samples, with different population size.

The obtained results show that lifetime predictions can significantly depend on the probabilistic modeling, and prove that the credibility index is able to recognize, in a qualitative and quantitative way, the most reliable modeling.

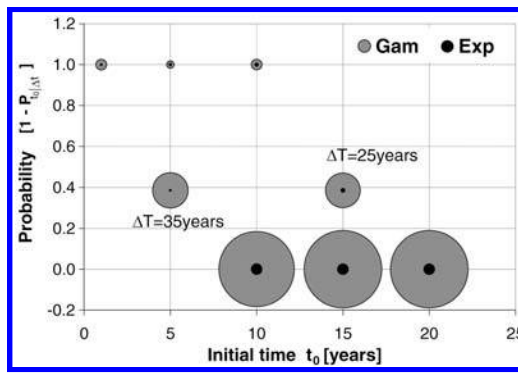

Figure 1. Comparison between the credibility of two distributions in competition in the modeling of the failure times. The radius of each circle is proportional to the value of the credibility index.

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Structural lifetime and elapsed time between first failure and collapse: Application to an arch bridge

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ABSTRACT

The structural performance of concrete bridges exposed to the effects of environmental aggressiveness is time-variant. In a life-cycle oriented approach to bridge design it is therefore necessary to evaluate the structural performance not only at the initial time of construction, when the bridge is intact, but also during the expected lifetime, by taking into account the effects of damage under uncertainty (Frangopol *et al.* 1997, Biondini *et al.* 2004). This allows to assess the residual lifetime under prescribed reliability levels for each failure mode of interest, or to plan proper maintenance and repair interventions to achieve the prescribed value of structural lifetime.

Structural failures can develop at local level, for example with the formation of a plastic hinge, or at global level, as a consequence of the activation of a set of plastic hinges leading to structural collapse. For bridge systems the identification of the local failure modes and of their occurrence in time can represent a crucial information to maintain a suitable level of performance and to avoid collapse over the structural lifetime (Mori and Ellingwood 1994). In fact, repairable local failures can be considered as a warning of possible occurrence of more severe and not repairable failures. Therefore, the time interval between the first local failure and structural collapse, or the elapsed time between these failures, can be considered as a measure of the required rapidity of the system to be repaired after first failure and represents an important performance indicator in a life-cycle design approach.

It is worth noting that the tolerance to local failures can also be measured in terms of structural redundancy (Frangopol and Curley 1987). However, even though they are related concepts, elapsed time between failures and structural redundancy refer to different system resources. In fact, structural redundancy denotes the ability of the system to redistribute the load after the occurrence of a local failure and does not provide a measure of the failure rate, which depends on the damage scenario and damage propagation mechanism. However, under this perspective the elapsed time between failures can also be considered as a measure of system redundancy in terms of rapidity of evacuation.

The prediction of structural lifetime and elapsed time between local failures and collapse of concrete bridges is investigated in this paper by means of a general approach to probabilistic assessment of concrete structures under diffusive attacks from aggressive agents (Biondini *et al.* 2004, 2006). The proposed approach is applied to an arch bridge considering different levels of uncertainty (Biondini and Frangopol 2008, 2010). The results show that the uncertainty on structural behavior and damage process may significantly affect the prediction of structural lifetimes and of the elapsed time between failures. Moreover, the expected elapsed time between failures, and then the required rapidity of a system to be repaired, decreases with increasing the level of uncertainty.

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Role of uncertainties on time dependent behaviour of prestressed and cable stayed concrete bridges

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ABSTRACT

The behavior of pre-stressed and of cable stayed concrete bridges may be strongly affected by the time dependent response due to creep and shrinkage of concrete. Same experiences showed us that, even after accurate design assessments, some unsatisfactory service behavior can arise, due to some level of uncertainty in the reference data. In this paper the role of the uncertainties has been investigated.

As sources of uncertainty, the pretensioning forces in the cables, the relative humidity and the concrete strength have been considered. For a given set of data, the creep effects has been handled by means of the A.A.E.M. Method. The effects of the uncertainties have been simulated through a Monte Carlo approach. Two topical cases have been studied: one less sensitive to uncertain data, the other one strongly affected by them.

In the case of a cable stayed bridge, made of a concrete deck, subjected to creep effects and suspended to a set of pretensioned cables, the role of uncertainties in the pretensioning forces, in the relative humidity and in the concrete strength do not influence the time dependent behavior sensibly (Figure 1). The standard deviation with respect to the mean tension in the cables is relatively small and, most of all, it progressively reduces. The system seems to be self stabilizing over time. On the contrary, in the case of a prestressed cantilever beam, the effects of uncertainties in the pretensioning forces and in the concrete strength cause a relevant variance of the tip deflections (Figure 2). Both deflections and their variance increase with time and they may strongly modify the vertical attitude of the structure. These effects are emphasized when the prestressing is applied few days after curing.

The achieved results and, in particular, those concerning the prestressed cantilever beams, outline the limits of the traditional deterministic analyses.

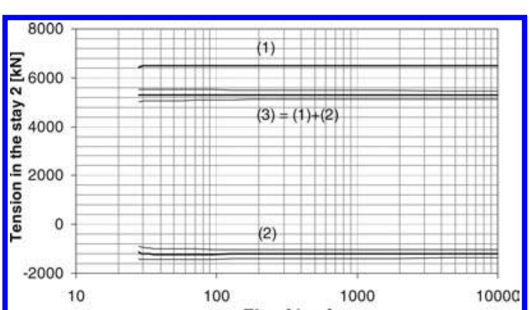

Figure 1. Effects of uncertainties in the initial pretensioning of the stays. Tension variation in a stay over time. (1) self-weight only; (2) pretensioning only; (3) = (1) + (2): mean value μ (thick line) and standard deviation σ from the mean μ (thin lines).

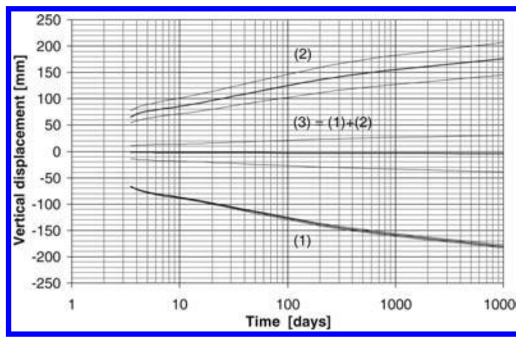

Figure 2. Prestressing time $t_0 = 3.5$ days. Vertical displacements at the tip of the cantilever over time. (1) dead loads only; (2) prestressing only; (3) = (1) + (2).

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Variability of measured modal frequencies of a suspension bridge under actual environmental effects

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ABSTRACT

This paper addresses the quantification of normal modal variability of a suspension bridge making use of long-term monitoring data. The output-only modal frequency identification of the bridge is effectively carried out using the Iterative Windowed Curve-fitting Method (IWCM) in the frequency-domain. The seasonal temperature-frequency correlation is formulated using multi-sample averaging technique for each vibration mode to eliminate the random variations rising from the identification algorithm. Then the effect of temperature on the measured modal frequencies is eliminated and the traffic-induced and wind-induced modal variability are quantitatively evaluated, respectively. The analysis results reveal that:

(1) From the identified frequency sequences in a typical day, it can be observed that the measured frequencies present instantaneous changes because of the nonstationary properties of the ambient loadings. In order to eliminate the random variations rising from the identification algorithm, the daily averaged frequencies using multi-sample averaging technique were obtained, i.e., the seasonal correlation analysis of frequency-temperature. The maximum and averaged relative variations of modal frequencies reduced from 3.013% and 2.633% measured at 10-min intervals to 2.168% and 1.268% using multi-sample averaging technique.
(2) Temperature is the critical source causing modal variability, and there is an overall decrease in modal frequency with temperature for all the identified modes. A 6-order polynomial regression model is further applied for the modeling of daily averaged frequency and temperature. It is found that the maximum and averaged relative variations of frequency induced by temperature are 1.975% and 1.175% for the variation of temperature covering a full cycle of in-service conditions.
(3) The influence of traffic loadings on modal frequencies is not significant as the temperature. The modal frequencies are observed to slightly decrease with the increase of acceleration RMS. it is found using the linear regression models of acceleration RMS-frequency that the maximum and averaged relative variations of frequency induced by traffic are 0.262% and 0.104%. On the whole, the correlation between modal frequency and traffic loadings is very week and of no significance.
(4) The influence of wind speed on modal frequencies is also not significant. The modal frequencies are observed to slightly increase with the increase of wind speed. It is found using the linear regression models of wind speed-frequency that the maximum and averaged relative variations of frequency induced by wind speed are 0.215% and 0.163%. On the whole, the correlation between modal frequency and traffic loadings is very week and of no significance.
(5) The modal variability caused by wind speed and traffic loadings is notably less than by temperature and identification algorithm. And the influence of identification algorithm on modal frequencies is as significant as the temperature effect. Thus, for reliable performance of vibration-based damage detection methods, the improvement of the modal frequency identification algorithm properly considering the nonstationary properties of the ambient loadings is very important.

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Fuzzy reliability assessment of bridge piers in presence of scouring

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ABSTRACT

The design of bridge piers placed in the riverbed has to deal with variable hydraulic forces acting on structures with foundations which may be prone to scour effects (Da Deppo 1994, Ballio 1999). The conventional design of drag forces, scour depths and foundation bearing capacity is carried out by means of equations which involve a certain number of uncertainties regarding flow, sediments, structural and geotechnical parameters. A sound reliability assessment of bridges piers against scouring risks must take into account the uncertainties inherent in these parameters.

Due to the phenomenological nature of these uncertainties, a fuzzy approach seems to be the most adequate to deal with this kind of problems (Dordoni 1999). In the first part of this study the role of the uncertain parameters is assessed by considering separately their effects on the drag forces and on the general and local scour evaluation. In the second part, the previously obtained responses regarding the drag forces and the scouring depths are used as input data for a fuzzy analysis of the pier foundation reliability.

Different design contexts are studied: embedded foundations and foundations with exposed piles; effects of clear water and live bed scour conditions; different ratios between vertical superimposed and lateral hydraulic loads.

A sensitivity analysis allows to estimate the influence of the various uncertain variables on the partial and on the global responses.

The defuzzified output, expressed in terms of centre of mass of output membership functions, assumes values that are noticeably different and more severe than the deterministic results. The results of the cases studied (pier shaft exposed and foundation piles exposed) are summarized in Table 1.

Table 1. Percentage difference between the deterministic analysis and the result of the fuzzy assessment.

	Pier shaft exposed	Foundation piles exposed
Hydraulic force	+19%	+46%
Scour depth	+9%	+13%

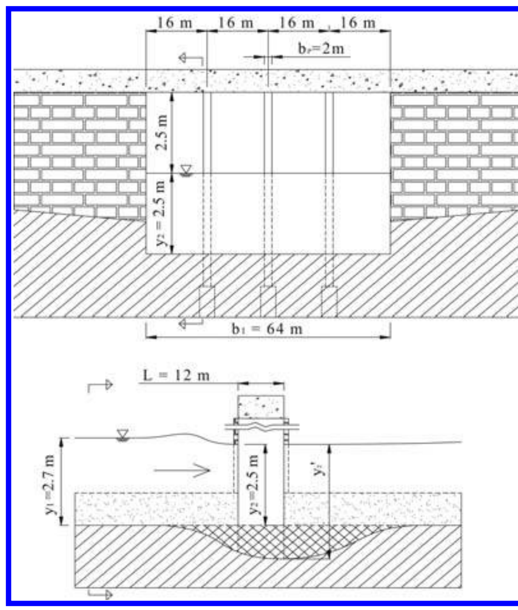

Figure 1. Schematic representation of the restriction and piers positioning: front view (upper panel) and section of the bridge.

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Non linear finite element analysis of a 50 years old reinforced concrete bridge: Comparison with experimental data and sensitivity analysis

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ABSTRACT

In this paper, a non linear finite element analysis of a full scale reinforced concrete bridge tested in Ovik (Sweden) is presented. In order to quantify the load carrying capacity of the bridge, a collapse load test combining bending and shearing load effects was performed within European Union founded project "Sustainable Bridges – Assessment for Future Traffic Demands and Longer Lives".

The first objective of the numerical simulations is to evaluate the robustness and the relevancy of the different constitutive laws (concrete and steel/concrete interface) to model large-scale structures subjected to complex mechanical loadings coupling bending and shear up to failure. By using accurate damage mechanics based models, satisfying qualitative and quantitative results have been obtained. Figure 1 provides a quantitative comparison between the numerical results and the experimental ones. It leads to prove that the proposed finite element model of the reinforced concrete bridge is relevant.

A satisfying damage pattern was also obtained (figure 2). In particular, the cracks appear in shear areas, in concordance with experimental results.

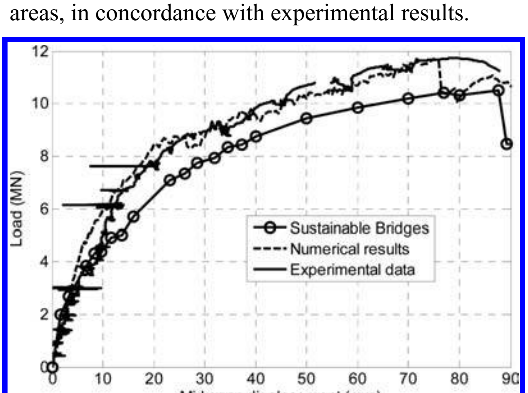

Figure 1. Comparison between the experimental load/mid span displacement and numerical results (proposed analysis and Sustainable Bridges numerical results).

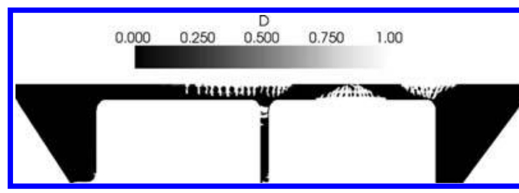

Figure 2. Damage pattern obtained numerically for a 100 mm mid-span displacement.

A sensitivity analysis was also performed to quantify the influence of local degradations on the global mechanical response of the bridge. To achieve this objective, a new steel/concrete interface constitutive model has been used to represent corrosion.

Two aspects were investigated. The first one concerns the effects of steel cross sections reductions and the second one is related to the effect of bond strength variations. Due to the absence of corrosion in the bridge reinforcement, results can only be corroborated with literature information: it comes out that it is nevertheless in good agreement with available results.

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Field test and analysis of the dynamic factors for bridge in urban railway transportation system

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ABSTRACT

Linear Induction Motor (LIM) system is a new urban transportation system between maglev and traditional rail transportation. In China, many cities have adopted the Linear Induction Motor (LIM) rail transit system, but specialized codes for design or assessment of this type of vehicle loading in the urban railway transportation system have not yet been established. In this paper the dynamic factor formulations for railway bridges adopted in various countries are reviewed. Then, a typical bridge on the Guangzhou metro line 4 is evaluated both experimentally and theoretically to determine vehicle-bridge coupling vibration characteristics. The vehicle is represented as a whole vehicle model of secondary suspension with 6 degrees of freedom, and the bridge is modeled by beam elements. The coupled equation is formulated using the principle of total potential energy with stationary value in an elastic system and solved by using Newmark-β method. Field dynamical tests were also performed on the bridge. The calculated and experimental displacement time-histories for LIM trains crossing the bridge were obtained and dynamic factors were developed. Through the theoretical and experimental analysis for vehicle-bridge coupling vibration, a formula for the dynamic factor that can be used in bridge design and assessment is proposed.

On the basis of analysis of electromagnetic force (EF), the equation of LIM vehicle-bridge interaction is derived in terms of the principle of total potential energy with a stationary value in elastic system dynamics. A special dynamic analysis program based on the MATLAB language of LIM vehicle-bridge interaction was developed.

Because the LIM train was less axis weight, lower moving speed etc, the DFs for design formula referenced railway codes or highway codes are shown to be conservative. Adopting the proposed formula of dynamic factor may be more reasonable for urban rail transit bridge design. The proposed DF formula can be used in the development of bridge design criteria for urban rail transit.

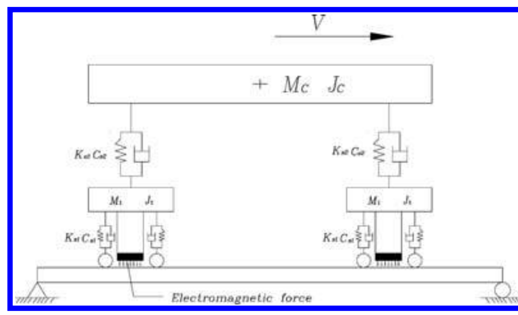

Figure 1. LIM vehicle-bridge vertical coupling vibration model.

Table 1. Comparison of theoretical and experimental dynamic factors.

Velocity (km/h)	Theoretical values		Experimental values
	Without EF	With EF	
40	1.046	1.052	–
50	1.058	1.068	–
60	1.072	1.087	–
70	1.068	1.075	1.045–1.10
80	1.066	1.073	

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A parametric study on the rocking behavior of bridge columns with spread footing foundation

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ABSTRACT

Some spread footings of real engineering practices of bridges in Taiwan were found to be uneconomically large due to the restriction of foundation uplift regulated in design code. Although the rocking mode of spread footings induced from foundation uplift is not acceptable in general, some researches have indicated that rocking itself can act as a form of isolation mechanism. Uplift of foundation can limit the earthquake forces transmitted to the column base, hence to decrease the plastic deformation occurring at the plastic zone. Besides, unless very massive footing, some uplift on the tension edge of spread footing during a major earthquake can not be avoided. Thus, the analysis based on the assumption that the foundation and soil are firmly bonded would not only lead to unreasonable large internal forces in columns, sometimes it would also underestimate the disadvantage that may be brought by rocking, such as large lateral displacements at deck and permanent settlement in soils. In this regards, realistically consider the effects of the interaction between the foundation, column and soil becomes an important issue. In order to gain better understanding of rocking mechanism and consequently to have more confidence in taking its benefit into consideration, an analytical model which can consider the nonlinear interaction between column, foundation and the underlying soil was established.

The analytical model established using Sap2000N is schematically shown in figure 1. In which, column at the plastic hinge zone was idealized by a 3D fiber beam-column element, while the other parts of the column were modeled by elastic beam elements. A lumped mass and point load were also added at the top of the column to simulate the tributary mass and the weight of superstructure. The foundation was modeled by rigid bars with a lumped mass at the center of gravity. Figure 1 also shows that the vertical bearing resistance of foundation was modeled by an elastic-perfectly plastic spring with its yield points equal to the ultimate bearing capacity of the underlying soils. Also, this zero-length nonlinear spring can not take the

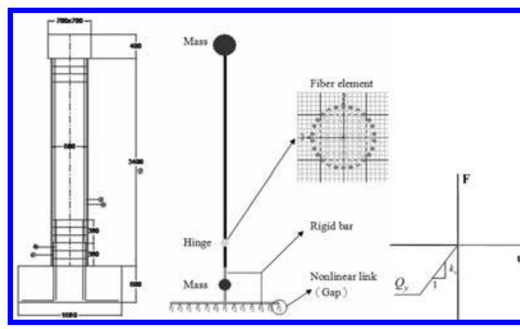

Figure 1. Analytical model.

tension force and thus can take the uplift of footing into account automatically.

Using this analytical model, an extensive parametric study was performed and the influence of foundation size, soil stiffness and soil ultimate bearing capacity on rocking behavior was calculated. From this analytical results, the isolation effect of rocking spread foundation was identified. It also shows that as the footing size and the underlying soil stiffness decrease, a better performance at the column's base can be obtained. However, it also results in an increase in displacement response at the deck's level. On the other hand, when the yielding of the underlying soil is allowed to occur, the decrease in soil ultimate bearing capacity can lead to minor plastic deformation at column base. However, with the yielding of the soils, the level of permanent settlement in underlying soil increases, too.

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Tests to collapse of masonry arch bridges simulated by means of FEM

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ABSTRACT

Modelling and analysis of masonry arch bridges is still a current and complicated topic. No effective or reliable method was developed so far which could reasonably and accurately simulate behaviour of real structures under the collapse loading. The reason for this deficiency is an exceptional complexity of that kind of structures. An alternative approach is proposed in the paper which locally simplifies structure of the bridge but provides satisfying representation of its global behaviour. Characteristic feature of the method is application of the mezo-modelling technique, being between micro- and macro-modelling, in discretization of the arches barrel. The arch is divided into even segments which represent parts of masonry consisting of several bricks or stones separated by individual mortar joints. Details of the model are shown in Figure 1.

The proposed model was applied to simulations of tests to collapse of several full scale single-span masonry arch bridges: 12 laboratory models erected for the purpose of the testing and 5 real bridges excluded from exploitation. One of the tested bridges (*Prestwood*) during the test is presented in Figure 2.

The limit state of all the analyzed structures was reached very easily during the calculations. An example of the results in form of deformed mesh of a selected structure at the limit state with map of plastic strains is shown in Figure 3. In the picture plastic hinges (marked with lighter colour) indicate the four-hinge failure mechanism possible to be simulated by the model.

In all cases the calculated limit load value, mode of failure as well as the location of the plastic hinges were in a good agreement with the corresponding results obtained from the experiments.

Apart from the satisfying results several other advantages of the approach can be found, like: limitation of the numerical problem size, simplification of the FE mesh or effectiveness and ease in reaching the limit state including development of the localized cracking of material which is a common and serious difficulty in case of macro-models.

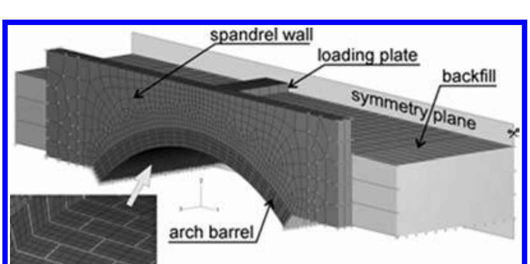

Figure 1. 3D FE model of a masonry arch bridge.

Figure 2. *Prestwood* bridge at the end of testing (Page 1987).

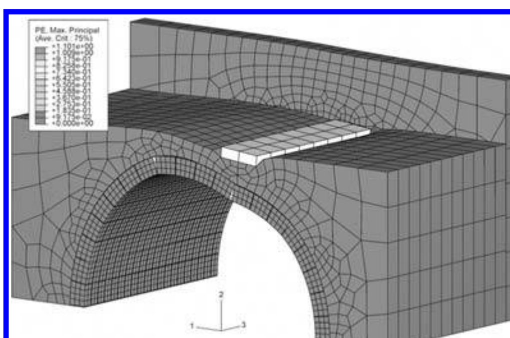

Figure 3. Deformed mesh of 3D model for *Dundee* bridge.

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Dynamic evaluation of bridge mounted sign structures

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ABSTRACT

The design requirement for ground mounted sign structures are fairly well defined in the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Signals and consists of applying an equivalent pseudo-dynamic loading to account for the dynamic effects of wind loads and ignores the dynamic effect due to moving vehicle loads. This design approach, however, should not be applied to the design of bridge mounted sign structures because ignoring the dynamic effects of the moving vehicle loads may produce non-conservative results, since the stiffness of the bridge structure can greatly influence the behavior. Not enough information is available in the literatures which provide guide lines to include the influence of moving vehicles in the design of the bridge mounted sign structures. This paper describes a theoretical methodology that can be utilized to account for the dynamic effect of moving vehicles. A case study is also included where this methodology was successfully applied.

In 2007, a Closed Circuit Television Camera (CCTV) support system mounted on the superstructure of GOWANUS Express way in New York City failed after 9 years of service. The superstructure of GOWANUS Expressway consisted of steel pier bent with cantilevered cap-beams, steel stringers and 8 inch concrete slab. The CCTV support attached to the railing consists of aluminum pole with cantilever arm for the camera support at the top. This support system was subjected to the dynamic effects due to wind loads as well as moving vehicle loads on the bridge. In order to evaluate this behavior, a set of differential equations of motion for the interaction between the bridge structures, moving vehicles and camera support system were derived. Unlike conventional equation of motions, the coefficients of equation, in this instance, need to be variable depending on the location and speed of the moving vehicles. In order to solve the equation of interaction, Bridge-vehicle interaction finite elements were developed using Modified Newmark's beta method. Using this element, full scale 3D model of typical span of Gowanus Expressway with moving vehicle was then developed.

The amplification factor of static loading and fatigue analyses for two dynamic loadings generated from moving vehicle and wind loads were studied using this model. Dynamic wind load were generated using AASHTO Specifications noted earlier. This procedure allowed meaningful computation of Fatigue lives of each component of CCTV support structure. It was concluded that the bridge-vehicle interaction finite element developed can provide a more accurate representation of the behavior of bridge mounted sign structures.

The result of these analysis enabled development of simple and effective retrofitting scheme for the existing CCTV support system

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A special kind of analysis: CFD modeling for design and assessment of bridge passive control devices

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ABSTRACT

During its life-cycle a structural system is generally subject to both static and dynamic loads: for the second type of actions inertial effects become more important resulting in dynamic amplification and cyclic response.

The effects of such loads can be controlled for example by adopting structural protective systems such as seismic isolation, semi-active and active controls and passive energy dissipation: the latter type can be effective against seismic and wind induced motion (*Harris* & *Crede*, 1976) requiring no kind of external supply.

Among the different types of passive energy dissipation systems, viscous fluid damper with annular orifice is deeply investigated in this work with the aim to set up a reliable numerical model supporting the design phase.

The operating principle (Fig. 1) consists of a viscous fluid, filling both chambers inside a cylinder whose volume is defined by a one- or two-rod piston, that is forced, as the piston oscillates, to move back and forth through the gap between the piston head and the cylinder pot: this causes high energy dissipation density due to the frictional stress produced by the viscous fluid (*Soong* & *Dargush*, 1997).

In the first part of the work a brief description of the fluid-dynamics is provided from an analytical point of view: the basic equations are solved considering an highly idealized situation in order to provide a reference solution; both Newtonian and Non-Newtonian incompressible viscous fluid are considered (*Wilkinson*, 1960).

In the second part finite element modeling of the problem is carried out: initially the idealized analytical model has been reproduced by means of ANSYS code considering only the central portion of the annular conduit; subsequently an improved model is built which includes a portion of the lateral chamber near the orifice outlet: influence of boundary effects and of Non-Newtonian behavior on the velocity distribution across the orifice is investigated by means of both ANSYS and ADINA codes.

Comparison of numerical and analytical results in term of velocity and viscosity profiles across the orifice is carried out. Velocity vector plots at the orifice-chamber interface obtained with both finite element models are compared and discussed.

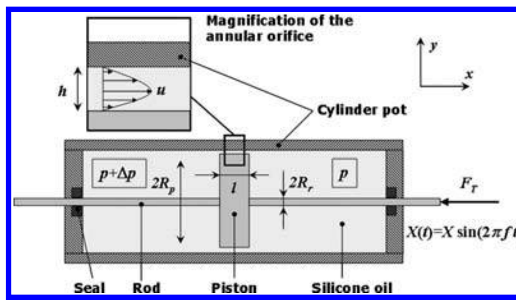

Figure 1. Sketch of a viscous fluid damper with annular orifice.

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Residual service life of existing railway bridges

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ABSTRACT

The residual service life of existing railway bridges is analyzed and assessed in this paper. Fatigue and damage assessment are considered to be the most critical topic issues in order to estimate the residual life of these types of structures. In fact, with the aging of existing steel bridges and the accumulated stress cycles under traffic loads, assessment of remaining fatigue life for continuing service has become more important than ever, especially for decisions on structure replacement, or retrofit interventions. Experience from engineering practice indicates that fatigue analysis based on code provisions often underestimates the remaining fatigue life of existing bridges due to overestimating live load stress ranges, and, at the same time, underestimating the stress history. In order to bridge the gap between common engineering practice assessment of existing bridges and code provisions, this analysis has been performed along with traffic prevision.

Railway bridges represent a strategic part of an ancient network and, in several cases, they have already reached their traffic capacity limit. In this context, bridge condition state assessment and consequently maintenance/replacement operations become more and more necessary. The average age of sixty percent of Italian railway steel bridges is about one hundred years as they were built between 1900 and 1920. In order to bridge the gap of a minor attention on historical metal bridges, a deep analysis consisting in a numerical study, based on previous experimental investigations (Pipinato et al. 2009a, 2009b), has been carried out at the University of Padova, in collaboration with the railway national authority, RFI-Rete Ferroviaria Italiana: non-linear Finite Element analyses have been carried out, giving some key issues on the fatigue assessment of railway bridges. In particular, according to the experimental findings of Pipinato et al (2009a, 2009b) related to flexural and shear behaviour of some critical fatigue riveted details, a Finite Element (FE) analysis, taking into account material non linearities due to plasticization of materials and contact phenomena between the rivets and the plate and the plates themselves, is developed to obtain some indications on the failure mode of the critical fatigue details and accurate stress variations for the estimation of the residual fatigue life.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Curved-tendon and fiber beam-column element for analysis of cable-stayed PSC bridges

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ABSTRACT

The objective of this study is to develop time-dependent fiber beam-column element and tendon element, which are capable of predicting the long-term behavior of cable-stayed PSC(prestressed concrete) bridges. For this purposes, analysis models for finite element method were developed based on the flexibility based fiber beam-column model originally proposed by Spacone et al. (1996). For the PSC girders with pre-stressing tendon, additional curved time-dependent tendon element cooperates with fiber element. The performance of proposed analysis models is evaluated by comparing with the experimental results in the literature. Finally, program simulated the construction of the Pasco-Kennewick intercity bridge in the state of Washington.

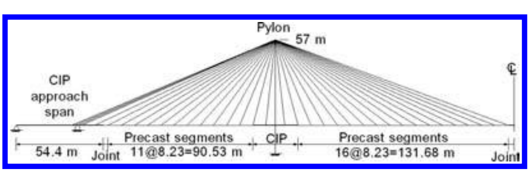

Figure 1. Half elevation of Pasco-Kennewick inter-city bridge.

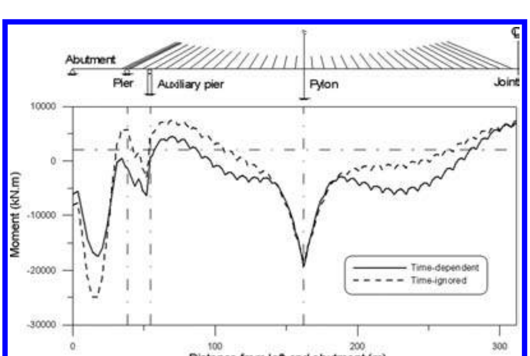

Figure 2. Moment of deck at final construction phase.

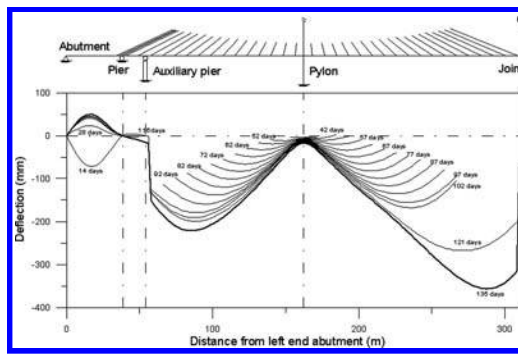

Figure 3. Deflection of deck at various construction phase.

Khalil et al. (1983) simulate this bridge with time-dependent analysis but material was considered in elastic range.

Most of bridge dimensions and material properties are borrows from that simulation. An attempt was made to adhere to same concrete dimensions, cable properties, prestressing information, and construction phase with actual bridge, but some assumption regarding time schedule, concrete age, and initial tension in cables were made.

The largest difference between time-dependent analysis and mechanical analysis was about 8,300 $kN \cdot m$, 4,100 $kN \cdot m$, and 4,760 $kN \cdot m$ at the approach span, side span, and main span, respectively.

The simulation result of cable stayed bridge may be helpful in determining the required camber or initial tension in the cables during each construction phase in order to achieve a desirable profile at final phase.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Remaining structural capacity of an early post-tensioned bridge verified after removal

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ABSTRACT

In southern Switzerland a post-tensioned concrete bridge (Fig. 1) built in 1952 showed signs of improper grouting of the ducts and subsequent corrosion of the prestressing steels. That is why a monitoring system was installed to detect wire breaks based on acoustic emission analysis. The monitoring revealed a wire break about every 35 days and the owner decided to prop the bridge by additional piers and to replace it by a new one. The demolition of the bridge was used to establish a detailed condition survey of both, the concrete and the reinforcement, especially the tendons. Like this, excellent data is available on the actual condition of the bridge prior to removal. This information was used to calculate the structural capacity of the bridge (Fig. 2) and possible failure scenarios depending on applied traffic loads and ongoing deterioration.

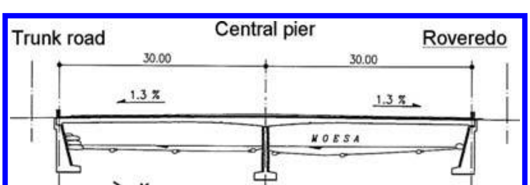

Figure 1. Longitudinal section of the bridge.

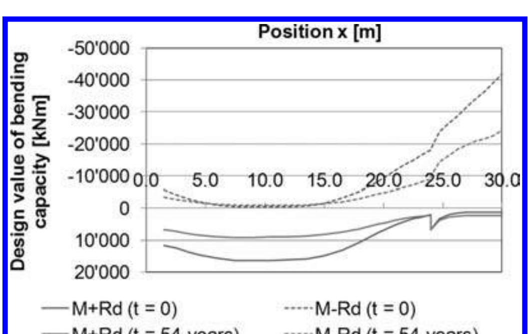

Figure 2. Bending capacity (left span only).

Applying the lower bound theorem of plasticity requires identifying an admissible stress state. For haunched girders these calculations become quite complicated.

Applying the upper bound theorem of plasticity means identifying the relevant failure mechanisms. Changing the values of loads i.e. external work and resistances i.e. dissipation is an easy task and the same applies for extrapolations into the future.

The calculations have shown that the partial safety concept is not adequate for this kind of questions, because design values have no physical meaning. For material properties and resistance models, characteristic values may be applied, but for permanent and traffic loads expected values for the considered scenario should be used. To account for the probabilistic aspect of structural safety, it is more feasible to consider the frequency of an expected load scenario rather than the probabilistic distribution of the action force.

Monitoring the change of condition parameters like detecting wire breaks by acoustic emission analysis is a good tool for early warnings. Since normally monitoring is installed in an advanced aging stage of the structure, the initial condition is not known and integrating measured condition changes does not lead to a reliable result.

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An analysis of long time deflection of long span prestressed concrete bridges

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ABSTRACT

The results of the presented solutions and the developed analytical and design methods will help creating the sufficient theoretical tools for reliable and economic structural design bridges without deflection impairments. The paper reviews the causes of underestimation of the long-time future deflections. Special attention is directed to prestressing – a procedure to find the optimal arrangement of tendons layout is presented, allowing avoiding the tendons contributing to deflection increases. Developed computer program is freely available and a method of repair procedure and a repair example, as well as recommendations for the design practice are also presented.

The design of structures is more and more directed towards the entire lifetime design with multiple concurrent objectives. Apart from durability, the most important factor in the whole life design of reinforced and, in particular, prestressed concrete bridges, is the Service Limit State. From this point of view, 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.

Due to excessive deflections, several bridges had to be either closed or repaired well before the end of their initially projected lifespan. The cost of reduced service life of structures is tremendous for society, the owners and users. In fact, it greatly exceeds, in strictly economic terms, the cost of catastrophic failure due to mispredicted safety margin.

Reliable prediction of bridge deflections during their construction as well as during their service life is of crucial importance for achieving good durability and long-term serviceability. Obviously, difficulty of predicting deflections is closely related to the properties of concrete (strength, elastic modulus, nonlinearity, creep, shrinkage, etc.), both initially and with time.

The long-term deflection behaviour of long-span prestressed concrete box girder bridges has often deceived engineers monitoring the deflections. A survey of many bridges monitored in various countries showed that all of them have experienced similar deflection histories. It has frequently been experienced that the box girders of many prestressed concrete bridges deflected far more than predicted in design. The deflection evolution has often been counterintuitive, with slowly growing deflections in the early years, followed later by a rapid and excessive deflection growth.

Figure 1. Bridge with excessive deflections after reparation process.

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Reliability-based assessment of RC bridges in a marine environment considering spatial and temporal variability of deterioration processes

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ABSTRACT

Durability analysis of deteriorating structures under uncertainty has to be based on structural reliability concepts and methods. In this paper, a stochastic finite element computational framework for reliability assessment of deteriorating reinforced concrete (RC) bridges is proposed. Spatial and temporal variability of the deterioration due to corrosion under uncertainty is considered. The analysis procedure considers the deterioration processes of concrete, steel and bond between concrete and reinforcing bars.

Nonlinear models for steel, concrete and bond are used to describe the behavior of deteriorating RC bridges. The concrete is modeled by means of four-node plane-stress elements, while the steel bars are represented by two-node truss elements. The bond-link element exhibits a relative slip between the two materials.

A computer code written in MATLAB version R2006a is used to generate the random variable inputs and to conduct the reliability analysis. The commercially available software ANSYS performs the nonlinear finite element analysis of deteriorating RC bridges. Monte Carlo simulation is used to compute the probability of violating various limit states.

For the sake of illustrating the proposed model and method, an external girder of a typical RC bridge is considered herein. The results from the space- and time-dependent reliability analysis of the girder are presented in terms of serviceability and strength, for different concrete covers and overload factors. This information can be used to predict the reliability and residual life of deteriorating RC bridges.

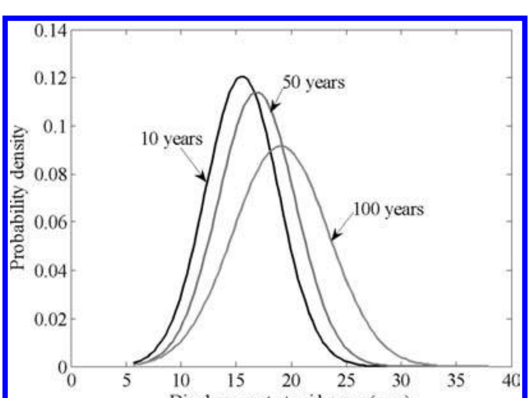

Figure 1. Probability density functions for live load deflection at mid-span at different bridge ages.

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MS13: Management & maintenance of long span bridges
Organizers: A. Chen & D.M. Frangopol

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Anticorrosive coating and maintenance of suspension bridges in China

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ABSTRACT

The main cables and hangers of suspension bridge, corrodes easily due to exposure in the sun, the rain, and repeatable stress combination. Its anticorrosive coating is of great importance among maintenance work. Based on research and study of anticorrosive coating development of suspension bridges home and abroad, this paper summarizes the anticorrosive coating schemes and maintenance, repair and service practice of suspension bridges in China, and proposes coating system suitable for bridge in China. It can be referred to in main cable protection and maintenance scheme.

The anticorrosive coatings applied in China include: "non-drying anticorrosive paste+ wrapping steel wire+ paint"," non-drying anticorrosive paste+ wrapping steel wire+ rustproof paint+ polysulfide sealant+ finish", and the compressive dehumidification system introduced from Japan. The most widely application is "phosphating rustproof paint+ non-drying polyisobutylene sealing paste+ wrapping steel wire+ phosphating rustproof paint+ polysulfide rubber sealant+ polyurethane (epoxy)finish."

Figure 1. The Shantou Bay Bridge.

Figure 2. The Shantou Bay Bridge, China.

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Integrity management of concrete bridges using Spatial Information Systems

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ABSTRACT

Risk assessment forms a cornerstone in the life-cycle integrity management of concrete structures subject to corrosion. To formulate the required risk assessment models, it is necessary to account for all available experience as well as data of relevance for the modeling of the deterioration processes. In the context of life-cycle integrity management concrete bridges represent large-scale systems and this necessitates that risk models account for the system effects and that such models together with relevant input data and results may be managed efficiently. The present paper presents a Spatial Information System (SIS) which facilitates on the one hand a consistent representation of system dependencies in the modeling of corrosion deterioration of concrete structures and on the other hand the efficient and intuitive management of the required input data and the analysis results.

The main aims with the proposed framework and tool are the following:

- To develop a probabilistic model of the deterioration for an entire large concrete structure taking into account the most relevant uncertainties and dependencies in the exposures (environmental conditions), the material characteristics and the model uncertainties associated with the applied deterioration models.
- To account consistently for the uncertainty associated with the performed inspections as well as the spatial and temporal allocation of inspections.
- To facilitate a practical applicable framework for the visualization and management of input data to the developed models, inspection data, repair activities and analysis results.
- To facilitate that risk assessments and maintenance management strategies may be easily updated on the basis of inspection results.

The general system representation in the risk assessment is adapted from Straub et al. (2009) and Malioka (2009) utilizing a hierarchical Bayesian temporal and spatial modeling approach. In regard to the visual representation and management of spatially distributed data very little literature is available as of present. Visual condition mapping has been reported in Wawrusiewicz (2007). However, for what concerns the integration of all model components as well as data and information required to support risk informed decision making in regard to optimal life-cycle integrity management of concrete structures, little or no research has been reported as of yet. In the present work the approach to the spatial risk modeling is thus adapted from Bayraktarli et al. (2006).

Finally, to illustrate the capabilities of the developed framework and SIS tool an example is provided where deterioration risk assessment and maintenance strategy optimization for the Farø bridge are considered.

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Research on structural performance evolution of a long span concrete bridge

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ABSTRACT

Structural performance evolution analysis is one of critical problems in safety assessment of concrete bridges in life-cycle. For the purpose of quantitative analysis on structural performance evolution of long span concrete bridges, numerical simulation methods of several routine mechanical problems and some degradation mechanical problems are firstly solved. Then an integrated computational methodology for simulation of carbonation- or chloride-induced degradation process of concrete bridges is proposed and an analysis program named Concrete Bridge Durability Analysis System (CBDAS) is compiled by FORTRAN 95. Finally, simulation results are presented for a long span prestressed concrete continuous rigid frame bridge exposed to concrete carbonation environment. The mainly research works are as follows:

(1) Simulation of complex mechanical problems
According to mechanical features of routine structural analysis, simulation of prestressing effect, concrete creep-shrinkage effect and structural system transformation in construction process are briefly introduced. According to mechanical features of durability structural analysis, simulation of deterioration of materials' mechanical properties, loss of various materials' section and the evolution of structural mechanical performance are mainly solved.
(2) Concrete Bridge Durability Analysis System
A computational methodology for simulation of carbonation- or chloride-induced degradation process of concrete bridges is proposed through integrating the above theoretical analysis on specific simulation methods of several routine mechanical problems and some degradation mechanical problems. According to the proposed finite element-based method an analysis program named Concrete Bridge Durability Analysis System (CBDAS) is compiled by FORTRAN 95. CBDAS totally includes eight subsystems that are shown in Figure 1.
(3) Numerical example
The CBDAS is implemented to simulate structural performance evolution in given service life

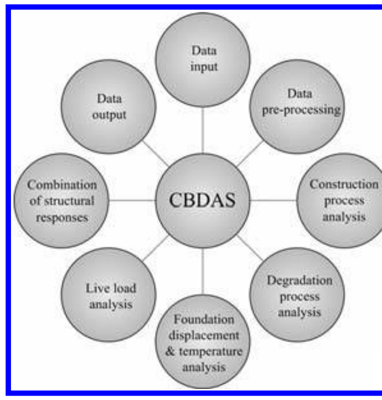

Figure 1. Integration of CBDAS.

of a 83 + 150 + 150 + 83 m prestressed concrete continuous rigid frame bridge exposed to concrete carbonation environment. The degradation process of various mechanical property indices on key sections are given respectively in serviceability limit state and ultimate limit state. The results show that: for prestressed concrete bridges, degradation of mechanical property indices in serviceability limit state should be paid more attention to.

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Example of bridge maintenance program applied by toll bridges agencies in Europe: The Rion-Antirion Bridge

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ABSTRACT

Located 250 km west of Athens in Greece, the Charilaos Trikoupis bridge, also called Rion-Antirion Bridge is a 4-pylon cable-stayed structure. Its 2,252 meter continuous suspended deck is the longest cable stayed deck in the world and its reference span of 560 meters ranks in the world top 10 longest cable stayed spans.The Concessionaire, Gefyra S.A. is responsible for the design, construction, financing, maintenance and operation of the bridge during the 35-year concession period. Therefore, Gefyra decided to work with Advitam to build methodologies and system to optimize structural inspection & maintenance process.

The structural health monitoring system (SHMS) of the bridge, which was initially developed to follow the long term structural deteriorations and improve maintenance, is now used for real-time earthquake management as well. The SHMS of the Rion-Antirion Bridge was initially developed through a risk analysis process performed during the design phase. The inspection and maintenance manual was issued in accordance with the results from the risk analysis. This provides the guidelines for all inspection and maintenance activities on the bridge and in order to complete the surveillance, the SHMS was set up to provide engineers with real-time data, as well as with other information which is unobtainable from visual inspection.

The structural health monitoring system was designed, supplied and installed by Advitam and it consists of hundreds of sensors. These include accelerometers on the free-field ground and pier bases to evaluate earthquakes intensity and bridge response, and accelerometers on stay cables, towers and deck to address the dynamic effects. Load cells on the stay cables monitor their tension, while there are deformation transducers on the expansion joints, meteorological stations on the deck and automatic water detection system in the piers, 60 m under the sea level.

The acquisition system on the bridge is composed of four independent acquisition units, which are powerful real-time data processing computers, one for each span. It digitalises the analogue signal at 500 Hz and provides the necessary data to structural engineers. Various types of files are created; dynamic ones with 100 Hz sampling in case of an alert, automatic ones at a specific time interval or even after request, and finally historical ones with lower sampling frequency. The software is parametric and modifications on the sampling, duration and so on are possible if required for more specialised post-processing and analysis. The SHMS has been providing valuable information and data to structural maintenance engineers and designers since 2004, concerning the long term evolution of the structure and the impact of special events.

The recent development of the system is based on the automatic analysis of earthquake data. A decision tree for earthquakes was developed, based on the ground acceleration intensity and on other structural parameters, which specifies all the immediate actions, for example the level of required inspection, and the need to interrupt the traffic. This principle has been implemented in the SHMS in order to provide the operator with an automatic decision-making system.

Based on the knowledge brought by the SHMS since 2004 on the dynamic Bridge behaviour, Gefyra and Advitam will continue to develop and implement new functionalities, including a real-time wind management system, and a general event detection and management system called 'Smart Monitoring'.

The objective of the latter is to minimise the volume of redundant data and store data with a specific sampling duration for special post-processing and analysis.

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Construction stages simulation for cable-stayed bridge on geometrical nonlinear analysis

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ABSTRACT

There are three main sources of geometrically non-linear behaviour of cable-stayed bridges: the cable sag effect, the beam-column effect and the large displacements effects. In this paper the cable sag effect is considered by the multiple-straight truss elements and the stress stiffness matrix is presented, the beam-column and the large displacements effects are considered using geometrical nonlinear analysis based on the Updated Lagrange method.

Stress stiffness matrix of cable elements contains unstress length of cables. Repeatedly stretching of cables can be used at the construction of the cable-stayed bridge for the purpose of improving structural mechanical behavior. The unstess length of cables may change at each stage. Supposing the tension force applied on cable is P_1, The unstess length can be achieved by Equation below:

$$l_{01} = \frac{l_1}{1 + \frac{P_1}{EA}}$$

where, l_1 is the distance of the two nodal coordinates of the cable element, E and A are the Young modulus of the material and the cable cross section, respectively.

The geometric configuration of the old elements and nodes is changed during the construction analysis of cable-stayed bridge. The node positions of the new elements can be gained by three methods: tangent to old elements, parallel to old elements and original model coordinate, which are shown in Figure 1, in which the dashed lines are the initial model positions. The construction stages analysis for cable-stayed bridge is performed with the tangent model which is shown in Figure 1a or the the parallel model which is shown in Figure 1b.

The purpose of construction control analysis of cable-stayed bridge in this paper is that the architectural configuration should be satisfied after construction control analysis is finished. The theory of construction control analysis is that construction stages analysis is performed with parallel model firstly, the incremental displacement, ΔU, is obtained, the old node position is gained by subtract ΔU from original nodal coordinate and the newly activated node coordinate is the initial model coordinate which is shown in Figure 1c. The nonlinear iteration is performed until the convergence tolerance is achieved.

The program for construction stages simulation for cable-staged bridge, Binas software, is compiled, the computational accuracy and the effectiveness are proven by the examples of the cantilever beam with cables and the Sutong bridge at the end of this paper.

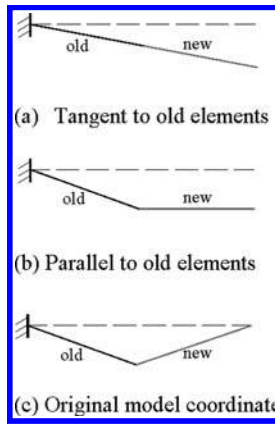

Figure 1. Three activated method of new elements.

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Use of sensors for efficient design of innovative seismic protection techniques for monuments

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ABSTRACT

One of the most difficult problems in the design of seismic protection for monuments is the establishment of their in-situ properties in order to be able to use them for an as accurate as possible model of the structure. In addition, any retrofitting method that will be used should be inconspicuous, and it should neither violate its form, nor change its structural behavior. The application of SMA and other innovative devices in protecting monuments are reported by Croci (2000) and Chrysostomou et al. (2008).

In this work the in-situ dynamic characteristics of an aqueduct (Fig. 1) are obtained using accelerometers to record the response of the structure to ambient vibrations, and hence finite element models which are updated based on those measurements are developed.

The dynamic characteristics of the aqueduct were determined twice: the first in June 2004 and the second in May 2007. The need for this arose from the fact that the application of the SMA devices on the structure was performed three years after the characteristics of the structure were determined, and it was therefore considered prudent to redetermine those characteristics so as to confirm the starting point. As it is explained, there was a considerable difference in the behavior of the monument between those two measurements. The main contributing factor was the effect of the level of the water in the nearby salt lake. In June 2004 the lake was dry while in 2007 the water level was high and it was obvious that the foundation of the structure was under the ground water table.

The effectiveness of the use of shape-memory-alloy wire-dampers for the protection of the monument from earthquakes is evaluated through their application on the monument and in-situ measurement of the changes in its dynamic characteristics (Fig. 2). It has been shown that the application of the SMA wires on a real structure changes significantly the dynamic characteristics of the structure.

Figure 1. Larnaca aqueduct.

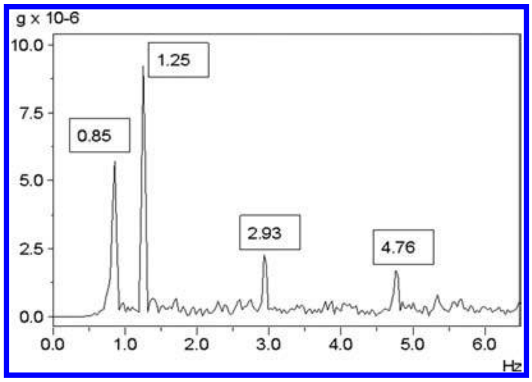

Figure 2. Frequencies for 4 SMA wires.

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In-node data processing in structural monitoring with wireless sensor networks

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ABSTRACT

The main advantages of wireless sensor networks are fast deployment, little interference and self-organization. These advantages come into play if an unattended operation of the network can be achieved for a reasonably long period of time. Since wireless sensor networks are battery powered, a power saving operation is a key issue. Power consumption is reduced by using low power hardware, energy efficient communication protocols and duty cycling.

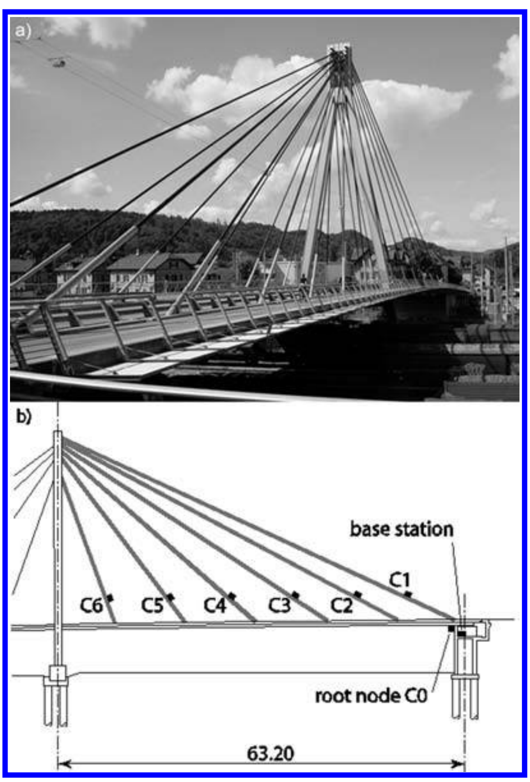

Figure 1. a) Stork Bridge in Winterthur, Switzerland. b) Locations of the network nodes.

Table 1. Estimations of selected natural frequencies of cable C1 to C6 in [Hz]. M1 and M2 are measurements made with a wired instrumentation.

Cable	Mode	M1	M2	WSN	Error (%)
C1	3rd	3.91	3.85	3.91 ± 0.06	0.77
C2	3rd	4.38	4.35	4.37 ± 0.05	0.11
C3	3rd	5.06	5.02	5.06 ± 0.03	0.40
C4	3rd	6.55	6.58	6.58 ± 0.05	0.23
C5	3rd	12.03	11.85	11.91 ± 0.19	−0.50
C6	3rd	7.77	8.00	7.78 ± 0.14	1.33

However, in data intensive applications, e.g. vibration based monitoring, these technologies are not sufficient for achieving a sustainable system lifetime. Significant power saving is achieved by in-node data processing with the goal to reduce dramatically the data that is communicated over a wireless channel. This data reduction, however, is a challenging task, since it has to be performed with very limited computational and memory resources.

This paper investigates the capabilities of this concept with a long term field test on a cable-stayed bridge. The goal of this deployment was to monitor the tension force of cable-stays via natural frequency estimations (Fig. 1). The paper shows that the data processing can be performed despite the significant restrictions provided by the low power hardware. Although the raw data is based on 12 bit data acquisition and the algorithms are performed with 16 bit operations, the results are accurate (Table 1). Furthermore, in-node data processing can be performed with nearly negligible power consumption. This allows, depending on the measurement interval, to reach system lifetimes of a year or more with a set of batteries.

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Capabilities of non-destructive testing of RC structures – quality assurance for crack repair using ultrasonic echo

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ABSTRACT

Cracking is a common phenomenon in concrete structures. Depending on their width and depth, cracks can significantly reduce the durability of the structure. To prevent the loss of durability and premature failure, the cracked structural member needs to be repaired. Several repair techniques can be applied for this purpose. One of the most common repair methods is epoxy, polyurethane or cement injection into the crack.

Having reliable quality control measures is of great importance to ensure the effectiveness of the repair procedures. So far, taking cores has been the only way to get reliable information about the quality of injection.

Experiments have proven that classical non-destructive testing methods to assess cracks, such as ultrasonic time of flight measurement don't work well for concrete. Rebars and contact between the crack surfaces (e.g. caused by grain) can act as acoustic bridges which makes the interpretation of the data more difficult.

In the past, best results were achieved by means of applying two dimensional surface scanning with an ultrasonic transmitter at one side of the crack and a scanning laser vibrometer at the other side, followed by three dimensional reconstruction (3D-SAFT, Synthetic Aperture Focusing Technique) of the data (Mielentz et al. 2001).

Based on these experiences, a new practice orientated method was developed which uses advanced array techniques (linear array, multistatic array) with dry-point-contact shear wave transducers for data acquisition and 3D-SAFT reconstruction.

To investigate the application of ultrasonic echo in evaluating the effectiveness of crack repairs, a set of laboratory measurements was carried out. Several commercially available ultrasonic devices as well as an in-house system developed at BAM were used to examine composite specimens, specimens with notches and specimens with cracks. Specimens with notches or cracks were investigated before and after filling/ injecting the notch/crack.

Significant differences between empty crack/ notch and the filled crack/ notch could be observed, if the crack/notch was filled with cement or epoxy. However, ultrasonic shear waves proved to be unfavourable for testing cracks filled with polyurethane.

The SAFT image-based data analysis revealed a big advantage over the single point measurements in interpretation of the test results. The SAFT reconstruction process includes many single measurements to calculate the image of the investigated object. In most cases, it was possible to distinguish between real indications of the crack and the spurious indications (e.g. caused by edge effects) in the reconstructed image.

Significant differences between empty and filled cracks could be observed. In a number of cases, the results indicated that the cracks were not completely filled. These indications were verified by ground-truth data. In summary, the outcome of this project clearly demonstrated the potential for future developments of a non-destructive quality control system for crack repair with cement or epoxy.

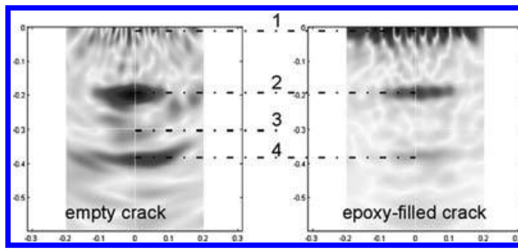

Figure 1. Results obtained at a specimen with cracks before and after injection with epoxy. 1: SH surface waves, 2: backside reflection, 3: artifacts caused by corner reflector effects, 4: multiple backside reflection

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Reference-free health monitoring system using chaos theory

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ABSTRACT

In this study, an attempt is made to develop a reference free health monitoring system that can detect the damage with ease, using the data collected from the vibration measurement. In order to investigate the damage state, it is, in general, necessary to measure the intact state of the structure in advance. However, there are many existing structures which have no such data. Thus, it is very useful to develop a new damage detection method without using the reference data. The proposed system can identify the location and intensity of damage without using the baseline reference data. Based upon the chaos theory, it is possible to perform the damage detection by paying attention to the chaos characteristics of the structure of response displacement wave. This study considers a simple symmetric structure shown in Figure 1 and Table 1 as a structural model. This study investigates the damage cases in which the 20% deterioration occurs below the node 14. Figure 2 is the results in which chaotic excitation is given to the structure against the direction of x axis. The standard deviation of the Difference of Spread of Local Area of Attractor (*DSLAA*) obtained at the 4th story is the largest among other stories. Besides the 4th story, remarkable difference of the standard deviation of *DSLAA* is not confirmed. From these trends, it is predicted that the 4th story has something abnormal. By evaluating the standard deviation of *DSLAA*, damaged story can be detected without using any reference information.

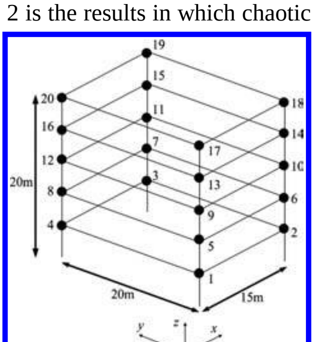

Figure 1. Structural model.

Table 1. Structural property

Moment of inertia $I_{y,z}$	0.0108 m^4
Young's modulus E	210 Gpa
Weight/m	27.68 kN/m
Poisson's ratio	0.25
Damping ratio for the 1st and 2nd mode	0.01

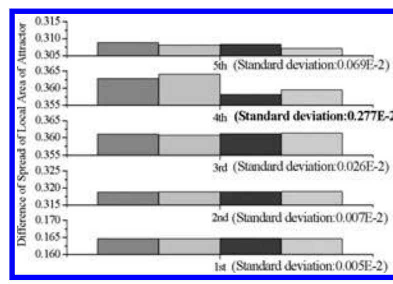

Figure 2. The results of Difference of the spread of local area of attractor.

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SHM in integrity management of deterioration prone concrete structures

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ABSTRACT

Uncertainties encompassing predictive models of concrete bridges make it difficult to decide the timing of management activities. These high asset value structures are subjected to a constant increase in loading frequency and severity, and exposed to harsh environmental conditions, often decay at rates higher than envisaged in design. Bridge management systems provide a systematic framework for repair and maintenance management. Key components of the system include regular assessment of existing condition through a comprehensive program of regular inspections, and predictive models regarding the future condition of the structures.

Visual inspections are subjective and qualitative. Occasionally supplemented with testing and monitoring but the need to access the vicinity of structure is a major hurdle in their use. Hence, the information obtained through the inspections, testing and monitoring and cannot be used explicitly and efficiently for performance prediction purposes. This aspect is highlighted in some detail in the paper.

Inspections provide information that is continuous in space but intermittent in time. Structural health monitoring (SHM) is typically continuous in time but spatially limited to certain points on the structure (Fig. 1). This information can be effectively combined to develop a powerful decision support tool for management of deterioration prone structures (Rafiq, 2005). This would increase confidence in the predicted performance by reducing associated areas of epistemic uncertainty in the probabilistic models. SHM supported predictive models can be used effectively to provide early warning to potential failures, detect and predict the rate of structural deterioration, and aid a decision support tool for the planning of maintenance on structural systems.

The concept of proactive health monitoring is presented whereby the monitored parameter is only remotely related to the deterioration process. Its application allows the planning or justification of preventative actions on the bases of increasing risk of certain defects being developing. This aspect is exclusive to proactive SHM where the structurally important defects have not yet physically initiated.

The methodology is demonstrated through its application on a concrete element subject to chloride induced deterioration. The extent of deterioration varies at different locations of the system due to temporal and spatial effects of the deterioration. The use of multiple sensors is presented for the above case and for the case where increased confidence in the systems performance is required at critical locations, or increased robustness of the SHM is required. It has been shown that the proposed methodology is applicable for these scenarios and that the overall performance of the element/system can be obtained by rationally combining similar data obtained through sensors at different locations.

Much work is need in this area before the approach can be implemented in practice. Key challenges in this regard are discussed in the paper. These are being addressed and will be presented in near future.

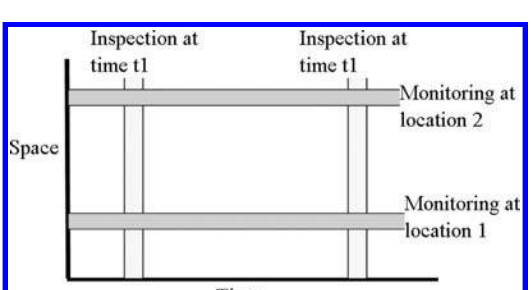

Figure 1. Interaction between information in space and time (Inspection vs SHM).

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Wireless sensor technology for continuous health monitoring of structures

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ABSTRACT

This paper gives an overview of continuous monitoring and local positioning principles for health monitoring of structures using wireless sensor networks, including recent results of the research project WiseSPOT (Novel wireless sensor nodes with smart antennas for localization of faults in structures). A novel approach for the development of a new generation of miniature low-powered wireless sensor nodes that will utilize advanced smart antenna technology for continuous monitoring, localization and tracking of events in is presented. Furthermore, the results of a first demonstrator of an optimized directional antenna add-on board are presented.

Despite this significant increase of the application of Wireless Sensor Networks (WSN) in various industries, their application to the infrastructure monitoring industry remains low, and this is mainly due to the limitations introduced by the current technology of wireless nodes. More specifically, wireless nodes currently used in WSN have significant limitations in energy consumption, communication range, cost, and their localization efficiency. These limitations impede their wide integration in applications such as health monitoring of large infrastructures (bridges, tunnels, water supply systems, transport networks, etc.), environmental control and many other industries, requiring deployment of sensor nodes on a large scale. The WiseSPOT research project aims to develop a new generation of miniature low-power wireless sensor nodes that will utilize advanced smart antenna technology for continuous monitoring, localization and tracking of events in the network environment (i.e, crack detection in large structures, land movements in landslide prediction etc.). Localization is a network function that requires the synergy of both hardware and software mechanisms, and is applied both for finding the relative and absolute locations of sensor nodes, as well as for localizing these events on the structure. An integral part of this research is the development of fast, energy efficient localization algorithms embedded on miniaturized sensor node hardware to be developed in the framework of the project. This technology is intended to develop a powerful remote monitoring tool for structural civil engineers that can be used for continuous monitoring of the health of large structures such as bridges, dams, the transportation and utilities infrastructure.

The paper describes a new architecture for developing a low powered wireless sensor which utilizes smart antenna technology. The proposed system satisfies the hardware needs for advanced localization capabilities. Initial results from the directional switching antenna add-on board developed under the WiseSPOT project showed very promising results as far as the communication range, power consumption and localization capabilities is concerned. To this end, further research will concentrate on the integration of the new antennas into a new sensor node which will utilise advanced localisation algorithms for the detection of faults in large infrastructure systems.

This two year project which started in January 2009, is co-funded by the Cypriot Research Promotion Foundation. Three SmartEN participants are involved in WiseSPOT and the project's results will be regularly disseminated to the SmartEN community.

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Role of monitoring in life-cycle assessment, prediction and management of deteriorating structures

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ABSTRACT

Assessment, prediction and management of deteriorating structures are crucially needed practices for the effective and economic upkeep of deteriorating structures. The uncertainties in the available information for conducting analyses pertaining to these practices impose a great challenge in the process. However, with the use of proper probabilistic tools for handling these uncertainties, more accurate outcomes do become achievable. In structures, one great source of uncertainty is related to the load effect. Code design values are traditionally used to establish maximum lifetime anticipated limits for these load effects. In many cases, these limits are highly conservative. In other cases, the loading demands of the structure unforeseenly increase over time to even cross these pre-set limits. This is clearly an undesired situation that should be avoided. As an effective solution, the demands of the structure may be monitored either throughout the lifetime of the structure, or during separate periods over its lifetime. Doing so greatly improves the knowledge regarding the real loading demands on the structure, reduces uncertainties in the input information, and provides warnings to structural managers when an overload takes place.

The objective of this paper is to illustrate how data obtained from monitoring can be integrated into the assessment, prediction and management of deteriorating structures. The role that monitoring plays in enhancing the outcomes of these processes is demonstrated. An example will be conducted over an existing bridge in the state of Wisconsin. Details of the bridge and its instrumentation are given in Mahmoud et al. (2005).

Inclusion of the monitoring data in structural performance evaluation most likely means combining this data with the prior existing information. A Bayesian approach is most suited for this situation. In engineering applications, treatment of monitoring data is associated with monitoring of extreme events. Extreme value distributions do not lend themselves easily to Bayesian updating (Bedford & Cooke 2001). The slice sampling technique is a recently developed technique (Neal 2003) that can also be used in difficult Bayesian problems as such.

As shown in this paper and illustrated in Figure 1 for an exterior girder in the bridge considered, the first sample monitoring data point produces a large effect on the prior information. This was followed by a decreasing effect as the sample size grows. Evidently, the reduction of the uncertainty due to incorporating the monitoring data is clear in the figure.

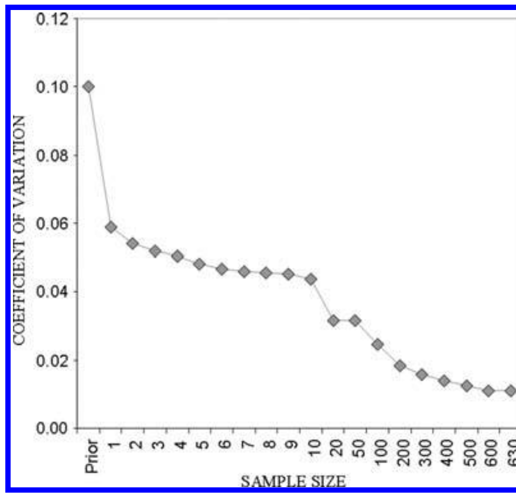

Figure 1. Changes in the coefficient of variation of the extreme value distribution parameter u_n with increase in monitoring information for the exterior girder.

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Predicting the life of reinforced concrete structures in severe marine environments

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ABSTRACT

It is generally agreed that reinforced concrete structures exposed to harsh marine environments will, within a space of one or two decades, start to show modest or even serious deterioration due to reinforcement corrosion unless special care is taken to prevent or reduce the rate of entry of aggressive chlorides.

Usually the life prediction is based on the rate of ingress of chloride ions. However, careful review of many older reinforced concrete structures shows that some have survived decades despite very high chloride levels and little or no protective measures or special additives. Conversely, there are cases where reinforcement corrosion is evident despite considerable concrete cover and high concrete quality.

A review is given of more than 300 cases for which corrosion initiation and corrosion progression took many years to occur. Considerable differences were found in the time to corrosion initation and in time to active corrosion. Figure 1 gives an example.

A small number of brief case studies are given, together with an extended discussion of the possible reasons why some reinforced concrete structures show much better long-term durability than others.

It is argued that long-term durability depends on pH reserves (i.e. alkalinity) and that this can be provided by aggregates such as limestone and non-reactive dolomites. This observation is consistent with corrosion science principles.

It is concluded also that calcium carbonate by itself (such as caused by 'carbonation') does not lower the concrete pH immediately adjacent to the reinforcement to permit corrosion initiation. The additional leaching of alkalies must occur before corrosion can initiate.

These findings potentially have important practical implications for the prediction of the life of reinforced concrete structures. This matter is currently under investigation.

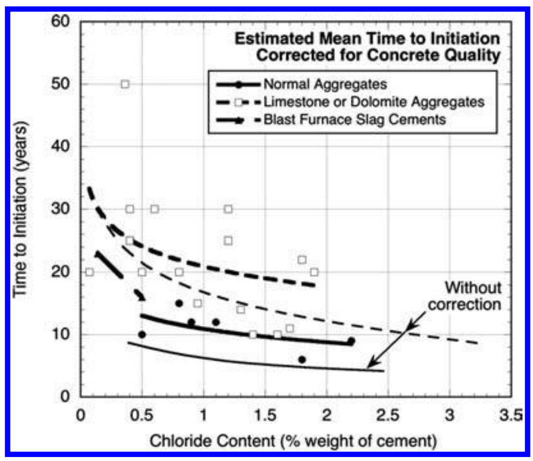

Figure 1. Mean time to initiation of corrosion as a function of chloride content in the concrete. The bold 'corrected' curves exclude structures affected alkali-aggregate reactions.

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Research and training challenges within SmartEN Marie Curie ITN

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ABSTRACT

This paper addresses the research and training challenges of the EU funded SmartEN Marie Curie ITN Programme which focuses in the area of Smart Management for Sustainable Human Environment. The programme aims to train the next generation of research leaders in a subject of paramount importance to European competitiveness through the development of an initial research and training network that will focus its activities on the development, effective integration and increased utilisation of emerging technologies in wireless sensors, communications and proactive management, targeting key issues of current interest to the European Union and internationally.

The programme brings together a collaborative research network that will focus its activities on research and training in the disciplines of Wireless Sensor Networks, Sensor Signal Processing, Non Destructive Evaluation, and Smart Proactive Management. Beyond the research challenges in the various individual disciplines involved in this programme there are many additional challenges associated with its multi-disciplinary nature and its trans-national and multi-centre research environment. Furthermore, the programme addresses many vertical and horizontal research themes, working both at a generic and application level. Key application areas to be considered within SmartEN include structural systems, heritage and infrastructure, transportation infrastructure systems, and urban microclimate.

The training challenges the SmartEN ITN network will face, include the development of a high quality comprehensive and multi-disciplinary training programme able to equip the young researchers with world-class scientific knowledge and necessary complementary skills in the area of research and development of IP. The intended programme consists of a number of interconnected and complimentary actions. These include the development of individual multi-disciplinary projects, training through personalized research performed locally and through intersectoral visits in academic institutions and industry, participation in multi-disciplinary scientific workshops and researching skills workshop, and dissemination and exploitation of research results in research conferences and workshops.

Individual multi-disciplinary projects, a core action of SmartEN project aim to provide researchers with opportunities to: (a) develop expertise beyond their core area through short duration placements, (b) focus more on the integrating technologies moving away from isolated insular research efforts and topics and achieving more effective component methodologies, (c) exploit the complementarity between the expertise of the various partners as well as their research and training infrastructure, and (d) promote long term networking and establish effective lines of communications.

In concluding, the aim of the SmartEN project is to provide innovative research and solutions in the above interdisciplinary area through the development of an initial research and training network. The significance and methods of effective management of these research and training key aspects are discussed in this paper.

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SmartEN – A research framework on smart management for sustainable human environment

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ABSTRACT

This paper presents a research framework which focuses on developing innovative technologies in the area of Smart Management for Sustainable Human Environment. This research framework underpins the development and implementation of the SmartEN Marie Curie ITN project.

This is an area of particular current interest worldwide, given the increasing concerns regarding the environmental impact of human actions, the use of the environment and climate changes. These are coupled with ageing infrastructure systems, continuously growing and changing demands on the built and natural environments and limited financial and depleting natural resources. Until now, research was focused on the development of proactive risk-based approaches for civil infrastructure reliability and management with benefits in improved performance, safety and cost. However, there are significant uncertainties associated with the various predictive models directly affecting the quality of the decision making. Recently, a new generation of miniature wireless sensor platforms which utilize novel digital signal processing has emerged. These can be adopted to obtain large quantities of highly diverse sensor data that are continuously collected over a long period of time from multiple locations providing significant insight on the condition, demands and performance of the system. These developments open up a completely novel area of multidisciplinary research towards the 'smart' management of sustainable environment.

The main objective of the SmartEN project is to implement a joint multidisciplinary research training programme which will be focused on the 'smart' management of sustainable environment. Project's activities will focus on research and training in the disciplines of Wireless Sensor Networks, Sensor Signal Processing, Non-Destructive Evaluation and Smart Proactive Management.

Project's aims will be achieved through: (a) conducting top level research and training and devise innovative solutions in the areas of monitoring and smart proactive management, (b) educating the next generation of intersectional and transnational researchers in the area of smart management of the sustainable environment and provide them with a unique range of skills and a network that will open up challenging and attractive career perspectives, and (c) taking forward the state-of-the-art in wireless sensor communications, digital signal processing and non destructive evaluation for the successful application of wireless sensors in the smart proactive management of the built and natural environment.

To this end, the aim of the SmartEN project is to provide innovative research and solutions in the above interdisciplinary area through the development of an initial research and training network that will focus its activities on the development and effective integration of emerging technologies in wireless sensors, communications and proactive management targeting key application areas of current interest to the European Commission and internationally and including monitoring and smart proactive management of Structural Systems, Heritage and Infrastructure, Transportation Infrastructure Systems and Urban Microclimate.

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Role of structural health monitoring in Pareto optimization of bridge management strategies

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ABSTRACT

Appropriate assessment strategies of bridges including visual inspections and monitoring programs are essential for determination of efficient maintenance strategies (Kim & Frangopol 2010). Nowadays, an important challenge is to include monitoring concepts in a general maintenance framework to determine optimal maintenance strategies (Frangopol & Messervey 2009, Orcesi & Frangopol 2010). Monitoring data provide additional information on the structural performance and can change the intervention strategy. Figure 1 shows, for three monitoring strategies, the cumulative monitoring duration that impacts, in turn, the accuracy of the additional information provided by monitoring on the structural performance. The effects of the duration of monitoring on accuracy of the prediction results and on the maintenance strategies decisions are investigated. As the monitoring duration increases, more information becomes available. The knowledge on the structural performance is then more accurate, but the monitoring costs are greater. Based on the accuracy of monitoring information, different decisions can be made.

The objective of this paper is to determine optimal assessment and maintenance strategies, considering multiple criteria. A reliability-based approach is proposed to include uncertainties associated with structural degradation of bridges. Decisions are expressed in terms of probabilities. Inspections, monitoring programs and maintenance actions are cost-dependent. Therefore, the associated expected costs are calculated. Since maintenance decisions depend on assessment strategies, the optimal in-depth inspection/monitoring times should be determined by taking into consideration all the possible outcomes of the performance assessment. Optimal inspection-monitoring-maintenance programs are determined by using a Genetic Algorithm based procedure (Deb et al. 2002). Optimal solutions are Pareto solutions that consider multiple criteria such as expected failure cost, expected owner cost and accuracy of the decision process, based on monitoring results. The proposed approach is applied to an existing bridge.

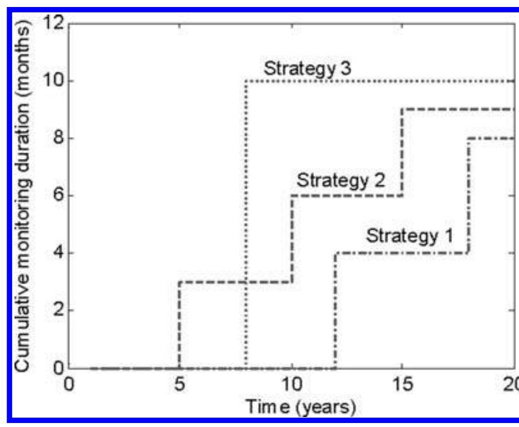

Figure 1. Cumulative monitoring duration for three strategies of monitoring.

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Optimization of life-cycle preventative maintenance strategies using genetic algorithm and Bayesian Updating

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ABSTRACT

Bridge maintenance is an effective way of maintaining the safety, condition and efficient operation of bridges. Preventative maintenance (PM) can play an essential part in the structural maintenance of bridges, and if planned effectively, can result in reduced whole life maintenance costs while improving their safety and performance profile. The authors have developed an optimization genetic algorithm (GA) methodology that enables the optimization of PM strategies applied to reinforced concrete (RC) bridges.

The PM measures are used to delay/prevent the reinforcement corrosion of bridge beams due to contamination from chloride ions present in de-icing salts. By combining probabilistic modeling of PM effectiveness and GA optimization principles was feasible to produce PM strategies that can maintain the reliability profile within the acceptable limits while minimizing the whole life costs.

A key element in predicting optimum PM strategies using the GA methodology is the accuracy of estimating the degree of deterioration of an element. To further improve the reliability of this estimation Bayesian Updating is utilized. The use of Bayesian updating enables the updating of the probability of failure based on site inspection data and the adjustment if necessary of the timing of subsequent PM interventions. To demonstrate the application and the effectiveness of the proposed updated GA methodology 3 case studies are presented in this paper. The studies also examine the influence of applying Bayesian Updating at different time frames in reaching the optimum PM maintenance strategy.

In case 1 Bayesian Updating has not been used. In case 2 updating takes place at year 5 while in case 3 the updating occurs at years 5 and 30. In the cases employing Bayesian Updating data on chloride concentration at the rebar obtained from two assumed inspections (year 5 and year 30) are incorporated in the probabilistic model. In cases 2 and 3 the inspections data showed that the chloride ingress was less severe than originally predicted. By incorporating this observation through the Bayesian Updating it was possible to reduce the pf and in case 2 to improve the strategy and result in a lower overall cost. This is interpreted as finding a strategy with lower cost than the initial that keeps the pf lower than a target pf at all times. The overall reduction of cost was 6% which can be a significant amount considering the whole structure. It is expected that further benefits can be obtained by incorporating values from regular inspections and more available PM measures through Bayesian Updating. These can have a significant effect and benefits in improving the reliability and cost of optimum strategies.

The methodology developed although specifically demonstrated here for RC bridge beams, is generic and can be easily adapted for application to other types of deterioration processes or structural elements and systems.

Further work will be carried out to investigate the full effect and potential of Bayesian Updating in conjunction with regular inspections and/or monitoring and the GA methodology.

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Health monitoring of a smart base isolated benchmark cable-stayed bridge using symptom approach

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ABSTRACT

This paper presents a smart base isolation system for cable-stayed bridges, which consists of passive hysteretic devices between the deck and the piers. The ASCE benchmark cable-stayed bridge developed in MATLAB is used as case study in an updated model developed in the commercial finite element code ANSYS. The ASCE benchmark on cable-stayed bridges has gathered, in the last years, the interest of many specialists in the field of the structural control and the dynamic response of long span bridges. Starting from the structural model of the original benchmark statement, a refined version is developed in a commercial finite element environment to include new modeling aspects, in the simulation of the stay cables dynamics and in the soil-structure interaction. The model comprises soil-structure interaction using impedance functions, the foundation are simulated by lamped masses with equivalent spring and dampers. Focusing the attention on the simulation of the structural dynamics, the cable model is refined moving from the single rod type representation, used in the benchmark, to a description with six rope elements for each cable. The monitoring of the bridge is approached in the symptom space, assuming that the bridge degrades and deteriorates with time. Damage to the longitudinal isolator between the deck and the pier is considered, while 42 observations are taken in account throughout the life span of the bridge.

Excitation is applied in the time domain in the form of traffic load. This is simulated as translating white-noise load intensity fields. Particularly, 8 traffic lanes, with a moderate percentage of heavy vehicles, are simulated as approaching the bridge from opposite directions, moving at a moderate speed. The loading process is carried long enough to assure the low structural modes can be excited. The moving traffic load is applied on the finite element model in the form of a vertical load, bending and torsional moments at the nodes of deck that are described by individual nodal time varying functions.

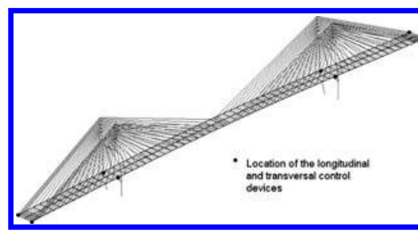

Figure 1. Finite element model in ANSYS.

The symptoms considered in the case studies are vibration related parameters at selected position along the deck. The proposed structural condition monitoring system is applied to detect the existence of different types of damage in the bridge. Symptoms are ordered in a rectangular Symptom observation matrix (SOM) opportunely normalized. Each column of the matrix describes the change in time of each symptom, while each row corresponds to a given observation at a given time t of all considered symptoms. By successive application of single value decomposition (SVD), it is possible to obtain full extraction of fault-related observations from SOM. In other words, it is possible to pass from multidimensional non-orthogonal symptom space to orthogonal generalized fault space, of much reduced dimensions. These information are useful for redesign of CM systems.

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Reliability of SHM procedures and decision support in infrastructure management

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1 INTRODUCTION

Infrastructure management is one of the topics that has received increasing attention from both the academic and the engineering community in the last ten or fifteen years. Indeed, the cost of infrastructure maintenance is becoming in many countries an overburden that reaches several GDP points, thus rendering the implementation of strategies for maintenance cost optimization a must for infrastructure owners and governmental organizations.

Consequently, the approaches to systems management, risk and maintenance engineering already common in the industrial field are becoming popular also in the civil engineering field, opening new research lines commonly referred to as *life-cycle engineering* approaches (Frangopol & Liu 2006).

Significant differences however appear when comparing the industrial systems to the civil infrastructure. The main difference appears comparing the operating life of industrial systems and products and of civil infrastructures. This difference implies that the causes of degradation of the safety and usability conditions are much wider for the civil infrastructure case.

A second but not secondary difference is related to the fact that the phenomena that render the behavior of real structures different from the corresponding design models are very complex and unknown to a significant extent. As a matter of fact, industrial products are very similar, if not identical, to one another while every civil structure is built in different environments and at different times, with materials like masonry or concrete the characteristics of which may be different from one case to another, thus rendering the real as-built state substantially different and to some extent unknown.

This latter consideration is very important when dealing with already existing infrastructure, the state of which may be very difficult to assess (Aktan et al. 2007).

2 INFRASTRUCTURE MANAGEMENT, SHM AND DSS

All the above considerations render the subject of optimal infrastructure management and the transfer of procedures already established in the industrial engineering field a very questionable issue. In the paper, such procedures are briefly reviewed in their adaptation to the civil infrastructure field and the question will be raised concerning how a theoretical model of the decisional process involved could be constructed in view of the formalization of a suitable Decision Support System (DSS).

Recognizing that a proper decisional model cannot be based other than on observation of the real behavior and conditions, monitoring approaches will be reviewed with special reference to the process of deriving information on potential degradation states from the monitoring data (Del Grosso et al. 2010).

Finally, taking into consideration some of the commonly used probabilistic reasoning schemes, the effect of the reliability of the damage identification algorithms on the reasoning models will be discussed and potential lines of research are traced (Del Grosso, 2008).

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Crystal clear data fusion in subspace system identification and damage detection

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ABSTRACT

Subspace-based linear system identification methods have been proven efficient for the identification of the eigenstructure of a linear multivariable system in many applications. Moreover, damage detection using null space based stochastic subspace detection techniques has also been proved to be useful in structural health monitoring during the last decade. Our main motivation in this paper is output-only structural identification and damage detection in vibration mechanics.

The problem consists in identifying the modal parameters (natural frequencies, damping ratios and mode shapes) of a structure subject to ambient unmeasured vibrations, by using accelerometer measurements or strain gauges. This is output-only system identification, as the excitation input is unknown and not measured. Examples are, amongst others, offshore structures subject to swell, bridges subject to wind and traffic, etc.

We analyze how the Stochastic Subspace Identification (SSI) can be adapted when several successive data sets are recorded, with sensors at different locations in the structure. For doing this, some of the sensors, called the reference sensors, are kept fixed, while the others are moved for a so-called multi-setup measurement. Like this, we mimic a situation in which lots of sensors are available, while in fact only a few are at hand. However, there is one unpleasant feature of structural identification of structures subject to ambient excitation, namely that excitation is typically turbulent in nature and nonstationary. Like this, the excitation factor can change from setup to setup and has to be normalized for the data fusion.

We describe a new merging algorithm for multi-setup measurements that is common for both covariance-driven SSI and the data-driven SSI using the Unweighted Principal Component (UPC) algorithm. This merging approach called PreGER (Pre Global Estimation Re-scaling) introduces the same excitation factor to all the setups, then merges the data and does the system identification in the final step. A new strategy called "crystal clear" is used to clean the resulting stabilization diagrams from unstable and noise modes in the identification step.

Furthermore, the uncertainty of the obtained modal parameters is evaluated. For this, an existing algorithm is adapted to the covariance- and data-driven SSI used in this paper.

Finally we derive a version of the null space based statistical damage detection algorithm that is adapted to multi-setup measurements.

All the presented algorithms are applied to the Z24 bridge, being a benchmark of the COST F3 European network. The system identification results show high quality, confidence intervals are computed on the obtained frequencies and damping ratios, and damage is successfully detected when comparing multi-setup measurements of two damage scenarios of the Z24 bridge.

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Experiments for damage detection by subspace identification on a tied arch bridge

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ABSTRACT

In this lecture it is proposed to identify the dynamic characteristic of a structure by vibration measurements. In the case of damage this characteristic will be alternated. At the present time the experimental modal analysis is often used to determine eigenfrequencies, modal damping and mode shapes. For damage detection finite element models will be optimized by experimental datas through model updating. The correct choice of parameters which will be adapted in the finite-element model is crucial for this procedure.

Here another method is to be presented for damage detection and localization. Black-box state space models can be identified by subspace method from measurement datas. These identified state space models represent the transfer function between input and output. The black box model for the intact system is compared with the black box model of the damaged monitored system. Variations of the structure can be detected by evaluating special damage indicators for instance by the static or dynamic influence coefficients ([Link et al. (2007) Link, Stöhr, and Weiland] and [Lenzen and Waller(2003)]).

Additionally we will present a first step to transfer black-box state-space models into white-box models in this lecture. White-box models are physically interpretable and permit direct damage localization. The possibility of extracting mechanical properties like mass, stiffness or damping direct from identified state space models will be shown by theoretical mechanical equations. Because of differences between theory, simulation and experiment this is more difficult by real measurements. Nevertheless the identified model parameters (e.g. Markov-Blocks) are able to detect and localize variances of mechanical properties.

Results from experimental measurements in our laboratory on a cantilever bending beam will show that the presented methods are able to localize changes of stiffness and mass. A rectangular steel pipe ($80 \times 40 \times 2.9$ mm) with a length of 2.45 m was used as test object. For vibration measurements eight one-dimensional acceleration sensors were attached equidistant. The mechanical structure was excited by impulse loads. Furthermore experiments on a prestressed concrete tied-arch bridge in Hünxe (Germany) will be presented. The bridge (built in 1952) had a span of 62.5 meters and was deconstructed in 2005. Main- and cross-girder, track-slab and the hanger consisted of prestressed concrete, the arch was built in reinforced concrete. On the verge of deconstruction it was possible to accomplish numerous vibration measurements. For the experiments two states as a variation of the structure were induced. First an additional support near the bridge bearing of one main girder was set-up. In a second experiment one hanger from one tied arch was cut through.

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Web based monitoring and assessment of bridges and structures

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ABSTRACT

Structures like bridges are exposed to a variety of environmental influences like wind, rain or traffic load which induce corrosion and material fatigue. Owners of structures want to or even have to know exactly the condition of the structure due to different reasons. Therefore over the last 10 years numerous monitoring systems for bridges and structures have been installed. Permanent monitoring systems in remote areas often have to work without connection to the local power and telecommunication networks. Moreover site visits to pick up measured data and automatically generated reports are costly and impose a certain time delay from measuring until notification of the owner. This fact emphasizes the need of energy autonomous and web-based remote monitoring and assessment.

Modern monitoring systems can be designed to constantly measure changes in parameters such as length, position, force, pressure, temperature or eigenfrequencies. This information is measured at desired time intervals and recorded for analysis. The potential applications of automated monitoring systems are almost unlimited, but would underlie the following general areas among others:

- Safety concerns: ensuring immediate notification of the occurrence of a change in a structure's condition that may indicate that the structure is becoming or has already become unsafe to use.
- Engineering data: supplying records of loading and movements to which a bridge is subjected, and the structure's response to the conditions.
- Usage data: providing records, such as weight and speed of traffic using a bridge. Cameras can be integrated to gather visual proof of traffic events.

Basically a modern monitoring system consists of a central processing node, at least one sensor, and some kind of connection to the central processing node. A power source – local power supply system or some kind of independent power supply system – as well as enough memory to store data is required, too. More sophisticated systems can also offer additional features like dynamic weight registration, to enhance the capabilities of the system depending on local circumstances and client's requirements.

Modern monitoring systems feature also web-based control and monitoring capabilities. Clients can access the measured data via a web-interface either in graphically prepared format or tabular format. Measurement data provided for download afford the opportunity to use it in calculations afterwards. Thresholds or more complex events can be configured for measured parameters to induce an alarm via SMS, email or simply on the web-interface.

Reconfiguration of the monitoring system like sample rate, measuring intervals or changing the algorithms for assessment is possible through the web-interface. Remote reconfiguring is reserved to the monitoring system implementers because of their expert knowledge of hard- and software.

Hardware design and installation are mature; however awareness of the benefits of a modern web-based remote monitoring system for structures still has the potential to increase. Such system provide an exciting future for clients, for research and development as well as for special applications (e.g. modal shape analysis or dynamic weight registration).

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Practical implementation of SHM with a special focus on the end user's needs

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ABSTRACT

Health monitoring for civil engineering structures is a challenge. Our structures are a prototype each and show small safety margins and a great exposure to the public. Bridges for example were the backbone of powerful empires from China to Rome and the Incas in America. The transportation infrastructure is directly related to economic success of a nation.

Structural health monitoring of civil engineering structures is more difficult than any monitoring of a well defined mechanical structures. The major number of uncertainties in geometry, material properties and the influence of the environment might have a higher impact on monitoring results than any minor damage. Therefore only complex approaches under consideration and compensation of the already known phenomena will be successful. This requires a more or less scientific approach.

Nevertheless an appropriate focus on the end-user's needs must not be forgotten to achieve a more widespread implementation and to increase the acceptance by the clients. End-users require robust and reliable structural health monitoring concepts with full automatic data analysis, interpretation of results and notification in case of problems. These concepts have to cover the whole availability period of the structures starting form the design and construction phase and ending with the demolition. The economic benefits have to be shown clearly.

The motivation to apply and order services based on the new technologies can be:

- Responsibility driven, which means the new methods to become standard applications supported by codes, standards and guidelines.
- Economically driven motivations, such as situations where a ranking of structures to be rehabilitated is necessary because of insufficient budget available or the need to use a structure for a certain time period longer than designed.
- Curiosity driven motivations comprise those cases where clients would like to know more about their important and complicated structures. Results can also lead to better planning for future structures.

The usual life cycle assessment methods are not very reliable and therefore tend to predict towards the safe side. Differences between theoretically calculated and observed lifecycles may differ in an order of magnitude. The lifecycle assessment by means of parallel structural health monitoring can considerably improve the accuracy of these predictions.

If we close all structures where the predicted lifetime has ended, our transportation network would break down completely. The pressure on the SHM community to produce prediction of longer lifecycles will dramatically increase in the near future. Structural health monitoring has the burden to identify those structures where less maintenance input will not lead to unacceptable deterioration and secondly to accurately assess those structures where critical stages have been or will be reached any moment.

Actually the main deficiencies of a widespread implementation of SHM are:

- Closed solutions, which due to the complexity of the subject are almost impossible to be identified.
- The step from laboratory to the field has not been successfully managed. In many approaches environmental conditions govern the response in the field. This completely invalidates the assessment results.
- A clear focus on the non scientific end user's needs.

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Recording and simulating environmental effects upon Tamar Suspension Bridge

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ABSTRACT

It has been noted from long term monitoring results that the static and modal data fluctuates over time, which is shown in Figure 1. It is noted that this correlates with measured wind speeds and warming of the bridge elements. For damage detection, an understanding of the environmental effects is essential, since they will be the major contributor of noise to data otherwise indicative of structural damage.

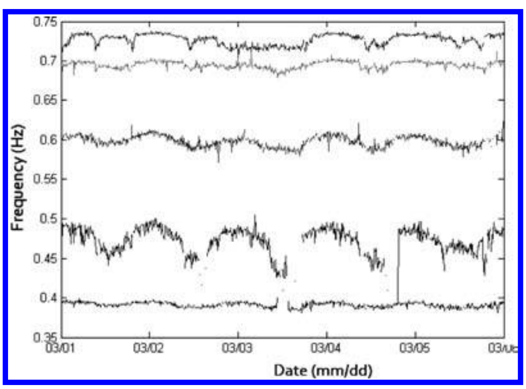

Figure 1. Modal frequencies of the Tamar Bridge throughout a week in March.

Instrumentation installed upon the Tamar Suspension Bridge, in south west England, UK, has been collecting modal data since 2006. Typical monitoring equipment has been used, such as accelerometers and strain gauges, as well as the recent use of total positioning station (TPS) to collect global displacements. This has been presented in an accessible program for ease of reference.

Accompanying the research, a finite element (FE) model of the bridge has been created, as shown in Figure 2, to provide theoretical results and further the understanding of effects from environmental loading. The accommodation of the hybrid structural system of suspension cables with stay cables has been considered, as well as geometric non-linear effects of the forces in the wires, and the catenary of the suspension cable.

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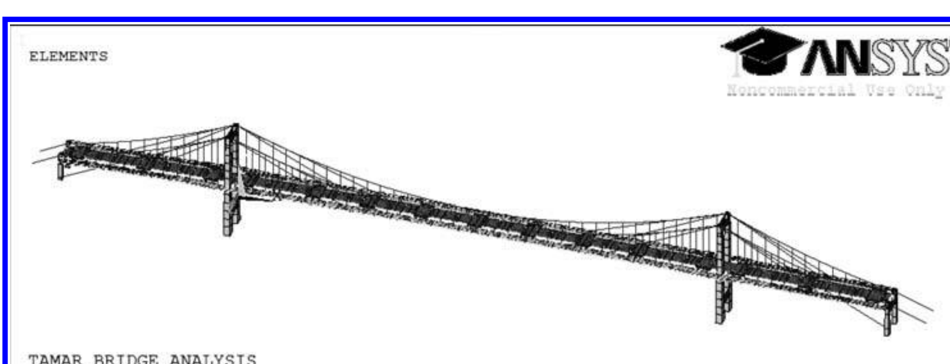

Figure 2. Finite element model of Tamar Suspension Bridge.

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A statistical method for masonry arch bridges damage detection

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ABSTRACT

This work deals with the application of a vibration-based damage detection method to masonry arch bridges. The motivations which lead to the introduction of SHM methodologies and non-destructive assessment methods in the maintenance and preservation of the historical bridges are presented. The critical issues and the challenges of the implementation of automatic diagnostic monitoring systems are addressed too.

The reliability of a monitoring system design relies on the identification of the sources of weakness and vulnerability. Therefore, the most common causes of damage for masonry arch bridges are illustrated. The foundation settlements are identified as the most threatening damage causes for historical bridges because of their accidental nature and their tragic consequences on the structural integrity. In order to investigate this damage scenario a scaled model of a masonry arch bridge was built in the laboratory and subjected to an extensive campaign of vibration tests.

The application of an experimental modal analysis technique on the results of the dynamic acquisitions allowed to identify the modal parameters and to assess their evolution in time. In order to investigate the influence of the environmental factors on the variation of the identified modal parameters some specific environmental tests were arranged.

A novelty detection algorithm was implemented to statistically prove the occurrence of the change in the state of the structure suggested by the decreasing variation of the identified natural frequencies in time. The concept of the vibration-based damage identification expressed in the terms of the Statistical Pattern Recognition paradigm is introduced and its methods are classified according to the required level of the damage assessment, the pursued approach to the problem and the availability of the damaged and undamaged data.

The outlier analysis based on the modal parameters identified from the vibration tests statistically proved the occurrence of a change in the system which can ascribed to damage after the exclusion of the dependency with variation of the environmental conditions. The results of the analyses carried out employing the natural frequencies proved the reliability and robustness of these features. The observed increase of the Mahalanobis squared distance with the testing sets confirmed the suspects of the evolution of damage characterised by an increasing extent in time. On the contrary, the damping ratios resulted to be too scattered and prevented to build a statistical model of the unfaulted condition. This influenced the ability of the method to distinguish the observations deriving from different states of the structure.

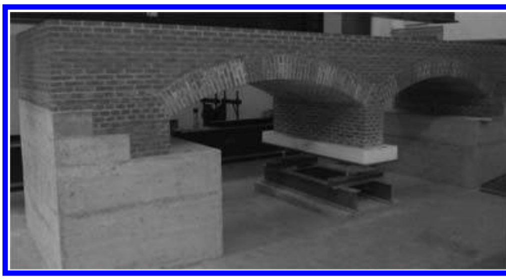

Figure 1. The experimental model of the twin-spans masonry arch bridge built in the laboratory.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Proposal of a workers and scaffolds monitoring and risk mitigation system for building sites

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ABSTRACT

Building site is a place where workers are exposed to many risks, and moreover it is characterized by one of the higher death-rate into the entire industrial sector. The main reasons are related to a wrong use or even to the missed use of the safety equipment by workers, falls form a height or into a dig, falling objects, the wrong assembly and use of scaffolds and their aging.

In particular, falls from a height has been identified as the main cause of workers death, while the scaffold collapse has been identified as the main risk factor during the erection stage of building site.

To face such a scenario, the design of a sensor network, both wearable by the worker and dislocated in strategic places into the building site, could be a proper proposal to monitor the workers safety. Several sensors can be organized in one or more networks, with variable dimension and range, in order to realize a monitoring and warning system able to perform punctual and continuous operations (otherwise not reachable in any other way) and to quickly alert the staff in case of an abnormal checked condition.

The ChiLab proposal concerns a network composed by two sub-networks: a Body Area Networks (BANs), one for each worker, with the main task of monitoring the worker status and in particular if he is fallen down; a Scaffold Area Network (SAN) able to monitor some important parameters such as stability of scaffolds erected into the building site.

These two sort of sub-networks can interact to each other in order to improve the management of the worker safety in building sites.

Any single network (BAN or SAN) has an own coordinator able to manage the data collected from the sensors, and any coordinator is able to directly or indirectly reach a gateway. There is just one gateway for each building site that has the purpose of forwarding the data collected to a remote device (for instance a personal computer), where a decisional algorithm generates alarms and alert messages.

Several different types of sensors can be involved into the networks. For instance, into the BAN network can be installed one or more accelerometers for falls monitoring, one or more pressure sensors embedded in the shoes to check if the worker is carrying heavy loads, one or more acoustic sensors for monitoring if the worker is exposed to an high rate of acoustic pollution, other types of sensors (temperature, humidity, biometrical sensors …).

On the other hand, a SAN network can be composed by strain gauges for axial forces monitoring, LVDT for lateral displacements monitoring, gyroscopes for structure tilt monitoring, high bandwidth accelerometers for structure vibrations monitoring and, finally, low bandwidth accelerometers for structure low frequency oscillations monitoring.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Observed dynamic characteristics of an overpass bridge during destructive testing

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BRIDGE DESCRIPTION AND DAMAGE SCENARIOS

Vibration measurement and analysis of dynamic characteristics of an overpass bridge during a full-scale destructive testing are described. The tested bridge is the S101 Overpass Bridge located in Reibersdorf, Upper Austria, west side of Vienna, Austria. The bridge crossed over the national highway A1 Westautobahn Austria. It is a post-tensioned concrete bridge with the main span of 32 m, side spans of 12 m, and the width of 6.6 m (VCE 2009). Measurement system consists of six triaxial accelerometers. During two days measurement, ambient vibration of bridge was measured using six sensor configurations.

Damage was introduced to the structure by cutting the pier column just above the pier footing. A hydraulic jack was placed on bottom of the steel column to provide a temporary support. Immediately after the cutting process was completed, the temporary steel column was lowered gradually by releasing the pressure in hydraulic jack. This caused the vertical settlement of the bridge at the location of pier column.

RESULTS OF VIBRATION ANALYSIS

Through systematic data analysis using spectrogram, output-only modal analysis, and multivariate outlier analysis, important results of the study are summarized as follow:

1. A non-uniform pier settlement – simulated as damage in this study, affects global stiffness of structure significantly. This is evident by the significant change in frequency of low-order modes. The effects are more obvious in torsional modes than in bending modes, as indicated by larger changes in frequencies of torsional modes than that of bending modes.
2. Damage also alters the mode shapes locally. Modal displacements at the pier-girder node for damage cases increase significantly suggesting immediate effect of constraint-losing at the boundary condition. The changes are evident from bending and torsional low-order modes and are well predicted by FEM. Effects of damage on mode shapes are more obvious in torsional modes than in bending modes as indicated by larger changes in modal displacement of pier-girder node of torsional modes than that of bending modes.
3. In general damping increases as the damage level increases. Estimations from system identification (Juang and Pappa 1985) indicate that damping in damage stages increase up to 2.5–3% from previously 1.5% in undamaged stage.
4. Multivariate outlier analysis (Worden et al 2000) is used to detect the presence of damage. The results show that using Mahalanobis distance of auto-spectra the presence of damage at the earliest stage can be detected with reasonable accuracy. When damage has significantly changed the characteristics of structure such as the case, all detection points are unambiguously detected as outliers indicating the clear presence of damage. Also the distance between threshold line and damage points in outlier detection are increasing as damage becomes larger. This distance can be used further as indicator of damage severity.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Case based reasoning systems for comparative assessment

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ABSTRACT

Aging and corrosion increasingly affects our infrastructure, what makes regular and systematic inspections indispensable, especially of critical structures (e.g. bridges as a part of the traffic infrastructure). An inspection normally consists of (vibration-) measurements and visual classification as a basis for further analysis and assessment done by experienced experts. It is obvious that this generally is a time-consuming process and besides that a certain amount of subjectivity has influence on the results. In this contribution a system is described which is based on the methodology "Case-based Reasoning" (Aamodt & Plaza 1994) for providing computer aid to experts working in the field of measurement analysis and structure assessment. The Case-based Reasoning (CBR) paradigm can be described as a drawing of conclusions from comparable historical cases to solve new problems including the integration of newly found solutions to step-by-step improve the future problem-solving-ability of the system, which consequently is based on a cooperative approach (human-computer) of knowledge acquisition (Freudenthaler et al. 2008). A solution in this context can be a single piece of information or even a kind of algorithm to solve an initial problem. Apart from numerous advantages of Case-based Reasoning, purely case-based approaches still have some shortcomings and the definition of basic rules can lead to higher performance and precision.

The current state of the CBR system for the interpretation of measurements could be termed "research prototype". Results of first experiments, carried out on measurement data of simple structures, reaching an overall error rate below 10%, were quite promising (Freudenthaler et al. 2009). This error rate mainly results from the experts' subjectivity, particularly in marginal cases, from shortcomings in data acquisition and from matters of fine tuning of the system. Nevertheless, for some situations the prototype provides remarkable results. To predict if simple structures are in excellent condition showed an error rate below 1%. To overcome the issue of subjectivity of experts, it is advantageous to keep the user informed on the current performance of the case base. Performance in this connection relates to the efficiency of clusters within the set of cases. This means that a user is informed how clear the clusters in the case base are before including new information. Relying on that, he can decide that cases, which cannot be classified unambiguously, should not be included used as reference cases for future problems. Not administrated case bases can become unsystematic and can lose in performance.

The application of systems for automated classification of measurements of simple structures or structure components promises good results and in certain cases even offers the chance of a fully automated process.

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European approach on integrated infrastructure risk management

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ABSTRACT

At present the European Industry recognized their obligation to **reconsider risk and safety policies**, having a more competitive industry and more risk informed and innovation accepting society in vision. Therefore the large collaborative project **IRIS** is proposed to identify, quantify and mitigate existing and emerging risks to create societal cost-benefits, to **increase industrial safety** and to **reduce impact on human health and environment.**

Motivation
Current practices in risk assessment and management for industrial systems are characterized by its methodical diversity and fragmented approaches. In retrospect these risk and safety paradigms resulted from diverse industries driven and limited by available knowledge and technologies. A change based on industry driven R&D work is needed.

Project Outline
The project is led and driven by industry to consolidate and generate knowledge and technologies which enable the integration of new safety concepts related to technical, human, organizational and cultural aspects. The partnership represents over 1 million workers.

The proposed project integrates all aspects of industrial safety with some priority on saving human lives prior cost reductions and is particular underpinning relevant EU policies.

Basic Concept
In short the concept of IRIS is to focus on diverse industrial sector's main safety problems as well as to transform its specific requirements into integrated and knowledge-based safety technologies, standards and services.

The project covers a broad range of industrial sectors and involves main stakeholders. Furthermore significant demonstration elements, training activities and technology transfer, also on international level, create a leverage effect for the acceptance in industry and society.

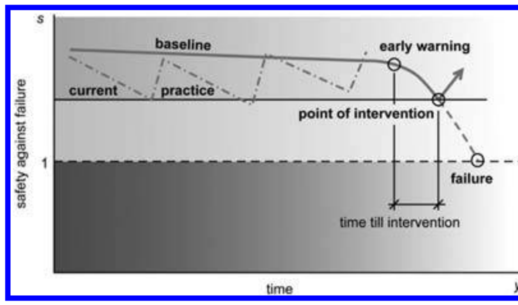

Figure 1. The current practice line shows preventive maintenance actually not necessary. A firm knowledge of the baseline will enable the omission of unnecessary works und such reduce costs, without sacrificing safety! Design can make use of the information and save wherever over design exists!

The Partnership (>1 million workers) has been selected out on the principles: complementarities, non competition and commitment to the subject of industrial safety. The top management is committed to take a big step towards considerably improved safety performance in the European Industry.

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MS16: Safety & management of bridges in Mexico
Organizer: D. De Leon

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Basis for risk management of bridges exposed to seismic loading

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ABSTRACT

Bridges located on seismic and highly populated zones involve a great amount of risk that should be mitigated. Quantitative descriptions of the seismic hazard and the risk involved due to the bridge importance constitute valuable components for the management of mitigation measures to optimize limited resources for civil protection and bridge maintenance in Mexico.

According to Meli (1994), the reliability of a structure is associated to a certain cost which should be minimized to balance safety with cost. Therefore, an optimization process should be performed where the objective function must include the initial cost of the work and the cost of the potential damages and other consequences in case a failure occur.

By recognizing the uncertainties inherent in the design process, especially the seismic hazard, it has been proposed, (Frangopol et al, 2001), to appraise bridge performance by using the expected life-cycle assessment.

In the offshore technology (Stahl, 1986) the expected life-cycle cost E[Ct], is expressed in terms of the initial cost Ci and the expected failure/damage cost E[Cd].

The acceptable (optimal) failure probability may then be calculated by minimizing the expected life-cycle cost respect the failure probability.

From well known structural reliability theory, the bridge reliability is calculated, (Ang and Tang, 1984).

The structure is a vehicles bridge built on the Benito Juarez International airport area, in the transition seismic zone III, in order to improve the traffic conditions. The bridge has a 400 m total span divided into 16 segments of 25 m each. The structural modeling was made through a finite element-based commercial software (RAM Advanse, 2006).

A family of bridge designs were obtained (AASHTO, 2002) by varying the original design dimensions and steel areas. These designs allowed for a series of alternative designs to measure the variation of reliability with cost under specified seismic intensities. The bridge designs were analyzed under given maximum seismic coefficients c/g, using the typical spectral form for Mexico City, and according to the range of intensities as reported in Mexican seismic hazard and failure rates studies (Esteva and Ruiz, 1989).

All the curves in the family shown in Fig. 1 are conditional to the ocurrence of the indicated intensity. In order to obtain the unconditional curve, the ordinates of the conditional curves need to be weigthed by the ocurrence probabilities according to the seismic hazard curve for Mexico City.

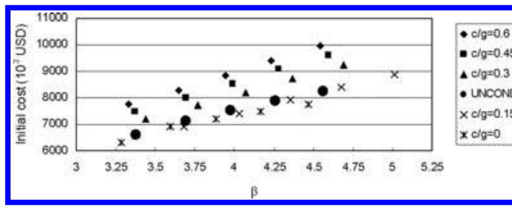

Figure 1. Family of conditionals and unconditional initial cost curves for a bridge on the zone III, Mexico City.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Seismic vulnerability of free toll bridges in Michoacan state, Mexico

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ABSTRACT

In Mexico, the free toll federal highway network has a length of more than 45000 Km and it is being operated by the Communications and Transportation Secretary throughout the General Direction of Highways conservation. Usually, highways bridge design follows the AASHTO seismic specifications which, before 1971, were based on the SEAOC buildings lateral force requirements but, after the San Fernando earthquake, CALTRANS code (1973) established new seismic design criteria. These criteria set the basis for the AASHTO code from 1975 to 1992.

However, and in spite of the fact that Michoacán State is located on a potentially seismic zone, many bridges there have been designed without any seismic consideration. Michoacán State has 485 highway bridges and it is the Mexican State with most of the bridges in the country. More than half of these bridges were designed between 1940 and 1965, with a structural type and design loads typical for those years. In addition, increased traffic demand and physical deterioration makes mandatory an extensive revision and generation of cost-effective strategies to protect and maintain those bridges.

In this paper, a bridge classification based on construction year, substructure type, number of spans, viaduct width and approaches and substructure material is presented. In addition, their epicentral distances to the near seismic faults, are considered as a simplified measure of seismic risk. Fundamental vibration periods and the influence of the superstructure concrete modulus and, in case it applies, masonry mechanical properties are also taken into account to state the seismic vulnerability of the bridge system.

Visually inspected damages are used for a preliminary assessment of safety conditions and repair/maintenance actions are proposed for each class of bridge importance and damage level.

In the future, a probabilistic damage assessment and life-cycle economic consequences appraisal will serve to improve the evaluation procedures and to set optimal maintenance strategies.

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New technologies to rehabilitate an old Mexican pier in a harsh environment

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ABSTRACT

A 70-year old concrete pier (a bridge-viaduct with 2 km long, 9.5 m wide, and 146 concrete arches), constructed with stainless steel bars, and exposed to a harsh tropical marine environment (>C-5, ISO), has shown good performance during its service life. Visual and detailed inspections have shown structural deterioration symptoms, but little corrosion was found on the stainless steel bar. Several studies performed on the old pier during the past five years, and using recent technology advances in bridge monitoring, have shown structural degradation due to the increase of load magnitude and frequency, mainly longitudinal cracks on some of the 146 arches. The paper describes the historical information obtained from the pier construction procedure and detailed inspection tests performed to the pier. Brief discussion about durability issues on this pier, based on the inspection results, and the rehabilitation procedure performed, to increase the service life of this 70-year old structure, is given in this paper.

Figure 1. Picture of the pier during its construction.

Figure 2. Recent picture of the pier.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)
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Corrosion damage evaluation and diagnosis of bridges in the Mexican highway network

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ABSTRACT

This work presents an innovative and original method to determine the degree of corrosion in bridges built in the Mexican Federal Road Network (MFRN), as a function of environmental factors. These factors include prevailing climate where the bridge is located, accurate bridge geographic positioning (e.g. coastal line, river distance) and its distance from industrial corridors. This information was obtained and analyzed using the last generation of Global Positioning Systems (GPS), incorporating the current structural data produced during the last bridge evaluation available by the national bridge administration management program (SIPUMEX). This work present the results obtained so far of the corrosion damaged bridges in the Mexican Federal Bridge Inventory.

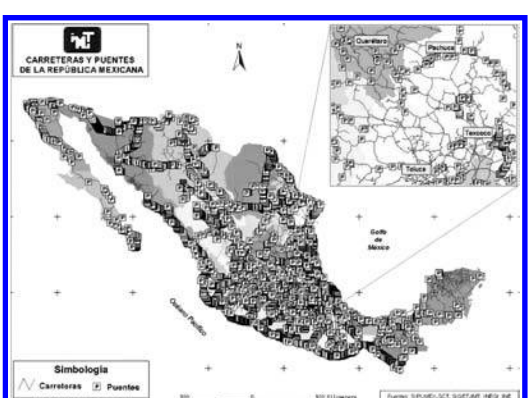

Figure 1. Inventory of all concrete bridges in the MFHN.

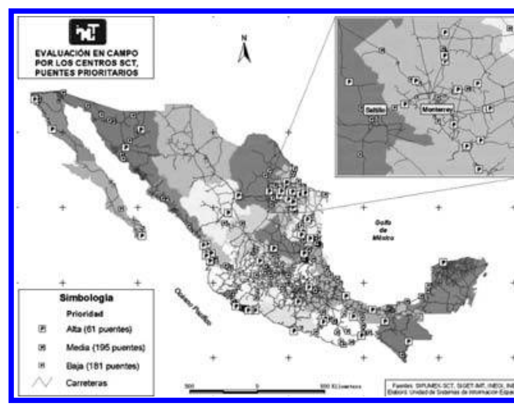

Figure 2. Map of chosen bridges for immediate inspection.

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MS17: COWI Group mini-symposium – Cable supported bridges

Organizer: J.S. Jensen

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Fatigue monitoring systems Great Belt Bridge

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ABSTRACT

The Great Belt Bridge is a major Suspension Bridge, located in Denmark. The main span of the bridge has a length of 1624 meters and carries 3 lanes in both directions. The bridge girder is a closed steel box girder with an orthotropic steel deck. The suspenders of the cable system are locked coil cables with a spacing of 24 meters and a maximum length of 180 m.

Since inauguration of the Great Belt Link in June 1998 the traffic growth has been significantly above the forecasts and in the same period large vibrations of the suspender cables has been discovered. In order to monitor the effect of the increased traffic on the fatigue capacity of the orthotropic steel deck, and monitor the effect of the cable vibrations on the fatigue capacity of the suspender cables, a structural health monitoring system was set up. This paper will present the results from the structural health monitoring system.

ORTHOTROPIC STEEL DECK

For year 2007 it was expected that the average daily traffic (ADT) would be more than 27,000 vehicles, and according to the initial forecasts this level of ADT should not be reached before year 2025.

The extraordinary traffic growth and in particular an increased share of heavy goods vehicles, has called for a reassessment of the fatigue capacity of the orthotropic steel deck and in particular of the welded joints of the trapezoidal stiffeners and the trough-to-deck plate welds. The results of various load tests and monitoring programs carried out during the period 2003 to 2007 will be presented in the paper. Further the paper will highlight the influence of the temperature of the asphalt pavement on the steel/pavement composite effect.

SUSPENDER CABLES

The large cable vibrations, with amplitudes on up to 2 meters for the longest suspender cables, was evaluated to cause fatigue failure in the locked coil cables at the upper socket entrance after a few years. A monitoring program was therefore set up in order to get more detailed information about the size and number of vibrations, and hereby be able to carry out a more detailed evaluation of the impact on the fatigue capacity. The paper will present the result of the cable vibration monitoring programme.

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Dehumidification of suspension bridge main cables

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ABSTRACT

Corrosion of main cables on suspension bridges is a major problem on a worldwide basis. It has been proven by experience that the traditional system developed about 130 years ago for corrosion protection of main cables generally does not provide sufficient protection. Very serious corrosion has been detected on the main cables of older bridges and somewhat surprisingly on younger bridges as well. In order to eliminate this serious problem, proven dehumidification technology has been further developed for application to suspension bridge main cables. The dehumidification method ensures that the atmosphere inside the main cables is kept sufficiently dry so that corrosion can not occur. The full paper includes a history of dehumidification, a description of systems for main cables, design considerations, information on the latest trend – integrated systems and information on the current status of existing systems.

The main principle of dehumidification is that steel does not corrode when the relative humidity (RH) is below 40%. Between 40% and 60% corrosion can occur, though at a very low rate. When relative humidity exceeds 60% the rate of corrosion increases dramatically.

Corrosion protection of steel bridge structures by dehumidification has been practiced for about 40 years. This started on the Little Belt Suspension in Denmark, where dehumidification systems were installed in the box girder and the anchor chambers during construction in 1965 to 1970. Since the Little Belt Bridge, dehumidification has spread to other bridges, first in Denmark and then worldwide. It has virtually become a worldwide standard to apply dehumidification to bridge box girders and anchor chambers, as it is recognized as the most effective and economical means of corrosion protection.

Dehumidification systems for main cables are composed of the following three main components:

- A dry air system capable of producing and blowing dry air through the main cables.
- A sealing system for the main cables, including cable bands, saddles and other connected components.
- A control and monitoring system.

These components are designed as an integrated system to suit the individual bridge and fulfill the specific requirements.

A dehumidification system for main cables has so many clear advantages as compared to a traditional system that it is obvious that it should be chosen instead of traditional system. These advantages include virtually 100% safety against corrosion, low construction cost, short construction period, maintenance friendliness, low maintenance costs, excellent documentation for effectiveness, environmental friendliness and good health and safety for workers.

The latest trend in dehumidification systems for bridges is integrated systems, i.e. systems that incorporate as many steel bridge elements as possible in one integrated system. An integrated system could for example provide corrosion protection for the bridge box girder, the towers and the main cables with just one dehumidification plant in a tower or a box girder. Several such systems have already been designed and will soon be constructed.

The current status for main cable dehumidification systems includes a total of 28 suspension bridges with completed or planned dehumidification of main cables in a total of 11 different countries, including 20 systems in service.

As the main cables of a suspension bridge are virtually irreplaceable, it is essential that they are protected from corrosion by the best means available. A dehumidification system is in all regards the optimal method for protecting main cables from corrosion. This is the only system which completely prevents corrosion, whereas other systems at best can only slow it down. Suspension bridge owners should take advantage of this technology and make plans to protect their cables and bridges for many generations to come.

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Imagination, good engineering and saving money

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ABSTRACT

Imaginative but soundly based engineering can lead to considerable cost savings over the long term for owners and operators of major bridges. Examples are given of such savings engineered by the Authors' company for a variety of bridge owners. The authors have used innovative engineering to proactively address both rehabilitation and maintenance issues on a variety of bridges, always with an eye on keeping bridge or lane closures to a minimum in order to minimize impacts on bridge users and/or maintain toll revenues as high as possible. The goal has always been to develop good engineering solutions to minimize both rehabilitation capital costs and ongoing maintenance costs.

Buckland & Taylor Ltd. has pioneered "structural health studies" of suspension bridges, whereby the bridge as it was built is modeled in the computer, and all known changes to the bridge since it was built are incorporated into the model. The bridge is then surveyed (at night during an overcast sky with no wind) and the profile of the bridge as surveyed should match the profile predicted by the model. In virtually all of the twenty suspension bridges treated this way, the surveyed profile has not matched the predicted. This means that something was not modeled because it was not known. The detective work then begins to discover what has happened that was not known, and is it serious? An early example of this was the discovery of stretch of the main suspension cables made with structural strands.

The Canadian Highway Bridge Design Code has introduced the ability to safely reduce load factors on the basis that more is known about an existing bridge than is the case with a bridge still on the drawing board, and not all things have to be equally safe. This can provide considerable cost savings.

Examples are given of rehabilitation projects where innovative but careful engineering has provided cost savings to the owner/operator.

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When major bridges need to carry more and heavier loads

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ABSTRACT

As traffic loads increase, and more lanes of traffic are required, what can be done – at an affordable cost – to increase the load carrying capacity of major bridges that are already in service? Some of the key issues are discussed, and examples are given of some original methods of increasing the load capacity of major bridges, that have been proven in practice.

A key element is the need to keep traffic flowing during any alterations, both for the benefit of the public and, if applicable, to maintain income at the toll booths.

And if alterations are to be made, a perfect opportunity presents itself of improving access for inspection and maintenance.

Examples are given from the authors' experience, and all have been executed successfully. They encompass suspension bridges, cable-stayed bridges, arch bridges, truss bridges and steel plate girder bridges, with concrete, steel and grating decks. Some were smaller bridges that can be regarded as prototypes for larger bridges.

It is emphasized that the key ingredients for success are a clear understanding of structural behavior, careful study of the expected traffic loads, derivation of suitable load factors, and, above all, creative thought processes applied with great care. High-powered analysis, while often essential, should serve the process, not drive it. Constructibility and practical considerations are more important.

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Structural health monitoring for Bosphorus Bridge

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ABSTRACT

The Bosphorus Bridge in Istanbul, Turkey, is one of the original streamlined steel box girder bridges. The bridge spans 1074 m between Europe and Asia over the Bosphorus Straits, an important shipping channel between the Black Sea and the Marmara Sea. The bridge has now been closed to heavy good's vehicles, however it currently serves 200,000 vehicle passages on a daily basis.

In 2004, during strong winds and icy conditions, a hanger stool near to one of the towers failed in fatigue. It had also been noted that a number of hangers were slack. The stool failure and the slack condition of a number of hangers prompted a review of the structure. Review into the failure of the hanger stool identified the primary cause as fatigue resulting from wind-induced vibrations. Appropriate mitigation measures were thus installed on the hangers.

Consequently, KGM sought advice on the state of the bridge and commissioned Flint & Neill to carry out a detailed assessment of the structure, a position survey of the towers and decks and a full inspection of the primary structural components. From this information, Flint & Neill developed a structural health monitoring system for the bridge with the purpose of providing KGM with information on the operation and condition of the bridge and providing vital health monitoring information in the event of a major earthquake or wind storm.

The volume of traffic restricts the bridge owners ability to close parts of the bridge to traffic, thus limiting the ability to perform maintenance and repair operations. The sheer weight of traffic reduces the life of many of the replaceable components, placing some elements at risk of premature failure.

The paper identifies the requirements for the system and the development of the design. It looks at the client's primary concerns and how these were dealt with by monitoring. The system build and operating constraints are described along with the client's longer term monitoring objectives.

The facility is expected to become a vital tool to the owner of the bridge in developing maintenance strategy as well as providing rapid-response structural assessment of this lifeline following extreme loading events such as earthquakes. The Structural Health Monitoring System presents a succinct tool to assist with the management of the structure.

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Bridge hanger deterioration

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ABSTRACT

The design of modern suspension bridges recognises that the deck hangers have a finite life, and one that is likely to be less than the target design life of the bridge. The ability to replace hangers in service therefore becomes a design case. This is a relatively new design constraint, specified in documents such at the Eurocodes[1]; for older bridges the replacement of deck hangers becomes a major maintenance exercise.

The performance of bridge hangers has shown that design lives of 40 years have been the norm on bridges built prior to the 1980s but that modern bridges should have hanger life expectancies greater than this. The paper opens with an examination the problems that have arisen in the performance of deck hangers and ways in which maintenance strategies can be developed to extend the design life of hangers. Steel wire hangers are susceptible to corrosion and fatigue with damage manifesting itself both externally, which is easy to spot, and internally, where detection becomes critically in ensuring the safety of the structure.

The inspection of hangers is therefore a critical issue, where there is a desire to understand the condition of the hangers and the remaining life of damaged hangers. Methodologies such as magnetic flux, x-ray and magnostriction systems are examined to assess their effectiveness in determining corrosion and wire breaks

The effectiveness of different protection strategies such as galvanising, internal blocking and external sheathing is investigated based on cases studies of hangers thought out Europe, America and Asia.

Finally a review of hanger replacement strategies is provided, showing methods of construction and the risk mitigation processes required to deliver a contract successfully. Often the enabling works to gain safe access require as much effort as the hanger changing itself.

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Bridge barriers and parapets

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ABSTRACT

The design of bridge barrier systems on long span bridges must strike a balance between providing safe containment of traffic while achieving an economic cross section with low transverse wind load demands. The problem exists as a solvable design problem in new bridges, where flexibility exists in striking such a balance and the economies of delivering all-year round wind protection can be explored. But with existing structures built without consideration of traffic impact effects, the consequences can be more significant.

The paper looks at existing vulnerabilities in bridges where steel towers, cables systems and overhead gantries are at risk from impact from heavy goods vehicles and the methods employed to assess the risk of damage and loss of service to the bridge as well as the methods of providing protection to the vital components.

The work follows a number of assessments of bridges in Europe and Australia where earlier designs failed to recognise that the greatest vulnerabilities to loss of service and indeed complete collapse arise from errant vehicles. The results of these studies are presented along with the methods of retrofit that have been employed. In some instances the addition of safety fences can be extended to include wind shielding around sharp obstructions, thereby improving the operational efficiency of the bridge in high winds.

Chief among the design issues are the need to provide sufficient containment within restricted carriageway widths and the ability to disperse large impact forces into thin plate deck sections. The design of such systems must also look at the effect of impacts from light vehicles to ensure that a greater hazard has not been introduced.

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Surfacing for orthotropic bridge decks

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ABSTRACT

Due to increasing traffic demand, the need to find improved surfacing systems for the UK's stock of lightweight steel box bridges arose in the mid 1980s and continues to the present day. These structures, built during the 1960s and 1970s were originally provided with a thin (38 mm) mastic asphalt surfacing system and rubberized waterproofing membrane.

The resurfacing of five major U.K. bridges with orthotropic steel decks is described, giving the background to the research into replacement systems and their development towards enhancing the life of the underlying steel deck. Following the research, a standardized method of system evaluation was developed for tendering purposes.

The paper describes practical aspects of system application, including deck inspection and repair. With reference to research findings and experience gained on a number of bridges, the paper will provide useful information on specifying and overseeing resurfacing contracts with long-term performance firmly in mind.

Two markedly different system types are discussed; a development of the original UK mastic asphalt system incorporating lake asphalt and an all-epoxy system. Conclusions are drawn on the effectiveness of the systems from the point of view of asphalt durability, enhancement of the performance of underlying steelwork and the reliability of the application process.

Many papers on surfacing composite action and fatigue in orthotropic decks have been published over the years, whilst others have covered the design of specialized asphalts; it is hoped this paper will help to focus on tying these issues to effective technical delivery

The paper addresses both design issues and practical considerations for the owner, specifier and applicator; however, lessons learnt extend beyond the cited systems.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Bridge bearings and joints

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ABSTRACT

Many years of experience of design and maintenance of bridges shows that movement structures are among the more expensive as far as maintenance costs are concerned. These are mechanical structures and require regular maintenance, or part or full replacement. The design philosophy should therefore be based on: As few as possible movement structures, long durability, easy to maintain and replace. This paper describes:

Typical articulation systems on major suspension bridges and minor urban cable stay bridges are described among others how wear and durability problems can be overcome.

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Replacement of suspension bridge suspenders and main cables

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ABSTRACT

Projects for replacement of suspenders and/or main cables are described. The Älvsborg Suspension Bridge crosses the Göta River at the entrance to Gothenburg in Sweden and is an essential link between the city and the industrial area to the north. The inspections, investigations and condition history of the suspender cables leading up to the replacement and the replacement are described. The Aquitaine Bridge crosses the river Garonne at Bordeaux in France. The total suspension system, main cables and suspenders, were replaced. The new main cables are protected from corrosion by dehumidification. At the same time the deck was widened to include six lanes instead of four. The rehabilitation work was executed with passing traffic of about 70.000 cars a day. Two examples are given of taking advantage of suspender replacement to gain other benefits. In the case of the Lions' Gate Bridge temporary suspender extensions were used to control stresses during reconstruction of the bridge, and in another case the opportunity arises of permanently increasing the vertical clearance for shipping.

MS18: Lifetime design of super long span bridges
Organizer: H.-M. Koh

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The prediction of long term operation and maintenance costs of long span bridges

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ABSTRACT

More than 40 years of operation and maintenance experience of major structures is hereby presented as costs on electric and mechanical components and structural elements on the structures. A number of structures is represented such as suspension, cable stayed, bascule and cantilevered concrete bridges.

Analyzing the data gives a clear picture of the distribution of both operation and maintenance costs as function of time. This distribution is depended on the type of structure and the quality of the components used in the structure.

The overall importance is the investment and the service life (performance) of the said component or structural element. For the components and structural elements it is possible to assess their service life and a list of experience based service life is set up.

The overall picture is very similar for each type of the bridges; namely there will within the first 10 years of service life be a limited level of maintenance costs. This level will raise and reach maximum after approximately 30 years depending on the type of bridge and the structural material. After 30 years the costs again will deviate between a relative high level and the maximum level depending on the repetition in replacement and reinvestment.

Based on insight in unit costs, the actual bridge and the service life of the components and structural elements, it is possible to predict average costs for operation and maintenance with time. Examples on the 30-60 years budget will be given.

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Deterministic and probabilistic durability design methods and their application to super long span bridges

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ABSTRACT

Many concrete structures including breakwaters, tidal barriers, offshore drilling platforms, have been constructed in marine environment. In Korea of nowadays, a number of long span bridges have been constructed and are also being built in marine environment as shown in Figure 1. Corrosion of reinforcement reduces the service life of concrete structures, thus durability design against chloride attack is needed to guarantee a long service life of the concrete members of super long span bridges such as concrete pylons, piers and footings.

Durability design approach for concrete structures in marine environment is divided into two categories: deterministic and probabilistic approaches. Simple deterministic durability design method against chloride attack is to calculate the service life using so-called error-function solution with respect to the Fick's second law of diffusion. This method is simple and relatively easy to use. A more sophisticated way to consider the changes of environmental temperature and surface chloride concentration is to use numerical solutions of the Fick's second law of diffusion. Life-365 service life prediction model is an example of this approach.

Deterministic durability design approach, however, can not consider the variability of parameters concerning the chloride penetration. On the contrary, probabilistic method, such as DuraCrete methodology, can be used in the durability design. This method uses the error-function solution of chloride penetration and considers the variability of parameters.

In this paper, the fundamentals of chloride diffusion are dealt first. Two approaches of durability design are presented and some design cases of long span bridges in South Korea are introduced. Diffusion coefficient is dealt next. An improvement of diffusion coefficient model is made based on the consideration of test method and mathematical expression. For probabilistic design, first and second order reliability analysis methods are applied to calculate the probability of corrosion initiation in order to compare the accuracy of results.

Figure 1. Long span bridges in Korean seaside.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Analysis and comparison of recent bridge failures in China and the rest of the world

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ABSTRACT

As the most important requirement for bridge sustainability, structural safe reliability attracts much concern of bridge engineers. Although many efforts have been put on the related scientific research and technology development, recent bridge failures reveal that the efforts are still not enough. This study is thus motivated to investigate bridge failures that have occurred in China and the rest of the world recently, i.e., the period between 2001 and 2009. According to information collected, the types and causes of bridge failures is analyzed in this study.

Collective studies conducted for bridge failures revealed that 37 and 39 bridges of various types failed in this period in China and the rest of the world, respectively. The age of the failed bridges ranged from 0 year (during construction) to 68 years in China and 140 years in the rest of the world. Based on the information collected, the following phenomenon is observed: 1) beam/girder bridges are most frequently failed since they are most popular bridge type; 2) arch bridge failures occurred disproportionally high in China compared with the rest of world, some further detailed study is thus required to tell the reason; 3) the most frequent causes of bridge failures were attributed to construction and maintenance, and these two causes constitute 37.8% in China and 35.9% in the rest of the world, of the total bridge failures, which tells that the more efforts of bridge engineering community should be put on the study related to bridge construction, maintenance and dismantle, the process of bridge design can meet the nowadays bridge construction requirement; and 4) the most frequent triggering causes of bridge failures were attributed to overloading and ship collision, which is expected to be avoided by strict vehicle and ship management.

ACKNOWLEDGEMENT

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Table 1. Causes of Bridge Failures.

		Number of collapses		Percentage	
Principal causes	Specific causes	China	World	China	World
Enabling	Design	1	2	2.7	5.1
	Detailing	1	0	2.7	0
	Construction	8	9	21.6	23.1
	Maintenance/dismantle	6	5	16.2	12.8
	Material	1	0	2.7	0
Triggering	Overload	9	5	24.3	12.8
	Collision	3	2	8.1	5.1
	Flood	3	3	8.1	7.7
	Strong wind	0	2	0	5.1
	Explosion	0	1	0	2.6
	Fire	0	1	0	2.6
	Landslide	1	1	2.7	2.6
	Terrorist	0	1	0	2.6
Unknown		4	7	10.8	17.9
Total		37	39	100	100

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Terminology for treating disproportionate collapse

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ABSTRACT

Disproportionate collapse is a complex problem for which the existing terminology and procedures are inadequate. A multitude of terms is used to describe structural characteristics and concepts in the context of disproportionate collapse. Some of them, namely collapse resistance, structural robustness and vulnerability are discussed in this paper. This paper distinguishes these terms and the associated structural characteristics, and suggests working definitions.

A disproportionate collapse is characterized by a pronounced disproportion between a relatively minor event and the ensuing collapse of a major part or the whole of a structure. Disproportionate collapse is prevented by ensuring collapse resistance, a property defined as the insensitivity of a structure to abnormal events. An abnormal event is an event that is unforeseeable or occurs with very low probability and is not considered in the ordinary design of a structure. Collapse resistance can be achieved by reducing the exposure of a structure or by reducing its vulnerability – two measures that aim at preventing failure initiation – or by increasing its robustness – a measure that aims at preventing failure progression.

The exposure results from the abnormal events that possibly affect a structure during construction and lifetime. The vulnerability of a structure is its susceptibility to suffer immediate initial damage in case it is affected by abnormal events. Robustness is the structure's insensitivity to such initial damage.

The basis of current reliability-based design codes for general structures is reviewed with particular regard to their suitability to prevent disproportionate collapse. Their inadequateness regarding the prevention of disproportionate collapse is outlined.

To give immediate guidance to the practicing engineer, a pragmatic design approach is proposed in which probability-based design procedures as described in the codes are complemented by an additional assessment and design measures with particular regard to disproportionate collapse.

A performance-based framework for designing against disproportionate collapse – applicable to any kind of structure, and thus also to bridges – is presented A set of design criteria is presented. These include design requirements, design objectives, design strategies, and verification procedures.

Design requirements are the specification of whether collapse resistance is required for a structure and, if yes, the specification of the level of requirements on design objectives, design methods, and verification procedures. The requirements depend on the significance of the structure with respect to the consequences of a collapse and on the structure's degree of exposure to abnormal events. Collapse resistance will not be required for every structure.

Design objectives are the basis of a performance-based design. They comprise hazard scenarios, performance objectives, and applicable combinations of actions and safety factors. Hazard scenarios are the abnormal conditions to be assumed in the design to affect the structure during construction and lifetime. In a threat-specific approach, they are specific abnormal events. In a non-threat-specific approach, they are notional actions or notional damage, without regard to the cause. Performance objectives specify the acceptable response of the structure to the hazard scenarios.

Design methods are methods that enhance the collapse resistance of a structure; available methods are event control, protection, increased local resistance, alternative load paths, and segmentation.

Verification procedures are used to demonstrate that a structure meets specified performance objectives when subjected to specified hazard scenarios. Verification procedures of varying degrees of accuracy can be performed, which are selected depending on the requirements and the design objectives.

Codification is moving towards such an approach already. This paper shows how these efforts can be continued to lead to a clearer description of design criteria and to a precise and consistent use of language.

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Identification of aeroelastic parameters for cable-supported bridges using measured accelerations

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ABSTRACT

After the flutter derivative-based aeroelastic formula had been by Scanlan, great number of efforts have been made to estimate the flutter derivatives from the test of bridge model in wind tunnel. Scanlan and Tomko proposed the extraction scheme for flutter derivatives from 2DOFs coupled motion tests (1971). Sarkar developed the Modified Ibrahim Time Domain (MITD) to estimate cross flutter derivatives along with direct flutter derivatives (1994). The procedures of these approaches are generally based on the response error estimation, which minimizes the relative error between measured displacement and predicted displacement.

However, the flutter derivatives are conceptually more closely related with the aerodynamic force equilibrium than with the predicted displacement itself in formulas. Therefore, this paper proposes a new approach to identify flutter derivatives based on the equation error estimation (EEE) which minimizes the relative error between the resisting forces (stiffness, damping and inertia) and the aeroelastic self-excitation forces.

The EEE approach requires not only displacement response but also velocity and acceleration history for system identification. In this approach, a displacement and velocity reconstruction scheme is used to calculate displacement and velocity history from measured acceleration. Hence, both these reconstructed responses and the measured acceleration is used for EEE method.

The validity of the proposed method is demonstrated through the experimental free vibration test. The flutter derivatives of a section model representing a bridge deck system are evaluated using the measured acceleration and reconstructed displacement and velocity using proposed method, and compared with the result of the prediction error minimization method (PEM) with measured displacement.

Though EEE results in less compatible results between predicted and measured displacements than PEM, the authors believe that the flutter derivatives identified by EEE contain more physically reasonable meanings based on the governing equilibrium equation. Moreover, the EEE is free from the convergence of ill-posed problem, and give very stable identification results.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Reliability-based durability design and quality control of long span bridge of Busan-Geoje Fixed Link project in Korea

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ABSTRACT

The Busan-Geoje Fixed Link comprises a 8.2 km motorway link from Busan, Korea's southernmost and second largest city, to the island of Geoje. The connection includes two cable-stayed bridges, respectively 1.6 and 1.9 km in length and a 4 km immersed tunnel with 50 metres water depth. The project is scheduled for completion in 2010. DAEWOO E&C is the leading contractor.

The durability design criteria for the Busan-Geoje Fixed Link – Bridges and Immersed Tunnel specify that the contractor must verify that the concrete in combination with the selected concrete cover – will prohibit initiation of corrosion during the design life (100 years) with a probability of 90%.

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. Durability design using DuraCrete was fufilled by the main designer of Busan-Geoje Fixed Link project, COWI. Concrete mixes determined by pre-testing showed that they fully satisfy the criteria suggested by DuraCrete design.

For concrete with a good performance for 100 years, the cracking of concrete which gives an effect to the durability of concrete should be controlled. Therefore most of specifications have the requirement of the allowable crack width. Many cracks however take place in early aged concrete by restraint to the volume changes due to combinations of e.g. hydration heat, thermal movements, early shrinkage, creep and settlements. Thus it is difficult to control the crack width because the evaluation of the strength and stress of early aged concrete is not simple. Until now no method of calculating the crack width in early-aged concrete is suggested.

But in Busan-Geoje Fixed Link project cracking of early-aged concrete was controlled by calculation of crack widths based on the stresses through hydration heat and stress analysis. Crack inspection showed that cracking in early-aged concrete can be controlled by calculating the crack width.

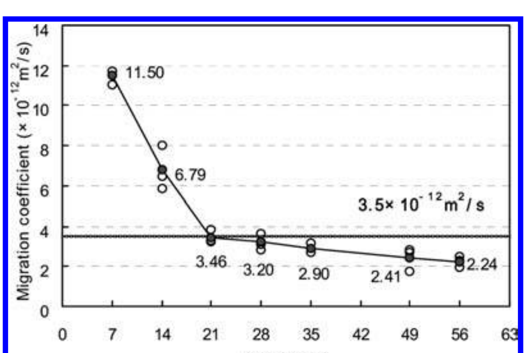

Figure 1. Chloride diffusivity coefficient of core taken from actual structure.

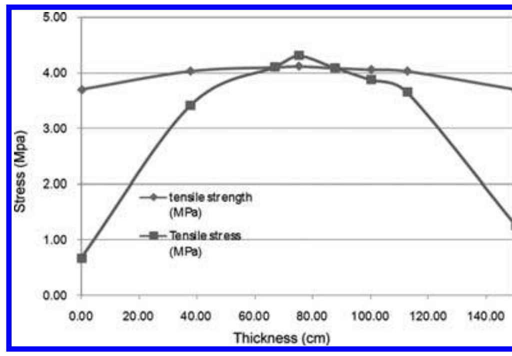

Figure 2. The distribution of tensile stress and strength in pylon wall which is used to calculate the crack width.

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Field surveying and FE model updating of a suspension bridge in service

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ABSTRACT

A suspension bridge is a structure that sustains dead and live loads through a main cable and stiffening girders or trusses. The main cable is a main structural member for a suspension bridge and, as a result, the introduced tension in a main cable has to be close to the designed one. However, the tension of the main cable can only be estimated indirectly by measuring the sag. Since the structural parameters of an as-built bridge show some variations, the estimating procedure should utilize every possible informations provided from field. This study proposes a technique for the establishment of an analytical model based on field-measured data and applied to the Gwangan Bridge in Korea, to propose an analytical model reflecting the as-built state of the bridge for the purpose of maintenance. The exploited field-measured data are surveyed coordinates of the target points located in the main cables, stiffening trusses and towers, thermal measurements at the time of survey, tensions of the hangers, and measured dead loads during construction.

Fig. 1 shows the measured sag and length of center span of the Gwangan Bridge.

An ambient vibration test has been also carried out to identify the dynamic characteristics of the bridge. Based on the identified natural frequencies, a FE model updating procedure has been applied to enhance the dynamic reproducibility of the model. The FE model updating produced closer results in terms of the frequencies (Table 1 & 2). However, the structural parameters have undergone greater change than expected. For this reason, a further consideration is being followed in the view point of manual tuning of a structural system.

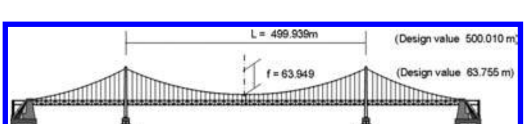

Figure 1. Measured sag and length of main span.

Table 1. Selected structural parameters.

Parameter	Upper/Lower Bounds	Weighting factor
Area lower chord	±15%	1
Area upper chord	±15%	1
Area diag. member	±15%	1
Area of K truss	±15%	1
Area of main cable	±5%	3
Tension of cable	±5%	1
Mass of main cable	±5%	3
Mass of Stiff. truss	±10%	2
Rot. mass of Stiff.	±10%	2

Table 2. Natural frequencies of baseline and updated model.

Vibration Mode	Identified from AVT	Baseline model		Updated model Unbounded		Updated model Bounded	
		(Hz)	(%)	(Hz)	(%)	(Hz)	(%)
1st lateral	0.1460	0.115	−21.48	0.125	−14.23	0.125	−14.38
1st vert.	0.2435	0.224	−8.21	0.244	0.02	0.241	−0.90
2nd vert.	0.2519	0.235	−6.73	0.252	0.03	0.253	0.00
3rd vert.	0.4630	0.398	−13.97	0.432	−6.62	0.435	−6.13
1st torsion	0.6227	0.576	−7.52	0.623	0.02	0.614	−1.45

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Forth replacement crossing – design for safe maintenance and management

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ABSTRACT

The Forth Replacement Crossing will be built across the Firth of Forth in Scotland to maintain and enhance a vital transport link in the country. The wide estuary will be crossed by a 3-tower cable stayed bridge supporting a pair of navigational main spans each 650 m, and an approach viaduct. The scheme design of the crossing, completed by the Jacobs Arup JV, aims to provide a fitting 21st century icon, standing alongside existing 19th and 20th century Grade A listed bridges. The specifications for the Design and Build contract to be let by the client, Transport Scotland, include enhanced measures to ensure durability and a full suite of access facilities to allow safe maintenance and management of the bridge.

DESIGN FOR DURABILITY

Durability is not an absolute property of a material but can be affected by both design and construction factors. To achieve a satisfactory working design life regular maintenance and some repair work will be required. Both the required protection (by durability design) and mitigation (by maintenance) have been assessed in order to ensure that the design life of 120 years can be achieved with a good degree of confidence. For the certain elements, where replacement is feasible and cost effective, a shorter service life as appropriate has been adopted in the design.

ACCESS FACILITIES

The main maintenance access entry to the bridge is at the south abutment. A secondary entrance is available via the north abutment.

The full length of the deck including the approach viaduct and cable stayed bridge is accessible by a pair of internal deck shuttles. Openings are provided in the top of the cable stayed bridge deck at regular intervals to gain access to the central zone between the carriageways.

The towers are accessible from the deck and an internal rack and pinion lift in each tower is the primary means to gain access above deck level with a stairway provided both below and above deck level to give internal access over the full height of each tower. Similarly the piers are accessible from within the deck and internal ladders / stairs provide access through the height of the pier shafts. Abseiling is envisaged to reach the external faces of the towers and piers. Suitable structural inserts will be provided in the external faces to aid abseiling.

Permanent moveable access gantries are provided below the deck to provide access to the deck soffit. Because of the configuration of the piers, some areas of the soffit will not be fully accessible from the gantries, so here the soffit will be accessed by lorry mounted underbridge inspection vehicles. It is also recommended that a stay cable inspection gantry is provided.

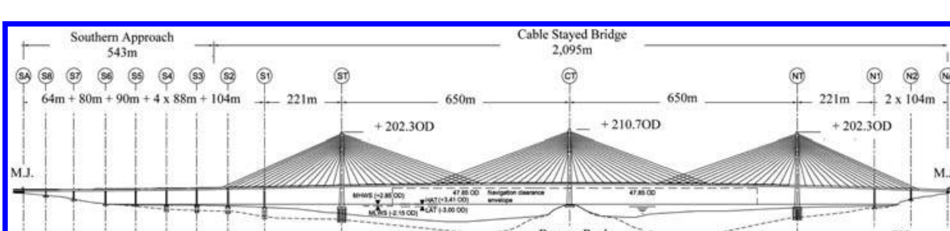

Figure 1. General arrangement – elevation.

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R&BD programs on the lifetime design technology for super long span bridges in Korea

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ABSTRACT

The need for sea-crossing long-span bridges in Korea has been recently revitalized by the ambitious plan of the government to link some of the 3,000 islands of the peninsula to the mainland. For example, the investment of ongoing projects intend to link some major islands of the southwestern coast with the mainland reaches approximately $10 billion until 2010 for the Province of Jeollanamdo. Additional investments for the construction of bridges are also foreseen until 2025 to promote economical and social balanced regional development all over the peninsula.

Thanks to this unprecedented bridge construction projects, remarkable technological accomplishments for long-span cable supported bridges have been achieved in Korea. In order to support such enterprise and sustain the next generation of bridges to be constructed in the peninsula, the Korean R&D community agreed with the necessity to develop and construct a new generation of high performance facilities by means of enhanced materials, advanced structural systems and technologies as well as upgraded or improved specifications or standards in a lifetime perspective.

Accordingly, the Korea Ministry of Land, Transport and Maritime Affairs (MLTM), formally the Korea Ministry of Construction and Transportation (MOCT), launched the "Program for 5 Years Plan for Construction Technology Innovation (2003∼2007)" to strengthen and systematize R&D programs as well as improve R&D management system.

This paper introduces recent research and business development programs for super long span bridges in Korea. Among the currently ongoing programs, Super Long Span Bridge center, a 7 year national R&BD program launched in 2009, is the largest single program dedicated to the development of cable-supported long span bridge technology. Overview and some details of the program are summarized with the emphasis on lifetime design technology (Table 1). Other important R&D programs are also presented.

Table 1. Super Long Span Bridge R&BD Program.

Center	Korea Expressway Corporation
Programs (institute)	1. Engineering Technology (Seoul Nat'l University) 2. High Performance Material & Application Technology (Research Institute of Industrial Science & Technology) 3. High Efficiency Construction Technology (Korea Institute of Construction Technology) 4. Test Bed & Operating Technology (Korea Expressway Corporation)
Sponsors	Ministry of Land, Transport and Maritime Affairs Korea Institute of Construction & Transportation Technology Evaluation and Planning (KICTEP)
Duration	7 years (2008.12∼2015.12) 3 stages (V3): Verification – Validation –Value Creation
Budget	94.6 billion KRW (about 80 million USD) Government (MLTM): 63.6 billion KRW, Private: 31 billion KRW

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Multi-scale system reliability analysis of bridge structures using dominant failure modes identified by selective searching technique

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ABSTRACT

Bridge structures often have innumerable potential system failure modes, i.e. combinations or sequences of local failures. For efficient reliability analysis of such complex redundant systems, many research efforts have been made to identify dominant failure modes with significant likelihoods, mostly based on the probabilities of failure modes found during an event-tree search; however, this approach is time-consuming due to repeated calculations of the probabilities of innumerable failure modes, which eventually necessitates using heuristic assumptions or simplifications.

Recently, a decoupled approach was proposed: dominant failure modes are first identified in the space of random variables without system reliability analyses, then an efficient system reliability analysis is completed to compute the system failure probability based on the identified modes. In order to identify dominant failure modes in the decreasing order of their relative contributions to the system failure probability, a simulation-based selective searching technique was developed by use of a genetic algorithm (Figure 1). The system failure probability is then computed by a multi-scale matrix-based system reliability (MSR) method that can account for the statistical dependence among the component events as well as among the identified failure modes. Lower-scale MSR analyses evaluate the probabilities of the identified failure modes and their statistical dependence. A higher-scale MSR analysis evaluates the system failure probability based on the results of the lower-scale analyses. A graphical representation of this multi-scale effect can be seen in Figure 2.

This paper presents this decoupled approach in detail and tests its applicability to complex bridge structural systems. The efficiency and accuracy of the method are demonstrated through comparison with Monte Carlo simulations. The results show that the proposed search method skillfully identifies the dominant failure modes contributing most to the system failure probability, and the multi-scale MSR method accurately evaluates the system failure probability with statistical dependence fully considered. Due to the decoupling between the failure mode identification and the system reliability evaluation, the proposed method is expected to be effective for larger structural systems.

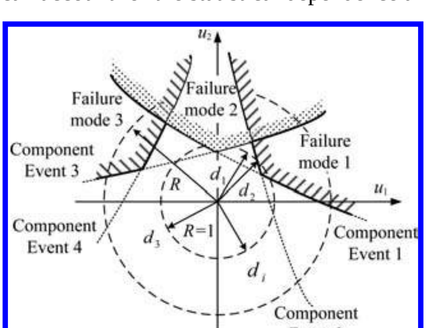

Figure 1. Three failure modes identified in the two-dimensional standard normal space.

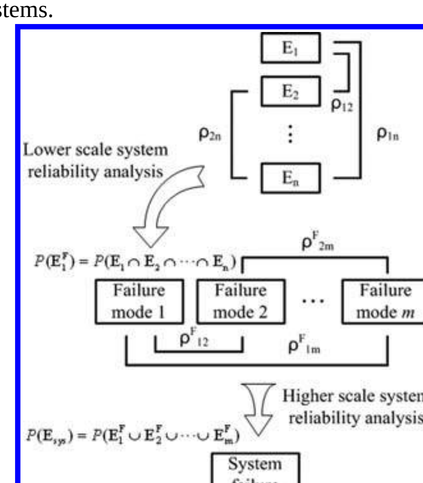

Figure 2. Hierarchical structure of the multi-scale MSR method.

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Elasto-plastic behaviors and ultimate strength of continuous super long-span suspension bridge

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ABSTRACT

In order to realize strait crossing bridge projects of the next generation, the development of new materials and new structural systems for super long span bridges, as well as economical, durable, rational design and construction method are required. From this background, the consecutive continuous structural system of a 3-span suspension bridge was considered first based on previous experience. However, this form requires the installation of an intermediate anchorage, and the number of tower foundations also increases, so this is not economical. Hence a multi-span suspension bridge is focused as an economical alternative.

At present, a wide range of studies has been reported as part of the planning for the straits crossing project, focusing on the structural characteristics, the economics, the methods on applying the live loads, the buckling characteristics of the central towers (O.Yoshida and T.Moriya 1997), etc., for 4- and 5-span suspension bridges.

In the research and development work as summarized above, there has been no studies investigating the elastic-plastic behavior or ultimate strength of the overall structure of multi-span suspension bridges (K.Nogami and M.Nagai 2002). Hence, in this study, analytical study was carried out to make clear the elastic-plastic behavior and ultimate strength of the overall structure of a 4-span suspension bridge having a central span length of 3000 m (K.Nogami A.Someya and T.Yamasawa 2006). This bridge employed a conventional single box section and a new structural type consisting of two boxes and the grating as the stiffening girder and of using 1770 MPa and new high-strength 2000 MPa cables.

The numerical calculation takes into account both geometric and material non-linearity. The materialstress-strain relationship used the incremental method in accordance with incremental plasticity theory. Also, the numerical analysis is adopted the Newton-Raphson method in combination with the incremental displacement method.

Table 1. Load factor at the ultimate state of LC2.

	Cross-section	
	Single box	Two-box + grating
Non initial imperfection, 1770 MPa	2.106	2.210
Initial imperfection, 1770 MPa	2.041	2.157
Initial imperfection, 2000 MPa	2.090	2.195

Table 1 shows the load factor at the ultimate state of loading condition LC2 which gives lowest ultimate strength based on the analysis results.

Since the dead load can be reduced by employing two-box and grating section, it is clear that the load factor at the ultimate state increases by 4∼5% compared to that of the single box section model. The load factor at the ultimate state of the high strength cable model increased by around 2% compared to that of the conventional cable model.

In four super long-span suspension bridge examined in this study, an application of the stiffening girder having the cross-section with two-box and grating and the high strength cable are effective to improve the load carrying capacity of the whole bridge system and can maintain the value more than 2.0 in conventional required load factor at the ultimate state. Hence, four super long-span suspension bridge is able to secure enough safety.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Probabilistic kernel principal component analysis for monitoring a suspension bridge under environmental variations

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ABSTRACT

Structural health monitoring (SHM) concerns with damage detection as well as damage assessment based on the obtained measurements. Since damage causes changes in the dynamic characteristics of the structure, it can be considered to be one of abnormality, which is detected by identifying those changes from the measurements and/or damage-sensitive features extracted from the measurements. When the structure under monitoring experienced abnormality, for example, installation of additional sub-structures, the damage detection strategies can also be applied to detecting the abnormality which is not caused by damage.

In reality, however, time-varying environmental and operational conditions can also affect the dynamic properties of the structure, and slight changes caused by abnormality may be blurred or undetected. Therefore, it is necessary to develop a technique to separate the effects of abnormality from those caused by environmental and operational variations.

Data normalization is a procedure to separate the effects caused by time-varying environmental and operation conditions from measured signal changes, so that possible abnormality can be detected in the presence of environmental and operational variation (Sohn & Oh 2008, Oh & Sohn 2009).

In this study, kernel principal component analysis (KPCA) incorporated with novelty index and generalized extreme value distribution is proposed to detect abnormality caused by installation of additional railways in the presence of environmental and operational variations. The proposed method is applied to an in-service Yeongjong grand bridge in Korea to investigate its feasibility. For the baseline and test data, hanger tensions measured on 2003 and 2004, respectively, are employed. KPCA characterizes the hidden relationship between measured hanger tensions and unmeasured environmental and operational variability, and abnormality caused by installation of additional railways is detected in a reliable fashion by incorporation of novelty index with GEV distribution.

10% of randomly selected baseline data are used for computing nonlinear principal components. Then, the entire baseline data are used to compute novelty index and the associated best-fit GEV distribution. A statistically meaningful decision boundary is established based on the best-fit GEV distribution. When a new data point is obtained from the structure that may experience abnormality, the corresponding nonlinear principal components are computed. Those nonlinear principal components are employed to calculate novelty index from the baseline data whose principal components have the minimum Euclidean distance from those of new data. Finally, the number of outliers whose values are beyond the pre-decided threshold is counted, and a sudden increase of the number of outliers is utilized to indicate the occurrence of abnormality.

This study demonstrates that application of a probabilistic data normalization technique, i.e., KPCA incorporated with novelty index and GEV distribution, to an in-field Yeongjong grand bridge demonstrates its feasibility to detect abnormality under time-varying environmental and operational conditions without explicitly measuring them.

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Integrated geometry control system for cable stayed bridge: Application to Incheon bridge

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ABSTRACT

The integrated geometry control system in the long-span steel deck cable-stayed bridge has been developed and applied to Incheon Cable Stayed Bridge. This system consists of 4 systems which are data integrated management system, structural analysis system, error adjustment system and measurement system. The data integrated management system stores the numerous analysis data and measured data into its system and shows them graphically to the engineer. It allows the engineer to understand and compare the necessary data more easily and quickly. The structural analysis systems is developed for the structural analysis with considering the geometric nonlinearity and modeling the cable as elastic catenary cable element. The error adjustment system is developed to analyze the differences between the designed values and measured ones of the geometry and cable tension and then to minimize the differences by adjustment of the cable length. The measurement system is combined to collect and show the required measurement data for the geometry control such as temperature, displacements of girder and pylon, cables tensions promptly. Each system is integrated and managed in order to combine the systems organically.

In incheon bridge construction, 4 numbers of cables in the side-span and center-span have been erected and adjusted at the same time along with the cantilevered deck erection in the center span under 6∼7 days of cycle time.

The integrated geometry control system integrated the analysis data by structural analysis program and measured data collected by wireless system at each construction stage. The engineer could analyze the data efficiently and make the proper decision for adjustment at an earliest time at each stage by using this system.

The integrated geometry control system made it possible to minimize the numbers of cable retension and controlled the final geometry and tension error within the target ranges.

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Redundancy analysis for a cable-stayed bridge using fibre model

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ABSTRACT

Tempozan Bridge is a 3-span continuous double-plane fan-system multiple-cable-stayed steel bridge with a centre span of 350 m and a total length of 640 m, as shown in Figure 1. The cables and some members are highly redundant, while damage to some other members like pendel bearings is highly likely to lead to reduced safety performance of the whole structure. To achieve a rational maintenance of a bridge within a limited budget with the life cycle cost and lifetime expected risk of the bridge taken into account, it is necessary to determine an optimum inspection program through comprehensive investigation on various factors including impacts of damage of individual members on safety performance of the whole structure and costs required for repair, strengthening and inspection.

For the purpose of determining limit states of a long-span bridge and causal phenomena engineeringly, the authors conducted static elasto-plastic finite displacement analysis. Sensitivity analysis was performed under incremental loads with a certain level of corrosion given to some members, and aging analysis was performed under constant loads with sectional area of individual members progressively reduced. The model used in the analysis was a global model consisting of shell elements for the main girder and fibre elements for the main towers and end piers, as shown in Figure 2. The analysis also used a technique of including the effects of local buckling in the material constitutive laws so that the dead load state was successfully reproduced with the effects received during the erection taken into account.

As a result, it was quantitatively demonstrated that damage in the main girder and cables had most significant impacts on the safety performance of the whole structure. The aging analysis (in Figure 3) revealed that influence of damage in the main girder or cables was significant on the vertical displacement at the centre of the centre span, although yield points were not reached at a corrosion of 9 mm.

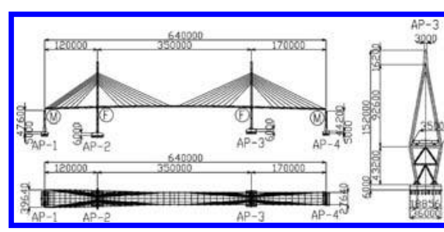

Figure 1. General view of the bridge under analysis.

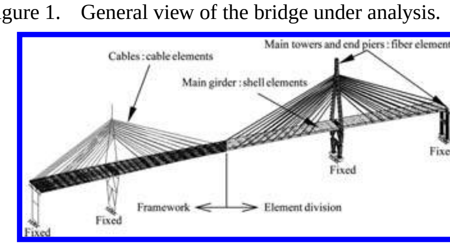

Figure 2. Analysis model.

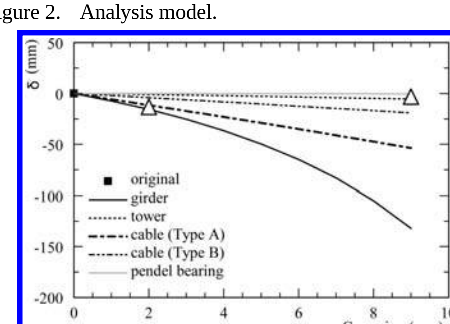

Figure 3. Aging analysis results (Vertical displacement at the centre of the centre span).

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Analytical study on detection of structural damages of a long-span suspension bridge by wind-induced response

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ABSTRACT

Nowadays, a great number of long-span bridges were constructed all over the world. Most of long-span bridges which are common over the sea are particularly difficult to maintain because of their specific conditions: a severe natural environment including strong winds, strong tidal currents and salt air, a large degree of continuing deformation of structures, an extremely large variety of structural members and materials, and the need to cope with the fatigue of structural steel especially in the case of any bridges which carry trains as well as road traffic. However, their service time have to be more than 100 years because these bridges are very expensive to design, construct and maintain. As a result, the health, durability, and safety of these bridges in a long-term service period are now attracting a lot of scientists and engineers. An issue arising will be a methodology how the structural damages can be detected from the monitoring data.

Besides, with increasing of span and slenderizing of structure, long-span bridges become more and more sensitive to wind. For a long-span bridge, with limited torsional stiffness, wind-induced forces, such as self-excited force and buffeting force, can cause destructive phenomena and need special attention. Self-excited forces causing flutter are in general dependent on the geometric profile of the bridge deck section, angle of wind attack and wind velocity expressed as reduced frequency. Meanwhile, buffeting is defined as the unsteady loading of a structure by velocity fluctuations in the oncoming flow.

Although many advances in design, construction as well as maintenance have been developed day to day, many problems of structure still remain unknown or unsolved. The ability to detect structural damages in a bridge before it endangers the structure has been of interest to engineers for many years. However, few studies were carried on to investigate how the damages of structure affect on wind-induced response of a long-span bridge.

This paper presents the analytical study on wind-induced responses of a long-span suspension bridge in case of some kinds of structural damage were occur. In this paper, a very detailed finite element model of a long-span bridge was developed and verified using field data, making this model as accurate as possible in representing the actual structural behavior. Using this finite element model, the reliability analysis of the bridge is performed considering dead and wind loads. After that, based on the realistic deteriorations, various types of structural damages of a long-span bridge are simulated to facilitate the discussion. All of the dynamic data for comparing damaged with undamaged cases were generated numerically from the finite element model. Lessons archived from this study are expected not only to maintain this bridge but also to improve our understanding of the real bridge performance as well as to provide useful feedbacks for future design.

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New high performance steels for long-span bridges

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ABSTRACT

This paper presents developing process, material properties, application and economic benefit of newly developed High-Performance Steel for Bridges (HSB) in Korea.

There were remarkable improvements of steel production which brought developments of high performance steel for bridges (HPS) in US, Japan, Europe and Korea from 1990's. Although they have different names, i.e., HPS50W/70W/100W in US, BHS500/700 in Japan, S460M/S690Q in Europe and HSB500/600/800 in Korea, they have possessed similar improved strength, weldability, higher toughness, and better weathering and fabrication characteristics.

HSB has been developed through the research of the High Performance Construction Material Research Center (HiperCONMAT) sponsored by the Korea Government. The targeting specifications of HSB were derived from the opinions of the steel users and various experts, such as designers, academia, fabricators of steel bridges, who require higher strength, higher toughness, and better weldability. HSB is produced by applying thermo-mechanical control process (TMCP) and optimizing chemical compositions resulted higher tensile strength and no reduction of yield strength for thick plate. High toughness of 47J at $-20°$ or $-40°$ reduces occurrence of brittle fracture at low temperature and improves cold-formability. In addition, the low Pcm and Ceq of HSB enhance weldability, that is, reduce cold cracking and lower preheating temperature.

HSB500 (tensile strength of 500 MPa) and HSB600 (tensile strength of 600 MPa) were successfully developed and registered in Korean Industrial Standards (KS D 3868) and added to the new Korean roadway bridge design code. Until now 46,800 tons of HSB has been used for bridge construction from January 2010.

In order to use HSB to bridge structures more effectively and widely, innovative steel bridge systems, such as hybrid girders, new steel-concrete composite girders, and a pin-connecting steel modular bridge system investigated in the HiperCONMAT, are presented. HSB800 (tensile strength of 800MPa) which are under development for long-span bridges is also introduced.

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Special Sessions

SS1: Use of health monitoring for life-cycle cost analysis & optimization

Organizers: D.M. Frangopol & A. Orcesi

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Optimization of the amount of structural health monitoring via probabilistic system assessment

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ABSTRACT

Nowadays, the structural condition of newly erected important elements of the infrastructure is monitored using complex structural health monitoring (SHM) systems. SHM systems are as well used to guarantee the safety of old structures that show significant defects. A continuous SHM leads to a large amount of data which makes the evaluation and interpretation very difficult.

One possibility to optimize the SHM process is to adapt the amount of monitoring according to the condition of the structure. A continuous monitoring procedure is only reasonable if a further increase of damage would lead to serious problems up to a structural collapse. The amount can be reduced to a periodic monitoring in case the structure shows only minor damage or still has enough reserves.

Normally, no quantitative information about the condition of the structure is directly gained from the measurements. This paper suggests a methodology that utilizes a probabilistic method to quantify the actual condition of the structure based on the time-history of collected monitoring data.

The structural system is described in a risk-orientated probabilistic model based on identified potential weak points. The limit state functions for these weak points incorporate measurable parameters, e. g. strains or deformations, which makes the model easy to update in case new data is available or damage is identified. The structural condition is quantified by applying appropriate reliability methods (First/Second Order Reliability Method, Monte Carlo Simulation) to the model of the structural system.

The time-dependent variables (mainly the monitoring data) can be accounted for by a statistical distribution that is fitted to extreme values that occur within a reference period. Hence, the time-dependent problem is traced back to time-independence and the reliability methods mentioned above are applicable. This methodology is tested for different monitoring periods to identify variations of the extreme value distributions among the monitoring datasets. The different distributions found were tested for their influence on the reliability index β when being used in the LSF.

In an exemplary application, a steel truss bridge is modeled as a serial system, so that the overall system reliability is highly influenced by the component with least reliable component, which can be identified by looking at the highest sensitivity.

A way to identify and locate potential damage is to compare the sensitivities of the components in subsequent evaluations. A strongly decreasing sensitivity of a component can be an indicator of load relocation, which in conclusion might indicate that a damage has occurred in that component or in the vicinity.

Load relocation away from damaged components can lead to a calculated increase of the system reliability. Hence, this increase can also be used as an indicator for damage assessment. The methodology introduced, and especially the calculation of the reliability index of the structural system β and the sensitivities of important parameters, can be used to adapt/optimize the SHM process in terms of monitoring periods/amount of monitoring, and to evaluate the structural condition (i.e. damage) of different components in an infrastructure network.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

The potential link between bridge management systems, structural health monitoring and bridge weigh-in-motion – progress and challenges

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ABSTRACT

Bridge management systems (BMS) have been developed in many parts of the world. The majority of systems manage inventory and inspection information and many also provide analysis capabilities to forecast deterioration, recommend preservation activities (such as maintenance, repair, and rehabilitation), functional improvements (strengthening and widening), and structure replacement. These advanced BMS offer powerful predictive and cost-benefit analysis tools that help determine the most cost-effective activities on individual bridges and over a network. With few exceptions, these BMS all rely on visual bridge inspection techniques and inspector judgment, are heavily dependent on adequate inspector training, and are performed at discrete intervals.

Structural health monitoring (SHM) is being applied on bridge structures around the world. These systems vary from large specialized monitoring systems on signature bridges, to more basic systems designed to monitor a specific problem on a particular structure, or systems having various other purposes. Advances in data acquisition, storage, remote sensing, and structural instrumentation have made it possible to conveniently and remotely monitor many things including strains and deformations; cracking occurrence and propagation in concrete as well as steel structures, accelerations \ vibrations, and of course environmental conditions such as wind speed, temperature, and humidity. In the last few years bridges are being instrumented for weigh-in-motion (BWIM) to record truck axle loads and frequencies.

Despite obvious advantages, there has been little progress in the integration of SHM and BMS. This paper explores the synergies that exist in merging the capabilities of BMS with the information made possible with SHM. Challenges are presented to researchers and industry to develop means to provide the type of condition based data that BMS need, and to develop ways to utilize existing available SHM data to improve BMS with a view to help bridge owners better manage risk on the bridge network.

This discussion paper presents a synopsis of the potentials for linking bridge management systems (BMS), bridge health monitoring systems (BHM), and bridge weigh-in-motion (BWIM) systems. Suggestions are provided for BMS, BHM and BWIM developers to enhance and develop new ideas. It is hoped that this research will further advance the concepts of the integration of these systems and provide suggestions to BMS, SHM and BWIM system developers for future development, and benefit agencies who are already implementing monitoring solutions or who are considering monitoring projects.

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Probabilistic optimal bridge monitoring planning

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ABSTRACT

Structural performance cannot be assessed and predicted accurately due to various epistemic and aleatory uncertainties. Recently, structural health monitoring (SHM) has been considered as a significant tool to reduce the uncertainties associated with structural performance assessment and prediction. Under limited financial resources, optimal monitoring planning should be considered.

In this paper, probabilistic methodologies to establish optimal monitoring planning with uniform and non-uniform monitoring time intervals are introduced. For the monitoring plan with uniform time interval, the statistics of extremes and availability theory are used. A bi-objective optimization based on minimizing the total monitoring cost and maximizing the availability of the monitoring data for performance prediction is formulated. Results of this formulation are shown in Figure 1. For the non-uniform time interval monitoring, the optimal monitoring planning is formulated as the minimization of the expected damage detection delay. The two proposed approaches can be used as effective tools for integration of SHM into cost-effective lifetime maintenance strategies of deteriorating structures.

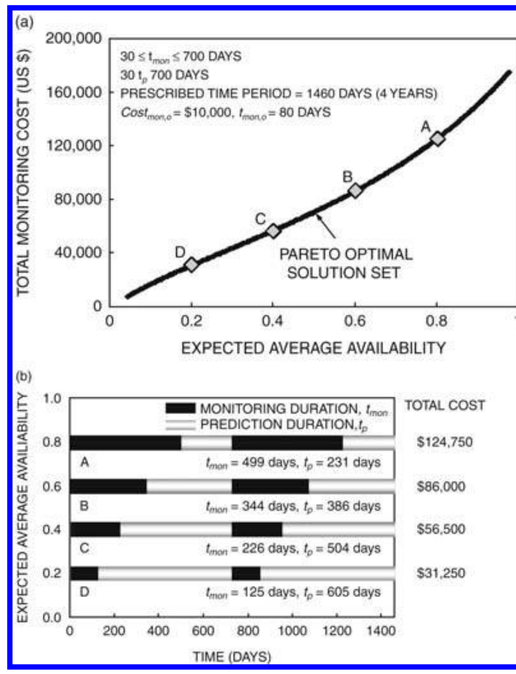

Figure 1. (a) Pareto solution set; (b) monitoring plans of solutions A, B, C, and D in (a).

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Optimization of bridge maintenance actions considering combination of sources of information: Inspections and expert judgment

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ABSTRACT

Due to the high costs associated with maintenance of existing structures, but also the significant uncertainty present in any prediction of future performance, the use of advanced probabilistic life-cycle deterioration models is fundamental in bridge maintenance and management. However, such models must be based on direct and indirect information on structural performance, including results of inspections, health monitoring, but also expert judgment and information on other similar structures. In the proposed approach, a probabilistic deterioration model, considering both condition and safety as indicators of performance, is used as a decision aid in bridge management. The data considered for defining the performance profiles over time is based on the expert opinion, resulting from observation of similar structures.

In order to reduce uncertainty and correct the existing predictions, the results of direct information, in the form of visuals inspections, are combined with initial prediction. Visual inspections are not free of uncertainty. Therefore, a probabilistic framework must be developed. The combination of these two sources of information (i.e., initial prediction and visual inspection) is performed considering Bayesian updating techniques, using Monte-Carlo simulation.

In this manner, it is possible to improve the knowledge on the performance of the structure, but also reduce the uncertainty in several parameters, including initial performance level, deterioration rate and effect of maintenance actions.

Based on this new information, it is possible, considering multi-objective optimization using genetic algorithms, to define the optimal lifetime maintenance strategy for a structure. The three objective functions considered are: life-cycle cumulative cost, condition index, and safety index. The optimization procedure will provide a set of optimal solutions, associated with different life-cycle costs, from which the decision maker can choose, considering available budget and performance alternatives.

The methodology is applied to a set of bridge components. The results show the significant improvement in prediction quality obtained by combining different sources of information, even if the quality of the obtained data is limited. Moreover, the impact of the new information on the optimal maintenance strategies is also evaluated.

Overall, preventive maintenance actions prove to be an extremely useful tool for reducing life-cycle cost, and essential maintenance actions are paramount in limiting the effects of deterioration, keeping structures fit for use.

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Structural health monitoring: from data acquisition to optimum life-cycle management under uncertainty

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ABSTRACT

The objective of the proposed framework is to provide an efficient decision making tool for optimization of maintenance strategies of bridges based on structural health monitoring (SHM) information. Recent improvements of sensor technologies have made possible the use of monitoring data in structural performance analyses. There is a huge interest in including maintenance and SHM information in life-cycle cost analysis (Okasha & Frangopol 2009, Kim & Frangopol 2010). Indeed, by providing additional structural information, parameters of the random variables associated with resistance and loading characteristics can be better quantified (Orcesi & Frangopol 2010, Orcesi et al. 2010). Therefore, best maintenance strategies can be used. By improving the knowledge on structural performance, SHM should enable to control maintenance cost and allocate financial funds without compromising structural reliability. A multi-state reliability-based analysis process is proposed by considering fatigue, serviceability and ultimate limit states. Controlled load tests and long-term monitoring information are used to update some parameters in the structural reliability analysis.

An optimization procedure that integrates SHM information and deals with various limit states is used to determine optimal maintenance strategies. The design variables are times and types of maintenance actions and the objective is to minimize both failure and owner costs such that reliability constraints are satisfied. It is shown that optimal solutions cannot be seen as a juxtaposition of optimal results for different limit states taken individually and that lifetime management has to be considered as a whole at the scale of the civil infrastructure and not for each failure mode considered individually.

The I-39 Northbound Bridge over the Wisconsin River is used as an example to show how the monitoring information (available in 2004 after the instrumentation and long-term monitoring of the bridge during four months, see Mahmoud et al. (2005)) can be integrated in a general framework for optimization of maintenance strategies.

Figure 1 presents the comparison between two Pareto optimum maintenance solutions, A and B, in terms of cumulative costs. The cumulative expected failure cost is larger for solution A. However, the cumulative expected maintenance cost is larger for solution B.

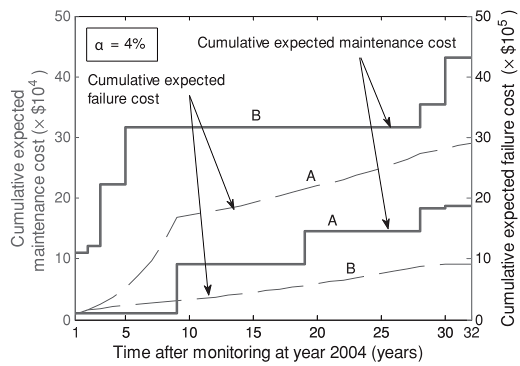

Figure 1. Comparison of cumulative expected failure cost and cumulative maintenance cost between two optimum maintenance solutions ($\alpha =$ discount rate of money).

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The role of structural identification in asset management

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ABSTRACT

A large portion of the nation's bridges has exceeded its original design life. The loads to which these bridges are subject to are often much larger than the designers anticipated. Today's typical State DOT is juggling a massive number of structures with limited personnel and inadequate budgets on a daily basis. As such, the maintenance of bridges cannot be focused on prevention. Problems are addressed as they appear based on urgency. The combination of large populations alongside budgetary and personnel constraints put considerable importance on every decision made by the bridge owner. There are tools available to help inform decisions, such as structural identification (St-Id) and life-cycle cost analysis. Individually these tools are effective, but problems still arise with application as well as candidate selection. It is therefore beneficial for a bridge owner to consider an asset management (AM) system, which can utilize both St-Id and life-cycle cost analyses to help optimize where and when funds are allocated to projects in an effort to get the most out of the both the budget and the bridge population. AM systems are entirely dependent on the quality of data which is utilized to influence decision making.

The process of St-Id as a means of mitigating uncertainty in the analysis and basic decision-making for a single structure is an established and reliable tool for an engineer or bridge owner. The authors have been utilizing St-Id over the past 20 years for structures varying from highway overpasses to signature long-span bridges. However, the application of St-Id as a tool for management of a large population of assets is a relatively new concept. This paper will address how St-Id can best be utilized within the umbrella of AM for a population of similar bridges; a common situation a State DOT may face. The foundation of using St-Id in AM is to estimate vulnerability of a population of structures in order to assess risk and facilitate the determination of appropriate candidates for further investigation from a cost-benefit perspective.

It is in obtaining quality, reliable data, that structural identification (St-Id) can play a role in an AM program. It is difficult for an engineer to associate the qualitative, visual appearance of a structure with hard numbers like capacity. Often there is too much uncertainty resulting from the complexity and redundancy typical to civil structures.

St-Id is by no means the solution to the nation's aging infrastructure problem. The in-depth St-Id process is far too expensive and time consuming to apply in a global manner to a large population of structures. It follows that full-scale St-Id is most applicable to a single or small population of assets.

The application to single assets will be presented in the context of a review of five bridges tested over the past several years. The details of these tests including modeling, experimental design, and testing are presented in various conference papers (Prader et al 2008, Weidner 2009a,b). The results of these tests, in terms of the decisions made by the bridge owner are summarized and discussed briefly herein as an example of the benefits of St-Id.

The St-Id process is viable for large populations, with adjustment to each step. This sparse, rapid version of St-Id can provide excellent performance metrics for incorporation into an AM system.

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SS2: Safety monitoring & maintenance strategy for long span bridges

Organizer: A. Chen

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Safety monitoring of the cable stayed bridge in the Commercial Harbor of Venice, Italy

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ABSTRACT

Theoretical and experimental investigation (ambient vibration tests and permanent monitoring system) on the structural behavior of a curved cable-stayed bridge is presented in the paper. The quoted bridge was erected in the Commercial Harbor of Venice-Marghera, Italy, and opened to traffic on January 2007. The bridge is characterized by six curved spans, of which the main two of 105 m and 126 m are cable-stayed, with the stays arranged on a single plane and connected to the centerline of the composite steel and concrete deck. Furthermore, the pre-stressed concrete pylon is single and inclined. As a consequence of the spatial structural arrangement, the dynamic behavior of the deck appears very complex both in bending and torsion.

The peculiar architectural layout of the bridge added significant difficulties in both accurate structural analysis and assessment of "as built" behavior. Hence ambient vibration modal testing were carried out after erection and a detailed FE model was tuned and validated on such data. The special geometric layout of the bridge and its infrastructural role provided also a strong motivation for the installation of a permanent monitoring system. The paper describes the characteristics of the monitoring system, based on the use of the SOFO™ fiber optic sensor family and conceived for both static and dynamic monitoring. A number of considerations are carried out on the capability of the quoted monitoring system to assess in due time the presence of unexpected damage on the most critical zones of the bridge, namely inclined pylon and stay-cables. Specific defectiveness index for describing the possible loss of pretension in such bridge components have been defined. Data provided from cables sensor are not considered in such analysis. This assumption derives from the fact that some sensors in the cables have proved not to work correctly due to their wrong placement during the construction phases.

Static and modal analysis were performed with a tuned FE model in order to evaluate the performance of the installed monitoring system, both from a static and

a dynamic point of view. Specific parameters able to describe the evolution of the structural response related to the presence of a certain level of damage were introduced. With reference to the dynamic analysis, the variation of natural frequency, the Modal Assurance Criterion (MAC) and the Normalized Modal differences (NMD) were considered as characterizing parameters for the structural health assessment.

Concerning the static analysis the variation of displacement levels was evaluated. Direct comparison of results emphasized that, for the considered case, static monitoring appears more sensitive to damage propagation inside pylon and cables, compared to dynamic one.

For the case presented, cable damage is not clearly detectable: for this reason, the correct and reliable instrumentation of each cable with a dedicated sensor plays a fundamental role.

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Iterative buckling analysis for steel cable-stayed bridges

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ABSTRACT

In general, cable-stayed bridges are composed of three major structural members: cables, girders and towers. The cable-stayed structural system of these bridges has shown itself to be an economical, efficient, aesthetical appealing and durable solution for the length of central span in the range of 200–800 m (Gimsing, 1983; Poldony and Scalzi, 1976). But the behavior of cable-stayed bridges is completely different with other typical highway bridges because these bridges are usually composed of three main structural members: cables, girders and towers. Cables must resist tension forces that are induced from dead loads and live loads. Girders and towers must resist bending moments as well as compression forces that occurs at the intersection between girders and towers. For this reason, buckling behavior of main members is very important in the design of cable-stayed bridges. In addition, Cable-stayed bridges exhibit nonlinear behaviors considerably due to cable sag, interaction between axial forces and moments, and large displacements (Wang et al., 2002; Shu and Wang, 2001; Xi and Kuang, 2000).

This paper illustrates a modification that eliminates the problem inherent to system buckling analysis when used to assess the stability of girder and tower members in cable-stayed bridges. The theoretical background of system buckling analysis is presented briefly, and the root of its inherent problem is clarified by describing a common cable-stayed bridge example. The axial force term in the geometric stiffness matrix is modified by adding a fictitious axial force to make the member more critical to the overall buckling of the structure. The converged effective lengths of girder and tower members are determined based on an iterative eigenvalue analysis with a continuously modified geometric stiffness matrix. The proposed iterative system buckling analysis is applied to example cable-stayed bridges that have center spans of 600 m, 900 m and 1200 m, and different girder depths. The effective lengths of individual members in these example bridges are computed using the proposed method and compared with those found using system buckling analysis. Figure 1 shows the example cable-stayed bridges for 600 m, 900 m and 1,200 m models.

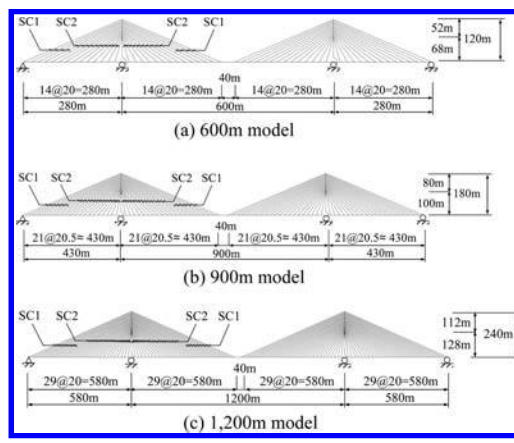

Figure 1. Example cable-stayed bridges.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Rehabilitation planning and stay cable replacement design for the Hale Boggs Bridge in Luling, Louisiana

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ABSTRACT

The Hale Boggs Bridge across the Mississippi River opened in October, 1983. It was the first cable-stayed span on the Interstate Highway system. An all weathering-steel structure, the bridge serves the public well. However, its stay cables' corrosion protection sheathing performance made the stay cable array vulnerable to both corrosion and service life depletion. Repairs were attempted during and following construction, to correct sheathing defects. The repairs performed poorly, and failed to protect the cables' main tension element.

Condition assessments and a rehabilitation study concluded that 39 out of 72 cables were in critical need of repair. Timely action was necessary. To address the damage, and to assure the structural integrity of the bridge structure, several strategies involving a range of repair and replacement options were evaluated using life cycle cost analysis. It was concluded that the strategy to replace all cables presents the best value among evaluated alternatives. The design of the complete 72-cable array replacement, completed in December 2008, is the first occasion on which this process was attempted in North America. This paper describes efforts made to improve the cables and extend the service life of this historic river crossing.

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Reliability assessment of Yonghe Bridge based on structural health monitoring

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ABSTRACT

This paper presents the reliability estimation of Yonghe Bridge based on structural health monitoring technology. The data collected by structural health monitoring system can be used to update the assumptions or probability models of random load effects, which would give potential for accurate reliability estimation. The reliability analysis is based on the estimated distribution for dead, live, wind and temperature load effects. For the components with FBG strain sensors, the dead, live and unit temperature load effects can be determined by the strain measurements. For components without FBG strain sensors, the dead and unit temperature load and wind load effects of the bridge can be evaluated by the finite element model, updated and calibrated by monitoring data. By applying measured truck loads and axle spacing data from weight in motion (WIM) system to the calibrated finite element model, the live load effects of components without FBG sensors can be generated. The stochastic process of Live Load effects can be described approximately by a Filtered Poisson Process and the extreme value distribution of live load effects can be calculated by Filtered Poisson Process theory. By combination temperature and wind pressure distribution in the present design code and unit temperature and wind load effects, the extreme value distribution of temperature and wind load effects can be achieved. Then first order reliability method (FORM) is employed to estimate the reliability index of main components of the bridge (i.e. stiffening girder). Furthermore, practical damage situations are considered to evaluate the influence of certain elements' failure on the reliability of undamaged components.

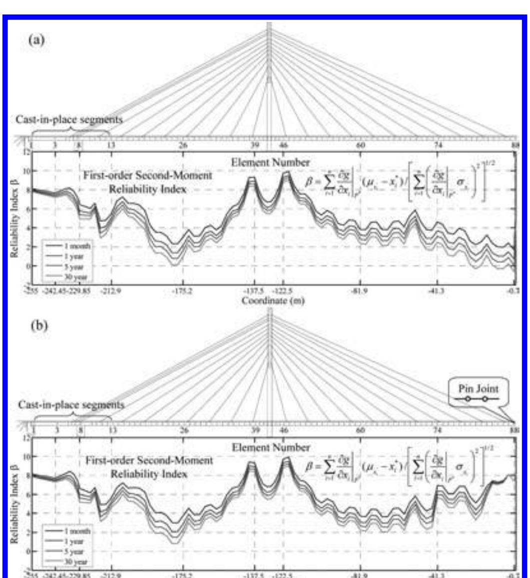

Figure 1. PDF comparison between measured and simulated vehicle load moment at closure segment.

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Risk management of newly-built bridge vibration effects on surrounding buildings

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ABSTRACT

A Concomitancy with economy development requirement and urban road traffic perfection, bridge construction demand faces a new upsurge in China. Space and distance between bridge structure and surrounding buildings are becoming smaller and shorter. Newly-built bridge, the real traffic situation of which is unknown before its service, will probably have bad vibration effects on surrounding buildings' safety. Evaluating reasonably newly-built bridge's vibration effect, which changes with vehicle flow, is a problem for existing surrounding building and is also an important measurement for risk management and reducing overall risk losses. The similar or the same bridge type for similar for vehicle load is relatively easily found while the similar or the same surrounding buildings are difficult to determinate in China. The similar bridge's dynamic effect analysis and quantitative calculation are adopted in this paper. Based on the reality research topic, this paper will present the solution to evaluate vibration effect.

Figure 1. Surrounding buildings 3D location of real research topic in Wenzhou.

Figure 2. Similar bridge test point selected in Shanghai.

Cast-in-place driven pile vibrations on construction, vehicle-bridge coupling vibration on operation, are considered as the most important dynamic influencing factors. Driven pile is treated as point vibration source moving down process. Hammer vibration energy propagates from head to all directions. Combining bridge location geological data with basic construction technology of other similar bridge, a 4ton diesel hammer with 2 m drop height is used to analyze driven pile dynamic effect.

Accordingly, different risk management and suggestions are put forward; a risk management handbook is formed to serve the bridge in the real topic successfully.

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Bridge safety assessment based on field test data with SORM method

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ABSTRACT

If the structural reliability index is close to the threshold value, the different algorithms may lead to different assessment conclusions due to the diversity of calculation results. By probabilistic methods ,the existing structures safety assessment not only need to consider the actual resistance, load effects and other field information, a reasonable choice of probabilistic algorithms is also necessary.

The paper reviews the FORM and SORM algorithm, their advantages, disadvantages and the corresponding origins are pointed out for the limit state equation used in bridges assessment.

According to the state inspection information and weighing vehicle data in motion, a large-span prestressed continuous beam bridge is assessed using the probabilistic method. The design model is updated by the different kind of field information respectively. FORM and SORM are adopted to calculate these models. The reasons for the differences between two methods and the variation law are studied.

The safety assessment of the example bridge shows that:

(1) The actual state of the existing bridge usually deviates from the design; state inspection can effectively identify the true state to establish an actual evaluation model;
(2) With the increase of vehicles, the code parameters, based on the findings of 90s of the last century, underestimate the vehicle load effect gradually. It is essential to establish the probability distribution and statistical parameters in accordance with the current traffic, and forecasting methods to consider upgrading vehicle class;
(3) Direct reliability methods have the higher suitability for the important long-span bridge, whose live loading characteristics may differ markedly from the descriptions contained in the code.

The comparison of Algorithms shows that:

(1) During the process of the existing bridges assessment, if the reliability index is close to the target value, the more accurate calculation method SORM needs to adopt, and evaluation index is essential to change from the reliability index to the failure probability. It is particularly suitable for the mid-span cross-section subject to the greatest positive moments.
(2) The difference of failure probability is almost depended on the negative normalized gradient component with respect to live load effect at the design point in the standard normal space. If the normalized gradient component is higher than 0.2, the difference of failure probability exceeds 5%. In this case, a more accurate algorithm such as SORM is suggested to calculate evaluation index.

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SS3: Bridge condition assessment

Organizers: A. Miyamoto & F. Tondolo

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Use of vibration parameters for evaluating structural degradation in R.C. elements and structures

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ABSTRACT

Degradation due to corrosion of reinforcing bars is one of the principal deterioration phenomena affecting reinforced concrete structures (Bertolini et al. 2004). It is widespread all over the world; every year a great amount of funds is assigned to maintenance and repairing works for deteriorated structure and infrastructure. Therefore the analysis of structural behavior of corroded structures, such as bridges, is becoming an urgent need for both risk evaluation and priority assignment of intervention. Corrosion of steel in concrete deeply modifies the structural behavior of reinforced concrete structures (Tondolo 2006). In fact, the oxide formation on the surface of the rebar in contact with concrete generates a significant volumetric expansion of this new layer of granular material. The radial pressure determines tensile stress orthogonal to the rebar axis and generates radial cracks on the surrounding concrete. Due to corrosion, a reduced performance of concrete elements is observed especially in service conditions. Bond mechanism is activated by cracking of concrete; its reduction generally is followed by an increase of crack opening. As serviceability and durability of concrete structures can be seriously affected by the aforementioned effects (Bhargavaa et al. 2006), control and monitoring of corrosion assume a significant importance in safety reassessment, as well as in the evaluation of the overall economic utility of an intervention (Ceravolo et al. 2009). Changes in the structure's dynamic properties may often provide quantitative evidence that damaging phenomena are underway.

This paper presents the results of laboratory tests conducted on concrete ties subjected to static load and simultaneous accelerated corrosion. The environmental attack due to corrosion of the rebar is simulated in order to evaluate the structural condition of simple concrete elements. The loading condition is derived from a bridge element under service load. The final aim is to obtain experimental information that is useful for more complex structures. Degradation due to corrosion is obtained through an electrochemical process (Giordano et al. 2009), which simulates the damage caused by about 27 years of exposure to environmental conditions XC4 (class for carbonation induced corrosion) with 25 days of test, during the propagation period. The simultaneous effect of load action and corrosion attack is established in order to take into account the interaction between stresses generated by tension load and the degradation mechanism. Crack opening due to both load ands corrosion are monitored during the tests. Furthermore, electrical strain gauges, permanently positioned on the external surface of the structural element, make it possible to measure ambient vibrations of the specimen. Deformation signals obtained from the vibration setup are then analyzed in order to extract modal quantities, including natural frequencies and modal damping, whose evolution in time is associated to structural damage, such as fatigue crack growth and corrosion.

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Structural condition assessment of long-span suspension bridges using long-term monitoring data

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ABSTRACT

This paper focuses on developing an on-line structural condition assessment technique using the long-term monitoring data measured by structural health monitoring system. The seasonal correlation analyses of frequency-temperature and beam-end displacement-temperature for Runyang Suspension Bridge are firstly performed, respectively. Then the statistical technique using a 6-order polynomial is further applied to formulate the correlations of frequency-temperature and displacement-temperature, on the basis of which the abnormal changes of measured frequencies and displacements are detected using the mean value control chart. The analysis results reveal that:

(1) The daily averaged values have better capacity to describe the correlation of frequency-temperature and displacement-temperature than the short-time values by eliminating the random variations due to ambient excitations. The minimum variance, maximum variance and mean variance of daily averaged frequencies are 0.649%, 2.186% and 1.403%, respectively. And the seasonal changes of the northern and southern displacements are 37.4 cm and 38.8 cm, respectively.
(2) A simple 6-order polynomial regression model will be developed to be a quantitative description of the seasonal correlations in the present study. The reproduction and prediction values of modal frequency and displacement favorably agree with the measured values, which indicate the satisfactory reproduction and prediction capability of the regression model. So the temperature-caused variability of the modal frequency and displacement can be effectively quantified.
(3) The mean value control chart is further employed to monitor the time series of condition index e for real-time condition assessment. The proposed method can effectively detect the damage-induced 0.2% variances of the modal frequencies and the damage-induced 2.0% variances of the displacements of the Suspension Bridge
(4) The results show that the proposed method can effectively eliminate temperature complications from frequency and displacement time series and exhibit good capability for detecting the minor abnormal changes of measured modal frequencies and displacement, which is suitable for online health monitoring for long-span suspension bridges.

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Time-dependent reliability of PSC bridge box-girders exposed to aggressive environments

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ABSTRACT

Bridge structures undergo time-varying changes in response when exposed to aggressive environments. While much has been done on reliability of time-dependent corrosion, little is known to the influence of concrete creep and shrinkage that can undermine the reliability of bridge box-girders. In this study, an MC90-based creep, and shrinkage model is depicted by using the finite element (FE) codes in conjunction with time-dependent corrosion. More specifically, time-dependent assessment is made for a composite pre-stressed concrete (PSC) box-girder bridge under aggressive chloride attack. This assessment is realized via an advanced stochastic finite element method (SFEM). The thin-walled structure is described by the composite degenerated shell element, see Fig.1, in which the embedded reinforcement grids are used to simulate the distributed reinforcements, and an automatic generation scheme is adopted to facilitate the finite element modeling of the curved prestressing tendons. The post-cracking behavior of concrete is depicted by the smeared cracking model with the Hordijk's tension softening relation. To improve the efficiency of reliability analyses, an approximate Importance Sampling arithmetic is proposed, in which a number of Monte Carlo simulations are used to get the proximate design points for the Importance Sampling; thereafter, the Importance Sampling is conducted with its center at the selected design points.

Reliability analyses show that under the combined action of creep, shrinkage and corrosion, reliability indices of the PSC box-girder undergo a three-phase reduction, and fall below the target reliability level earlier than the expected service life. In phase I (about 10 to 12 years after loading), a decrease in the reliability of tendon yielding is observed, see Fig.2 (a). Meanwhile, a faster increase in deflection occurs due to creep and shrinkage, which results in a significant decrease in the reliability index, as shown in Fig.2 (b). In phase II, the evolution of concrete creep and shrinkage becomes slower, and a mild decrease in the reliability index is observed. A significant decrease is found again in phase III (about 50 to 60 years after loading), which is mainly controlled by corrosion.

The proposed methodologies in this paper can be used for a better understanding of the deterioration process of PSC bridges, and development of optimal reliability-based maintenance strategies.

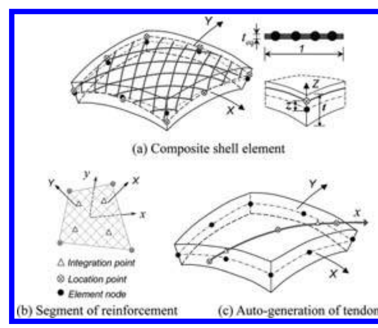

Figure 1. Composite degenerated shell element.

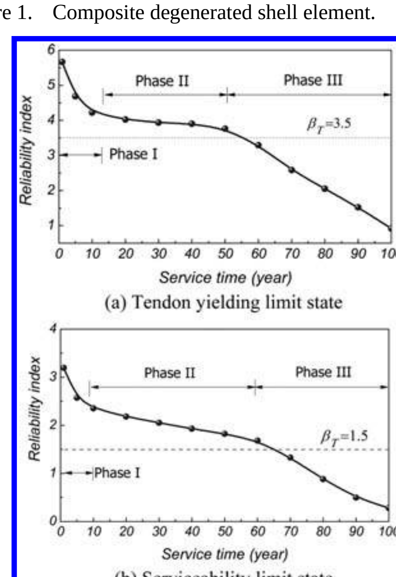

Figure 2. Reliability index profile.

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Condition improvement of deteriorating bridges by using high performance ceramic materials

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ABSTRACT

As time has elapsed since their completion, aging is advancing in the structures of the Hanshin Expressway. Damages to structural parts during this period of time, increased due to heavy traffic loads. In particular, fatigue damages and corrosion in steel structures, as well as damages in the pavements are problems that have to be urgently solved. Due to the public nature of roads, assurance of the road safety is required from the road manager and rational methods of management, as well as maintenance are required.

As repair sites are located in severe environmental conditions (water, rust, salt), new methods and materials that can be applied under these conditions are necessary. To cope with such severe environmental conditions, a new ceramic material was used for this research as a new repairing and reinforcement material.

The following characteristics are generally attributed to the new ceramic material:

(1) rapid hardening
(2) high strength
(3) no hardening shrinkage
(4) neutral
(5) excellent covalent bond
(6) accuracy
(7) high fire and heat resistance
(8) high thermal insulation
(9) chemical resistance
(10) workability under low temperature, including below zero
(11) harmlessness
(12) environment friendly and
(13) abrasion resistance.

Among these characteristics, excellent bonding adherence, high strength and early high strength, accuracy are properties that makes the ceramic material suitable as repair and reinforcement material.

The present paper reports the results of a series of experiments performed to verify its applicability to real world conditions in the field. The experiments consisted of tests on the material properties, tests on its ability as composite structures with steel (composite or hybrid structures) and tests on its workability in site when applied to structures (steel structures).

The experimental results, proved that ceramic mortar can be applied as reinforcement materials for mastic asphalt and undrained pavement, without problem from the adherence strength point of view.

Based on these results, actual application in the field, such as the ones introduced below, are being planned to reinforce structures of expressway viaducts.

(1) Due to its high bonding and rapid hardening properties, application to reinforcement of asphalt pothole.
(2) Due to its high accuracy and anti-corrosion properties, application to corrosion reinforcement of steel structures.
(3) Due to its high strength, application to reinforcement of orthotropic plates, by making composite or hybrid structures with steel members.

However, in applying ceramic materials to actual structures, the following topics have to be considered.

(1) Verify corrosion resistance and weathering resistance and confirm durability.
(2) Verify behavior under repetitive load and investigate fatigue problems.
(3) Investigate a method to reduce the hardening thermal effects on the adherence strength in steel structure.
(4) Consider compatibility with construction quality control by increasing the workable time during hardening
(5) Establish standard construction conditions and quality stability
(6) Investigate the effects of introduction of large scale equipment in its construction.

These topics will be eventually cleared, step-by-step, in future studies.

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Rehabilitation of Haynes Avenue Bridge

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ABSTRACT

Haynes Avenue Bridge in Newark, New Jersey, USA was built in 1931 and has a length of 1164 feet and a two-lane roadway width of 42.8 feet. It is a 9 span concrete encased steel through-girder and floorbeam, and 4 span reinforced concrete deck girder with floorbeam structure. The bridge crosses over the busy Amtrak and NJ Transit Electrified Northeast Corridor Lines, Conrail Rail Lines, Port Authority's Monorail Lines, and a Service Road.

The condition of the bridge deck was considered to be poor and the superstructure fair. In August 2006, the NJDOT elected to rehabilitate the bridge without replacing the deck. The rehabilitation scheme includes the removal of loose concrete from the concrete encased steel girders, repair of spalls and cracks on the deck, parapets, piers, abutments and stairs, repair of steel girders, replacement of the existing rocker bearings with seismic isolation bearings, replacement of deck joints and down spouts, repair and painting of bridge railing, and the removal of debris and vegetation from the bridge.

A seismic retrofit analysis was carried out and it was found that the existing rocker bearings and anchor bolts were inadequate for a seismic event. Therefore, the existing rocker bearings were replaced with seismic isolation bearings. The isolation bearings consist of steel reinforced elastomeric layers with a lead core. They were specially designed to reduce the seismic displacements.

The repair work over the Northeast Corridor had to be done within a four hour window, between 12 AM and 5AM, which included de-energizing and energizing tracks. Shielding was provided when full depth deck repairs were carried out. For the removal of concrete encasement or the injection of cracks, the tracks, ties, ballast, and catenary wires were protected from falling debris. A protective shield was prepared over the Port Authority's Monorail Lines.

The existing bridge deck was overlaid with asphalt and a Ground Penetration Radar (GPR) system was used to measure the depth of the overlay. GPR was also used to determine the deterioration of the deck. The existing asphalt layer was removed to carry out the deck repairs and was relayed with an asphalt layer. The GPR results were compared with results from other conventional techniques such as the chain drag test and the actual deterioration as found once the entire asphalt layer was removed.

The paper discusses how the rehabilitation of this bridge over one of the busiest transportation corridors in the country was done and how GPR was used very effectively in the deck rehabilitation work.

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Influence of corrosion on prestressed concrete beams: An experimental survey

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ABSTRACT

Steel corrosion is one of the most common causes of deterioration of reinforced concrete and prestressed structures. The phenomenon is particularly felt in the bridge structures, mainly if exposed to aggressive environments and subjected to thaw salts. Unlike the normal reinforced concrete, where the corrosion damage is highlighted by the formation of cracks in the concrete, and by rust spots, the failure for stress-corrosion occurs suddenly, as the formation of cracks in the metal does not require an intensive corrosive attack, causing a strong material loss.

The study of these aspects and the evaluation of the influence of the corrosion on the constitutive behaviour of tendons and the consequent structural implications are studied by means an experimental campaign carried out at the Laboratory of the University of Rome "Tor Vergata". Three series composed by three rectangular prestressed beams each (Fig. 1) have been subjected to four point bending tests. Besides the reference sound beams, the tendons of the elements have been artificially corroded, in order to obtain three different corrosion levels, equal to about 7%, 15% and 20% mass loss (Tab. 1). The obtained results (Fig. 2) highlight the influence of the corrosion on the global strength of the element, in terms of ultimate force reduction, and, mainly, on the variation of the failure mode of the corroded beam. Contrarily to the sound beams, whose collapse is governed by the concrete crisis, the failure of elements corroded with mild and high corrosion levels (14%–20%) is due to the strand local rupture; the beam subjected to low corrosion level shows a sharp ductility reduction and collapses abruptly for the contemporaneous crisis of concrete and strand.

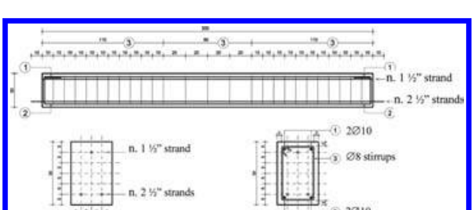

Figure 1. Beam geometry.

Table 1. Test program and corrosion level.

Series	Beam	Corrosion [%]	fc [MPa]
1	7	0	34.0
1	8	20	34.0
1	9	20	34.0
2	1	20	41.5
2	2	0	41.5
2	3	14	41.5
3	4	0	47.4
3	5	20	47.4
3	6	7	47.4

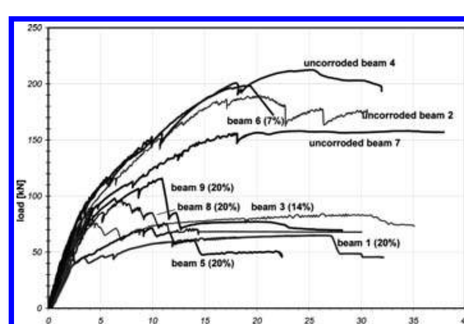

Figure 2. All series, load – displacement curves at midspan.

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Development of a compound inspection method to detect fatigue damages on orthotropic steel deck

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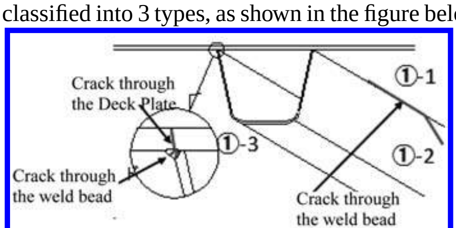
Steel road bridges having orthotropic decks, due to its lightness and short construction time, have been widely applied in the coastal area of urban highways. In recent years, with the increase in the traffic amount and the constant occurrence of overloaded vehicles, fatigue damages on orthotropic decks have been reported.

Fatigue damages on orthotropic decks can be classified into 4 groups.

① Cracks occurring in along the weld between the deck and U-shaped rib
② Cracks occurring in the weld between the U-shaped rib and transverse beams.
③ Cracks occurring in the butt weld between U-shaped rib and transverse beams
④ Cracks occurring in the weld between the deck plate and the vertical stiffeners

In particular, type① cracks originated in the root of the weld between the U-shaped rib and the deck can be classified into 3 types, as shown in the figure below.



Among crack types ① to ④, cracks of type ①-3 develop from the weld root in the direction of the deck upper surface. These cracks not only cannot be visually detected from the lower surface of the deck, but also, cannot be found by visual inspection from the upper surface of the deck without the removal the pavement. Thus, ultrasound inspection carried out from the bottom surface of the deck plate was applied. However, ultrasound inspections have the following disadvantages.

- Because they require contact with the steel orthotropic deck, proper equipment and scaffolding works are necessary.
- In high viaducts or water crossing, they require large-scale overall scaffolding works.
- Because of the above mentioned requirements, the efficiency or the whole inspection is not good.
- Comparing the inspection costs, the costs of the facilities are high, having budget restraints.
- Although the cycle of the inspections are not yet defined, periodic inspections are necessary.

Thus, the authors have developed a rational and economic method, which combined ①infrared inspection, ②eddy-current inspection and ③phased array ultrasonic inspection.

Through the study, it was found that a rational and economic inspection can be achieved when the 3 inspection methods are applied orderly by starting from a general and wide-range method, and deepening and narrowing to a more directed method, that is,

① Infrared inspection through thermal images obtained by an infrared camera mounted on a vehicle moving in high speed. (Screening)
② A simplified method of evaluation applying eddy current inspection to the points that were selected during screening.
③ Detailed investigation through phased array at points in which cracks perforating the deck were found by the simplified evaluation method.

In case shortening of inspection intervals becomes necessary, the present compound method, that made screening possible, can be applied without great increase in the inspection costs.

In order to improve the detection limits and precision of the present method, the authors shall continue to collect inspection data for further studies.

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Application of Bayesian Network for concrete bridge deck condition rating

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ABSTRACT

Bridges as one of the important infrastructures are suffering from many defects during their service lives.

Condition rating (a judgment of a bridge component condition in comparison to its original as-built condition) and damage detection are crucial methods and tools to conduct efficient maintenance and management of bridge structures. Condition rating index is a sign of concrete state of the material or bridge element at a given time.

Although the problems related to define and assign precise condition rating to bridges are almost obvious and based on the visual inspection findings, visual inspection is still the main action used to show bridge conditions. As a logical result of importance and practicality of visual inspections some refinements for condition rating methods are required.

Inspection results show significant variability. This variability has direct influence on the assigned condition rating. Condition rating results analysis and data from some investigations revealed that the condition ratings are normally distributed in many visual inspection reports. Since the bridge inspection results and condition ratings show normal statistical distribution probability theory can be considered for prediction of condition rating of concrete decks.

Bayesian Networks provide a method to represent relationships between propositions or variables, even if the relationships involve uncertainty, unpredictability or imprecision. They may be learned from data, created by an expert, or developed by a combination of the two. They capture knowledge in a modular form that can be transported from one situation to another. It is a form people can understand, and which allows a clear visualization of the relationships involved. Therefore Bayesian Networks can be used in modeling uncertainty. The central feature of the Bayesian Network approach is that given a scenario, a Bayesian Network depicts graphically the cause and effect relationship between various elements of the scenario.

In this paper the main goal is to propose a Bayesian Network which represents links of some cause and effect relationships between condition rating and symptoms or results found during concrete bridge deck inspection. Therefore it will be also possible to predict the most probable condition rating of the deck in future.

It is believed that the obtained network is tolerable to noisy input data because the probability transition of one state to another in output is smooth.

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Vulnerability analysis of a reinforced concrete structure by visual inspection

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ABSTRACT

The article outlines a new, reliable and robust approach for estimating the vulnerability of reinforced concrete sructures, developing a survey methodology based only on visual inspection.

The results of visual inspection are subjective assessments of the structural safety yet easy to obtain employing unskilled operators. Aim of our work is to obtain an objective assessment of the safety level by manipulating the linguistic judgments using fuzzy logic (Zadeh 1965), chosen for its ability to quantitatively treat the uncertainty of reality and inaccuracy of natural language. The goal of our method is to manipulate through fuzzy logic the subjective linguistic judgments, expressed by an unskilled inspecting staff, on the visual signs of degradation, in order to assess the current safety level of the construction. During visual inspection, assessment cards will be filled in by the inspecting staff, who will also be making photos and videos of the construction.

In the card for each structural element we have three columns. In the first column for each structural element are shown the degradation expression that are more relevant to the structural element itself. There can be one or more expressions, depending on the element under consideration. In the second column the inspection staff will subjectively chose the assessments of gravity attributed to the different types of degradation. The third column shows the weights that each judgment will have on the current safety assessments. To each linguistic variable, and both for the judgments G_{ij} and weights W_{ijk}, we associate a membership function $\mu(x)$ whose domain is [0, 1]. The triangular membership functions are shown in Fig. 1.

It is important to stress out that the card is constructed and the relations between judgment and weight on the safety previously stated according to the expertise of a technically skilled team, while the second column is filled in during visual inspection by unskilled staff. The reliability of the method is shown by analyzing an existing reinforced concrete structure, a bridge 3 km long, located in an aggressive seawater environment. The results are within the values of failure probability that we expected (Zhao & Chen 2001). Moreover the robustness of the method is proved when two different operators give slightly different linguistic judgments and derives directly from the ability of fuzzy logic to deal with uncertainty.

Table 1. Degradation assessment card.

Type of degradation Ei	Linguistic judgment Gij		Safety goal Wijk	
E_1 Shear stress	Small	G_{11}	Medium	W_{114}
	Medium	G_{12}	Large	W_{126}
	Large	G_{13}	Very Large	W_{137}
E_2 Longit. stress	Small	G_{21}	Slightly Small	W_{213}
	Medium	G_{22}	Medium	W_{224}
	Large	G_{23}	Large	W_{236}
E_3 Gravel nests	Small	G_{31}	Very Small	W_{311}
	Medium	G_{32}	Small	W_{322}
	Large	G_{33}	Medium	W_{334}

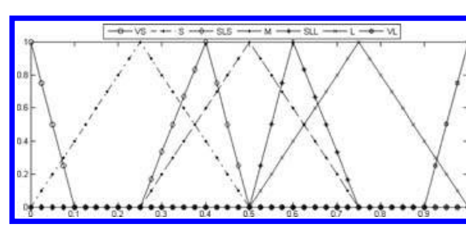

Figure 1. Weight W_{ijk} membership functions.

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Remaining lifetime and resistance of corrosion attacked bridge deck

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ABSTRACT

Bridges are the most significant components of the transport infrastructure from the viewpoint of safety, durability and serviceability and also from economic point of view. Therefore, it is necessary to paid enhanced attention to their design and exploitation to fulfil required function during the bridge working life. Present standard methods of the reliability theory are based on verification of the structural reliability from the viewpoint of limit states caused by the permanent, variable and accidental actions. Nevertheless the actual member resistance can be changed by various factors as for example degradation of materials due to aggressive environment, which are taking into account indirectly or neither in bridge member design. The investigation of actual reliability and determining actual remaining lifetime of structure are the important characteristics for evaluation of existing bridge structures.

The purpose of this paper is to analyze uncertainties of orthotropic steel plate deck in order to evaluate its random variable resistance using the finite element simulations and to get the time variant stress-state influenced by corrosion effects. The effort was focused on the determination of stress state of compression orthotropic decks creating the upper flanges of bridges with ballast beds. It is consisting of thin-walled sheets stiffened by the system of longitudinal and transversal stiffeners. All of the above mentioned factors are stochastic in their nature, so the probabilistic approach to the structural reliability assessment is necessary to use. There was developed the probabilistic model in ANSYS software environment using software support module PDS. Random variables were described by means of parameters of probability density distribution according to data determined experimentally. The model also comprised the geometrical and material nonlinearities of the structure. Nevertheless the actual member resistance can be changed by various factors as for example degradation of materials due to aggressive environment. The time variant loss of member resistance was computed based on some corrosion models.

ACKNOWLEDGEMENT

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The condition assessment and strengthening for the deteriorating prestressed concrete trussed combination arch bridge

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ABSTRACT

The first prestressed concrete trussed combination arch bridge was constructed in 1981 in Guizhou Province, China. Then the Jiangjiehe Bridge was constructed in cantilever assembling by derrick mast with main span of 330 meters in 1995. This arch bridge is combined through the trussed arch bridge and the T-shaped truss rigid frame bridge with the advantage of convenient construction, low cost and short duration. But now half of them are in great damage and disease after over 20 years' service.

In this paper, the condition assessment was performed on the deteriorating trussed combination arch bridges. Their main diseases include down flexure of the solid-web girder at midspan, Part prestress lost, concrete cracking in most truss joints between arch ribs and precast plates of lower and upper chords (Fig. 1), and part of precast plates fracture. Most of them are assessed at grade B, C or D for the technical condition. The static and dynamic loading tests show that the carrying-capacity greatly decreases with obvious impact effect under living loads. The low constructing quality and the design defect endanger them. They need to be strengthened in structures or degraded in load-carrying capacity.

Then some retrofitting and strengthening methods were proposed. The cracks need to be repaired to enhance the material durability. The joints need to be strengthened with bonded steel plates or CFRP sheets to enhance the structural integrity. The arch rings should be strengthened with cross-section enlargement to enhance the structural stiffness. And the CFRP sheets should be bonded to the bottom of the solid-web girder and the external prestressed cables should be fixed at the midspan to arrest the down flexure and increase the load-carrying capacity.

At last, some design and construction improvements were discussed to develop the prestressed concrete trussed combination arch bridge. this type bridge is worth developing with the latest design method. Most the existing trussed combination arch bridges can be kept in service after retrofitting and strengthening.

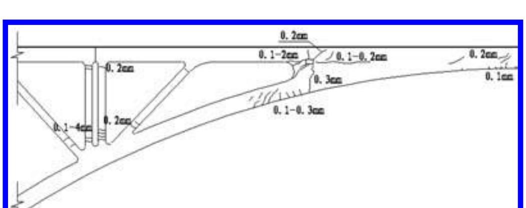

Figure 1. The truss joints crack greatly between precast upper and lower chords, web members and vertical members.

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Development of a bridge condition assessment system by using city bus

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ABSTRACT

Because many infrastructures such as bridges will entrance their last stage of designed durability in Japan, it becomes much more important to judge whether replace them or use them continuously over their designed durability while doing maintenance. An old bridge should be inspected much exactly by expert engineers rather than at its initial stage of lifetime. Because, a last stage bridge be exposed danger that the bridge will be damaged severe by usual action. Now, the structural health monitoring (SHM) gives us useful inspection data. And SHM have been researched all over the world. Whether will be judged inspection by expert engineers or SHM, a cost problem must be solved. We have been researching the reasonable method of a bridge inspection. In this paper, the concept of a bridge condition assessment system using city bus on a regular route.

In inspection by expert engineers or SHM, the cost problems for bridge maintenance should be solved first. The above results of experiment and sensitivity analysis show possibility of using a city bus acceleration monitoring for reasonable assessment of the bridge condition. Therefore, it can be considered to apply a newly monitoring system by using a city bus transportation system. A creative idea concerning using city bus transportation system to monitor bridges at a certain local area is proposed here. This system gives following merits. At first, only installing one acceleration sensor on a bus can monitor major bridges at a certain local area, so that the monitoring initial cost problem was reduced. Next, problem such as power supply to sensors will also be solved by using the bus battery. Another monitoring running cost problem is sensor maintenance, especially for necessary change of sensors sometimes, which could be solved by doing maintenance at bus station. Therefore it is no necessary to go to a lot of places to install sensors on bridges and to maintenance or change sensors there if using a bridge monitoring system with a bus. These merits solve almost all cost problems for sustainable bridge maintenance.

Main conclusions about foundational investigation for reasonable method of detecting sever damaged bridge maintenance can be drawn as follows.

(1) City bus rear axis acceleration response has liner relation with bridge acceleration response.
(2) It is possible to use a bus acceleration monitoring system for assessment of the bridge condition.
(3) Sensitivity analysis results by applied "Substructure Method" were shown that city bus acceleration is useful for evaluation of the bridge condition.
(4) A foundational investigation for reasonable methods of detecting bridge with severe damage by bridge maintenance is carried out.

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SS4: Bridges for high speed railways

Organizer: R. Calcada

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Resonant effects on a bowstring railway bridge with orthotropic deck

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ABSTRACT

The highly conditioned layout of high-speed railway tracks has led to an increased number of bridges and viaducts that use lower deck structures, due to minimum clearance limits. This is the case of bowstring bridges, where the deck, hanged by two lateral arches, often consists of an orthotropic grid of cross girders and longitudinal stiffeners joined together by a top plate or slab.

The orthotropic decks are particularly subjected to dynamic amplifications, due to their low mass, small span of the cross girders and also due to the direct charge by the train axles (Carnerero Ruiz, 2007). In spite of that, standards do not yet cover these specific problems. Until now, most of the studies on the dynamic behavior of high-speed railway bridges have focused on the analysis of the main structural elements, and these studies lead to the most recent standards (EN 1991, 2003 and EN 1990, 2005).

In this paper, the dynamic behavior of a bowstring bridge with an orthotropic deck, located in the Brussels-Köln high-speed railway line, the Prester Bridge, is analyzed.

Three-dimensional numerical models of the bridge were developed in order to evaluate the resonant effects due to global and local modes of vibration of the deck when crossed by the Thalys high-speed train.

The influence, on the vertical accelerations of the deck, of different parameters such as the train speed, the track modeling, the number and types of modes of vibration considered and the adopted modal damping ratios were studied and the obtained results were compared to the standard limitations to this parameter.

The performed analysis enabled to draw the following main conclusions:

1) For bridges where local modes play an important role on their dynamic behavior, the consideration of higher modal damping ratios for local modes, besides being a more reasonable assumption of the real behavior of those bridges, can significantly reduce the estimated values of maximum accelerations;
2) Once again, where local modes play an important role on dynamic behavior, the explicit modeling of the track leads to a better distribution of the loads reducing the dynamic response, especially at the each end of the structure;
3) The consideration of standard damping values for all modes, and the non-explicit modeling of the track, would lead to excessively conservative results for vertical acceleration.

Figure 1. Prester Bridge.

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Long-term monitoring of Sesia high speed railway bridge

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ABSTRACT

Structural monitoring has been widely adopted in the last years on small, medium and long-span railway bridges for the real-time assessment of structural reliability and safety, the control of serviceability conditions, the determination of actual load/stress spectra on main structural elements due to traffic loads. This structural health control approach allows to issue early warnings on potentially catastrophic structural damages, and also to plan in advantage maintenance and repair operations with consequently economic benefits (Ko, 2005).

Among many different techniques, the vibration measurement approach is one of the most widely adopted method for the dynamic evaluation of civil structures. This method is based on the inverse evaluation of structure dynamical modal properties (i.e. frequencies, shapes and damping ratios), generally by the elaboration of structural acceleration data. (Fujino, 2002). Modal parameters can then be adopted as reference for a monitoring system, as structural damages or deteriorations significantly change their values. It is also possible to develop numerical procedures for the real-time identification of damage entity and position along the structure (Fryba, 2001). Monitored modal parameters can be adopted in the updating of FE models, obtaining reliable real-time evaluation of forces/stresses in structural elements and, consequently, of global/local safety of the bridge. Strain measurements have been also widely adopted in the monitoring of civil structures for the real-time control of strain/stress load spectra in structural elements, in order to evaluate resistance safety and fatigue life behavior of local details (Li, 2002). Strain sensors embedded into structural concrete structural elements were adopted to evaluate directly loads, frequencies and velocity of vehicles on railway bridges (Karoumi, 2005).

In the last few years, the construction of new high speed (HS) railway lines across Europe was accomplished. The high value of design speed and the strict parameters for vertical and transverse acceleration imposed by tracks to vehicles required the realization of lines characterized by lower values of vertical gradient and curvature radius than classical railway lines. Many bridge and viaduct solutions were realized in these new lines by the adoption of classical concrete solutions together with new steel-concrete typologies (Chellini, 2009). Maintenance and serviceability of bridges and viaducts constitute a crucial aspect in the management of high speed railway lines, due to the economic losses connected with unplanned interruption of traffic. Monitoring systems were installed on most bridges and viaducts, to control continuously the structural health conditions and the serviceability performance of structures.

In this paper, short-term dynamic analysis results and long-term monitoring system preliminary data obtained for a new steel-concrete double-box composite high speed railway bridge are illustrated. Preliminary data obtained from the long-term monitoring system have been processed in order to evaluate the ability of the system in the estimation of the modal dynamic parameters and actual railway traffic spectra on the bridge.

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Fatigue assessment of Sesia high speed railway viaduct

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ABSTRACT

Fatigue and fracture are very important failure modes for steel structures, causing about 80–90% of total damage. In particular railway bridges, enduring million of stress cycles during their life, are expected to be highly vulnerable to fatigue phenomena. The fatigue assessment of new and existing railway bridges is one of the main issues in current engineering practice because of many concurrent events, as the rapid development of the European railway networks (European Commission, Energy and Transport DG, 2005) the increase of passenger and freight traffic, the introduction of new materials, the adoption of new steel and steel-concrete composite solutions and local details.

Present management approach of the European Railway Network expects the possible presence on the same line of many train families, with several layouts of weight and axel distributions, introducing new uncertainties about the most convenient traffic load spectra for fatigue assessments (Goicolea et al, 2004). These new operating conditions require the evaluation of adequacy and effectiveness of fatigue design rules actually provided by national and European codes, as these recommendations result from previous studies on old railway lines and could lead to an underestimation of actual railway traffic effects.

The general increase of operating speeds exposes bridge superstructures to resonance phenomena, especially when train speed goes over 200 km/h (EN1991-2, 2002). Steel and steel-concrete composite superstructures can experience significant dynamic interaction effects, more than other structural solutions, due to their light and slender structures, which mass is comparable to the mass of the train. As resonance phenomena may influence significantly the structural safety and the bridge service conditions, reliable tools for the proper evaluation of structure dynamic performance are needed.

Another innovative aspect in the construction of railway bridges is the more and more frequent adoption of steel and composite steel-concrete solutions characterized by new typologies of local details, designed to improve the local and global fatigue behaviour avoiding those critical factors that caused fatigue failures into previous solutions. To this aim, experimental fatigue tests represent a reliable and suitable tool to confirm design assumptions and to evaluate the real improvement in the fatigue response of new steel and steel-concrete composite solutions (Hoorpah et al. 2004).

In this paper, fatigue assessments of a new steel-concrete double-box composite high speed railway bridge are reported. A suitable procedure, composed of experimental and numerical analysis, was adopted to obtain a reliable evaluation of structure fatigue behaviour. Long-term monitoring data were analyzed to evaluate the real railway traffic spectra over the bridge. Consistent load stress spectra on local details were estimated adopting a suitable dynamic train-bridge interaction procedure coupled with the sub-structuring approach. Experimental fatigue tests permitted to estimate the S-N fatigue life curve of critical details and constituted the basis for the damage assessments of selected structural joints, adopting the Rain-Flow counting technique and the linear damage accumulation law of Palmgren-Miner.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

New considerations on track-structure interaction in railway bridges

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ABSTRACT

Due to the longitudinal interaction between track and structure, the presence of railway bridges may generate added stress on continuous rails and relative displacements between track and deck or substructure. Thus an analysis of these phenomena is needed from the structure design phase on for ensuring proper system behavior.

UIC Leaflet 774-3 and Eurocode EN 1991-2 state methodologies for the analysis of track-deck interaction, as well as actions to be considered and limit values both for stress in rail and displacements.

Although these phenomena may be analyzed theoretically, these reference documents approach interaction computations by means of numerical models that idealize the behavior of all elements and actions involved to obtain stresses and displacements.

The authors have more than ten years experience in these numerical models and in the application of these methodologies to the analysis of viaducts, some of which are singular.

The paper will describe the conclusions of the main research works of the authors as well as the practical experience gathered. Several issues not addressed in current standards shall be described, such as: considerations on deformations due to shrinkage and creep in concrete pre-stressed decks; combination of actions with a comparison of simplified combination methods versus step by step computation methods; special features of interaction with slab track; track-structure interaction under seismic conditions and finally the application of tailored methodologies to studies on special geometries such as skew bridges or underpasses.

Finally other relevant aspects shall be presented, such as the relationship between structural types and their influence on interaction analysis (eventual expansion devices), the issues associated to the use of deck movement control devices (blockers or dampers) and the alternatives, in several limit cases, to the definitive installation of expansion devices (special fastenings, stress relief and provisional expansion devices).

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Comfort evaluation of railway bridge vibration using bridge-train transfer function

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ABSTRACT

Recent trend of structure is that serviceability is more emphasized than stability. While re-searches on vibration serviceability of high speed railway bridge are already conducted and are being applied to designing of railway bridges in Europe and Japan. But the vibration comfort limit of bridge structure by heightening of speed is not clearly defined in Korea.

Therefore, improvement of quality of high speed railway bridge vibration serviceability is required to improve domestic railway transportation competitiveness and to ad-vance to overseas market.

In general, methods of calculating car vibration through bridge train dynamic interaction analysis can be classified into three. The first is to execute mov-ing constant force analysis, and calculate car vibra-tion with the use of vibration transfer function. The second is to use car model expressed with SDOF(single degree of free-dom) system. And the last me-thod is to execute dynamic interaction analysis by using 3 dimensional moving car model system. In the case of 3 dimensional system, various effects like inertial force and car rotation etc may be consi-dered. To evaluate the vibration serviceability of car that runs along bridge, accurate method may be modeling both car and bridge and executing dynamic interaction analysis. However, dynamic interaction analysis that uses such delicate model uses various elements for accurate modeling of actual bridge and car. Therefore, it is difficult to identify which element affects dynamic behavior. Also, the ground to specify the acceleration signal inside car obtained with the use of precisely implemented car model is insufficient. Therefore, inferring the vibration within car with the use of moving constant force analysis and car transfer function is estimated simple and appropriate method.

In this paper, vibration serviceability is defined as the performance to obtain the comfort of railway user for car vibration occurring in the accompaniment of structure deflection. And in designing newly installed structure, it was estimated as the performance to prevent excessive vibration to car by structure deflection. Therefore, the vertical vibration acceleration of car itself that passes through bridge was used as survey indicator. And by modeling 1 passenger car of KHST(Korea High-Speed Train) with moving constant force and SDOF system, 3 dimensional system, dynamic interaction analysis was done, and the responses were compared. From the results, the validity of transfer function was verified, and with the use of it, comfort deflection limit of railway bridge was developed.

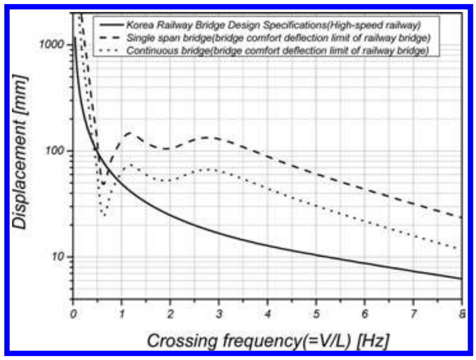

Figure 1. Proposed maximum displacements compared with Korea Railway Bridge Specifications(High-speed railway, 2004).

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Bridges E-2 and E-3 in the new railway to the Northwest of Spain

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ABSTRACT

During the years 2001 to 2007 it has been constructed in Spain the new railway line of high speed (L. H. S.) that connects Madrid with Valladolid, with a length of 179.6 kilometres. The line Madrid - Valladolid has been put in service to the public on December 23rd 2007, and the time estimated to cover the distance is 50 to 55 minutes, which makes an average speed of 215 kilometres per hour, but the railway platform is prepared to reach 350 kilometres per hour.

The cost of the works has ascended to 264.8 million Euros.

The proximities to Madrid of this new L.H.S., it implies the interference with essential services for the capital such as: roads, railways, big pipes of water supply, etc., and the new necessary structures must be constructed without interrupting the corresponding services.

The first bridge near to Madrid which was necessary to constructed was the structure called in the project E-2.

The project of the bridge E-2 was realized by means of a three-dimensional model of finite elements with the program SAP2000N that includes all the elements of the structure.

The solution consisted in the demolition of the current structure of arch and the construction of a new bridge over the existing railway, fulfilling the requirements of vertical clearance of 7.00 m and with a span of 22.64 m needed.

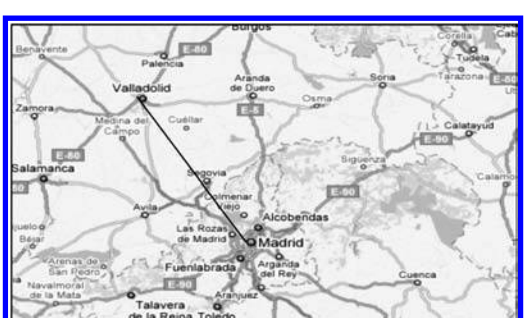

Figure 1. Plan from Madrid to Valladolid.

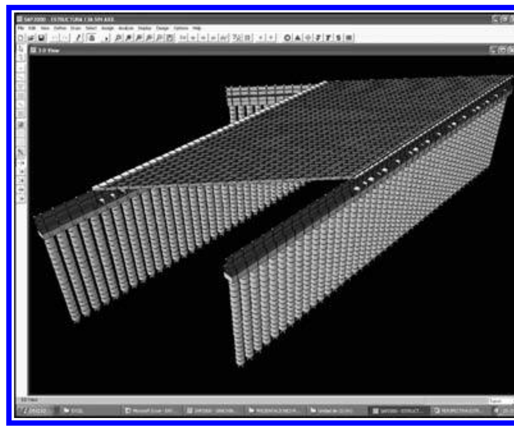

Figure 2. F.E.M. of bridge E-2 with 1203 nodes, 1098 frame elements and 345 shell elements.

Figure 3. The new bridge E-2 and to the bridge E-3A/3B in construction.

The bridge E-3 was divided in two Bridges called E-3 and E-3B. The bridge E-3A has 66 m of length with a span of 22,70 m. and a vertical clearance of 7 m.

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New bridge "in pergola" for the new railway of high speed trains to the Northwest of Spain

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ABSTRACT

During the years 2008 to 2010 it will be constructed in Spain the new railway line of high velocity trains, that will connect Valladolid with the northwest of Spain.

This part of the Spanish Infrastructure Plan will cost 200 millions of Euros with a length about 183 kilometers. The speed will be about 220 kilometres per hour, but the railway platform is prepared to reach 350 kilometres per hour.

The connection between Palencia and Leon comprises 122.20 km and in the middle of it there is the section of Río Cea-Bercianos del Real Camino with a length of 10.5 kilometers.

The present paper exposes the project and construction of one of the most singular viaduct in the section of Río Cea – Bercianos del Real Camino, from Palencia to León, which crosses over the existing motorway M-31. The solution of the bridge has been made as a viaduct "in pergola" with a length of 153 m and with a width of 14.5 m.

The project of the viaduct was made by the author of this paper, with a three-dimensional model of finite elements using the program SAP2000N, that it includes all the parts of the structure, such as: foundations made with piles (4,500 m of piles), mat foundations tying the piles, pillars, and the superstructure comprising a prestressed concrete slab, and the complete two abutments.

Figure 1. New railway line Madrid-Valladolid-León.

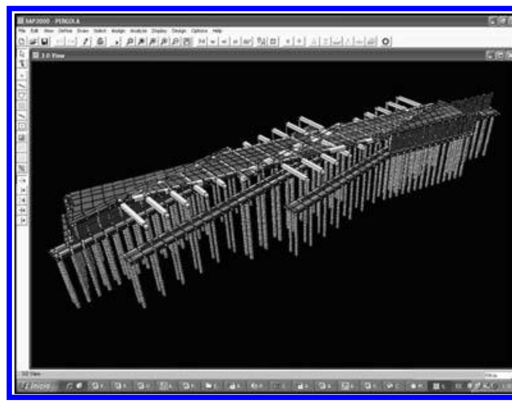

Figure 2. Three Dimensional Finite Element Model of the Pergola from the NE.

Figure 3. Aerial view of the A-231 motorway and the viaduct in Pergola in construction.

This model permits to obtain all the responses of the Pergola such as moments, shears, axial forces, etc. in all the elements. In order to construct the prestressed upper slab it is necessary to cut one of the senses (two lanes) of the highway and to deviate the traffic temporarily for the other sense (two lanes), in order to scaffold one of the parts of the viaduct without any risk for the traffic, and to prestress the slab of the deck. The viaduct is now in construction.

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Fatigue damage assessment of steel bridge members using paint cracking

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ABSTRACT

Since steel bridge members are usually covered with paint coatings, fatigue damage and cracking are developed underneath them. The paint coatings cover up cracks. But fatigue cracks of steel members may be able to be detected using paint coatings. The inspections of bridge are usually conducted by visual inspection. Therefore, if some information whether cracks occur underneath paint coatings were seen from a part of the paint crack, it could be useful. The purpose of this study is to detect damage of steel members using cracks of paint coatings. In the past study, monotony and alternating load experiments of specimens with paint coating cut out from removed highway bridges were conducted. And it was evaluated about damages of steel members and paint coatings in the plastic area. In this study, high cycle fatigue test is conducted using fillet welded joint specimens with paint coating cut out from removed highway bridges after Hyougoken-Nanbu Earthquake, in order to evaluate the damage of steel members using damage of paint coatings as well as fatigue strength of actual bridge members.

The principal results obtained through this study are as follows;

(1) The fatigue strength of vertical stiffener attached portion satisfies Class D recommended by JSSC.

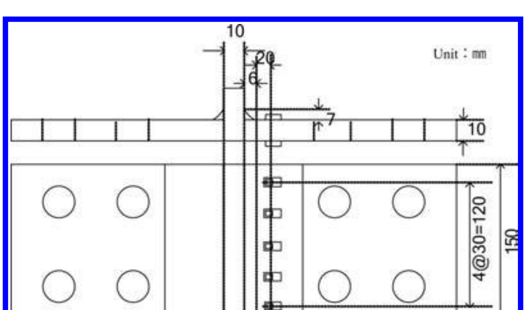

Figure 1. Form and Dimensions of Specimen.

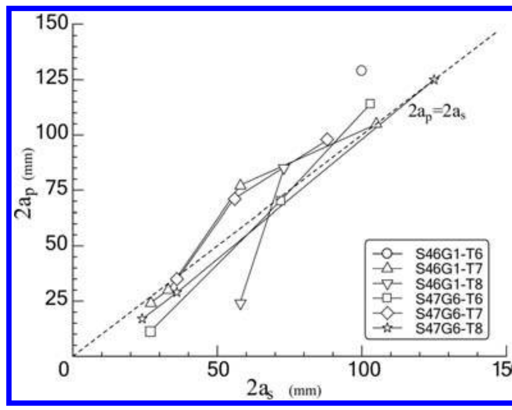

Figure 2. Crack Length in Steel Members and in Paint Coating.

(2) The paint crack develops after the steel crack happens and grows up after that by almost equal length.
(3) The paint crack generation life is almost equal to the breaking life of the welded joint fitting in a low stress long-lived area.

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Movable Scaffolding Systems for 72 m span in one casting stage

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ABSTRACT

The evolution of bridge construction equipments continues and we are now able to produce movable scaffolding systems able to cast in situ spans of 72 m or even larger, with girder weights up to 300 KN/m, in one stage only.

In this article we will present one of these equipments and a 70 m span bridge made with it, the Engano Viaduct, n\ear Santiago de Compostela, Spain.

Some high speed train bridges constructed with these new equipments will also be referred.

After building some heavy bridges for the Spanish High-Speed Train Line with 55 m span between piers, but with 54 m distance between supports for the MSS we went forward for another challenge – the mythic frontier of the 70 meters concrete spans.

The MSS capacity is measured by 3 factors:

– The admissible live load
– The distance between concreting supports of the MSS – concreting span
– The distance between launching supports of the MSS – flying span.

In previous jobs done in 2006-2009 we have used our most advanced models prepared to hold 300 KN/m girders plus the necessary external and internal formwork self weight, for concreting and considering its self weight and the external formwork self weight during launching.

Our models used in Vila Pouca de Aguiar Viaduct (Portugal) and Toxa and Martixe Viaducts (HST) in Spain were capable of supporting 280 KN/m slabs plus 60 KN/m of the formworks, with the supports of the MSS 54 m apart, and flying over 63 m.

For “Viaducto do Engano”, the MSS was previously designed 148 m long, for 63 m or less between concreting supports, with 1/5th span cantilever joints.

This design configuration of the MSS allows it to reach 72 m span between concrete piers if the concrete

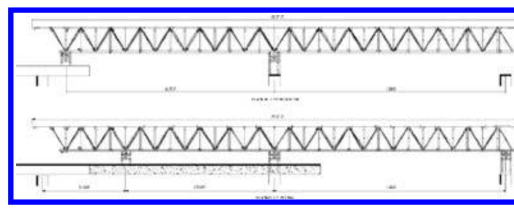

Figure 1. MSS 148 m long with 63 m between supports.

cantilever of the previous span could hold the rear reaction 9 m ahead of the rear pier.

But with an interacting design of both the bridge and the MSS, we concluded that we could use 1/4th of the span joints, with the MSS rear support 16 m ahead of the rear pier, i.e. with 54 m between supports.

This solution being globally cheaper than the previous one, also allowed us to conclude that by using 1/4th of the span construction joints and bringing the rear supports to the edge of the cantilever, the existing machines could also perform 80 m spans, with 63 m between the MSS supports, by placing the rear support 17 m ahead of the rear pier, over a 20 m cantilever.

With this project and some others with very heavy formworks we have learnt how to open smoothly and quickly those formworks, launch the whole machine and close the formwork in very short periods, and we know now how to get to 90 m spans and larger with Moving Scaffolding Systems, casting the full span in one stage.

This construction method already being the cheapest for Viaducts with multiple spans, allows new challenges for the bridge designers and also for the construction engineers providing them with tools that can replace the form travelers, cutting down the construction costs and delays.

We are ready for those new challenges and willing to assist and cooperate with bridge design engineers to allow them to start using this system in their projects and new sites.

SS5: Industrial smart material applications
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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Streicker Bridge: Initial evaluation of life-cycle cost benefits of various structural health monitoring approaches

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ABSTRACT

The Streicker Bridge is a newly built pedestrian bridge at Princeton University campus. The bridge represents the entrance gate to the campus, monument of structural art and is an important landmark. Thus, beyond its functional purpose the bridge also has strong symbolic and aesthetic impacts. The bridge is equipped with various monitoring systems with an overall aim to transform this structure into a multipurpose research and teaching laboratory. An important long-term research topic studied in this bridge-laboratory is the evaluation of life-cycle cost benefits of various structural health monitoring approaches. Four scenarios will be studied over time: global structural monitoring, integrity monitoring, local material monitoring, and various combinations of these three scenarios. At present, since the bridge is new and the project is in its initial phase, only preliminary cost estimations are available and presented in this paper.

Currently implemented monitoring systems are based on global structural monitoring approach and integrity monitoring approach (Glisic & Inaudi 2007). Global structural monitoring approach is based on the use of discrete long-gage average strain sensors, and it features high resolution and accuracy, but its spatial resolution depends on the density of the sensors which is frequently kept low in order to contain the costs. Integrity monitoring is based on the use of distributed deformation sensors with an excellent spatial resolution, but the strain resolution and accuracy are about order of magnitude lower.

The sensor networks for both systems were designed with respect to research and education aims of the project. However, they helped evaluate the cost parameters if similar approaches were used for the bridge safety and maintenance purposes. The cost of equipment for global structural approach will amount to approximately US$ 136,000 and installation will consume approximately 200 man-hours. The equipment for integrity monitoring will be more expensive, approximately US$ 185,000 with some 150 man-hours for the installation. At the time of writing the paper, the cost of material used to build the bridge is approximately US$ 3,100,000 (the bridge is nearly completed), thus the estimated costs of the equipment for monitoring based on global structural monitoring approach and integrity monitoring approach represent approximately 4.4% and 6.0% of the cost of construction material. According to National Safety Council a comprehensive cost of each pedestrian or cyclist fatality is estimated to US$ 3,840,000 (includes medical expenses, victim work loss, public services and lost quality of life), while injury costs on average $52,000. Daily Road User Costs (DRUC) for Washington Road are estimated (Herbsman et al. 1995) to be US$ 4660 per each day the road is closed due to malfunction of Streicker Bridge. The late-repair costs at the present stage could not be evaluated, but they will be studied in the course of the project.

The savings related to pedestrian/cyclist fatality or injury, DRUC and late-repair are considered as quantifiable benefits of monitoring. However, the loss of human life is invaluable, as well as the loss of the bridge considered as a monument of structural art and heritage for future generations, and their pre-servation represents unquantifiable benefit.

The initial evaluation of costs presented in this paper will serve as an input for life-cycle cost benefits of SHM based on global structural and integrity monitoring approaches using deformation sensors. The other approaches and other types of the sensors will also be included in the research.

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Optimal design of bridge SHM systems based on risk and opportunity analysis

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ABSTRACT

Structural Health Monitoring (SHM) aims to provide more reliable and up-to-date information on the real conditions of a structure, observe its evolution and detect the appearance of new degradations. The benefits of having a Structural Health Monitor-ing system installed on a bridge or any significant structure are many and depend on the specific application. Here are the more common ones:

- Monitoring reduces uncertainty
- Monitoring discovers hidden structural re-serves
- Monitoring discovers deficiencies in time and increases safety
- Monitoring insures long-term quality
- Monitoring allows structural management
- Monitoring increases knowledge

When designing a Structural Health Monitoring system, one should always focus on the specific requirements of the structure under exam. To achieve an optimal design it is however beneficial to follow a well-defined and proven procedure.

The first step in the design process consists in identifying the risks and opportunities associated with the bridge under examination. Examples of risks include the probable degradation mechanism due to ageing (e.g. corrosion or fatigue) or external actions (e.g. seismic, impact or overload). Examples of Opportunities include the existence of reserve capacities due to better material properties, synergic effects and over-design. Risks and opportunities are present in both new and existing structures. The purpose of an SHM system is to identify and quantify them, so that the consequences of risks can be avoided (e.g. a collapse) and the benefits of opportunities can be exploited (e.g. safely extending the lifetime of a bridge).

Next, the expected responses to the expected degradations and the effects of the possible opportunities are established and an appropriate Structural Health Monitoring Systems is designed to detect such conditions. Only at this stage, the appropriate sensors are selected. When selecting the best sensors for the specific task, it is often necessary and beneficial to combine different measurement technologies. Once the sensors are installed and verified, data collection can start. If these logical steps are followed and the monitoring data is correctly acquired and managed, data analysis and interpretation will be greatly simplified. This process guarantees that each sensor placed in the structure serves at least one specific purpose and leads to a lean and cost-effective system.

This paper presents a generalized 7-step methodology for designing on optimized Bridge SHM monitoring system and a practical example from a field application: the new I35W Bridge in Minneapolis.

Table 1. 7-step SHM Design methodology.

Step	Properties
1	Identify structures needing monitoring
2	Risk/Uncertainty/Opportunity analysis
3	Identify responses
4	Design SHM system and select appropriate sensors
5	Installation and Calibration
6	Data Acquisition and Management
7	Data Assessment and analysis

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Smart textiles and their application in bridge engineering

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ABSTRACT

The materials of tomorrow will be multifunctional serving multiple roles and crossing multiple disciplines. Sensor-embedded load bearing elements will be able to transmit data, report in-service load conditions, and enable the assessment of health as well as perform their traditional role. Already used extensively in construction and in composites, textiles make an excellent candidate for a smart material. Their manufacturing process is well suited to the direct integration of fiber optic sensors through weaving and warp-knitting. As such, large-area sensor-embedded textiles can be manufactured offsite, are mass producible, and are inherently cost effective. For the engineer, they can provide both reinforcement and monitoring capabilities across an array of applications that span both new and the retrofit of existing engineered structures. In doing so, in-service data can be utilized to validate design assumptions, to control construction operations, to assist with life-cycle management actions, and as a mechanism to provide alert during extreme events. This paper highlights recent progress in the development and testing of sensor-embedded textiles within the European research project "Polyfunctional Technical Textiles Against Natural Hazard" (www.polytect.net) with special emphasis on the consideration of smart textiles in bridge engineering. With respect to bridges superstructures, smart composites and smart textiles open the possibility to reinforce existing bridges economically, to construct new bridges with high performance light weight materials, and to manage maintenance intensive bridge superstructures more optimally. With respect to bridge substructures, smart textiles provide a tool to address challenges associated with weak soils, to increase safety and to reduce risk at worksites and to revisit the underlying assumptions present in existing codes and guidelines to ensure consistent levels of reliability across the foundation, abutments, and superstructure. Also particular to the substructure and foundation elements is the possibility of their reuse when bridge superstructures are replaced. In such cases, their load history and performance over time can be critical in deciding whether or not they can be reused or must be reconstructed.

Figure 1. Textile-matrix-stone composite (courtesy of the Karlsruhe Institute of Technology and Selcom Multiaxial Technologies).

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Conical couplers for non-pigtailed, free-space optical coupling to fiber optic sensors for bridge monitoring

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ABSTRACT

Traditional structural monitoring utilizes electro-mechanical sensor system. However, in recent years, innovative fiber optical sensors (FOS's) have attracted intense interest. Specifically, fiber Bragg grating (FBG) sensors have shown great utility for safety management of bridges. They can be either embedded into bridges or simply mounted onto the surface to measure various parameters. One major drawback of the embedding application, however, can be the need for extending a fiber "pigtail" off the structure for mechanical coupling with the optical source and detector. Therefore, we have developed techniques of free space optical coupling to FBG sensors, including the machining of 45-degree mirror in the fiber. In this technique, we have developed spliced multimode coupler/ single mode FBG sensor fibers and embedded them into composite panels, and used free space optical coupling to interrogate the sensor and measure mechanical strain. However, it is impractical to machine the mirror after the fiber is embedded into the bridges. It is also difficult to maintain the orientation of the 45-degree mirror if it is polished before being embedded. Therefore, in this paper, we have developed rotationally symmetric concave conical optical couplers, that couple light independent of the rotational state of the fiber with respect to the outside of the structure and the optical source. We theoretically studied the conical couplers on both plastic optical fiber (POF) and multimode optical fiber (MMF), using computer simulations. In our preliminary experimental attempt, we were able to machine the conical coupler into a POF with a core diameter of about 3 mm. We also have been able to couple free space light into FBG with the conical coupler on MMF. We believe that with further modification to the current system, we will be able to also couple the light out of the FBG through the conical coupler. Therefore, we conclude that this approach could potentially solve the problem of fiber pigtailing in structure monitoring with FOS's.

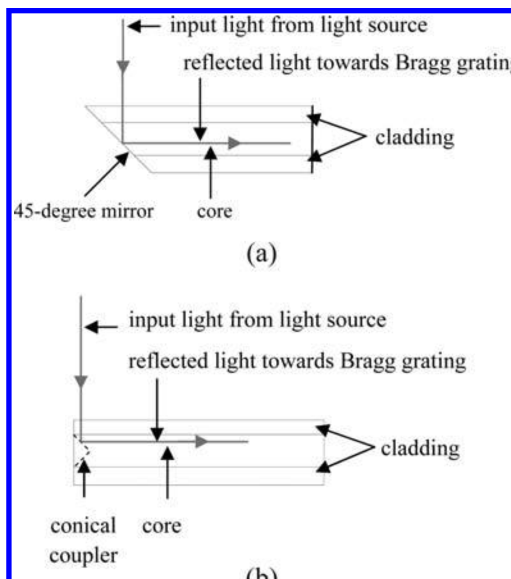

Figure 1. Free space coupling to FBG: (a) with 45-degree mirror; (b) with conical coupler.

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SS6: Advances in structural robustness:
dependability framework
Organizer: F. Bontempi

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Dependability of complex bridge structural systems

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ABSTRACT

Modern structural design in Civil Engineering has changed and has to change furthermore to cope with the actual challenging superstructures and large infrastructures which are now of interest for the Society. Invariably, Society asks for more demanding safety and performance levels, while ever-increasing interest is focused on durability of constructions (life cycle) and accidental resistant capacity, widening then both the time horizon and the event possibility.

All these requirements are, in some sense, in contrast with simplified or even over simplified design formulation and assessment, unfortunately sometimes too much emphasized, by traditional standards and codes. These approaches are in fact devoted to generally simple and habitual design situations and cannot be extended beyond this field of application, as sometimes regrettably proposed.

This paper will summarize the basic ideas that form, in the Authors experience, a modern approach to structural design. One can consider:

1. the emphasis on the systemic nature of structures, that leads the concept of structural systems with its indispensable hierarchical organization;
2. the importance on the ordered formulation of the design problem, taking methodically into account both the qualitative and the quantitative aspects of the problem, leading to the so-called Performance-based Design;
3. the necessity to have a clear vision of the complexity of the design problem;
4. the need to formulate in a very general way the requirements for structural quality introducing the composite concept of dependability.

In the paper, some reference is made to the proposed design for a long-span suspension bridge.

Finally, it is believed that these ideas can lead not only to a proper development of future constructions but also (and perhaps more important) to a way to educate engineer students to develop a wider point of view of the design problem.

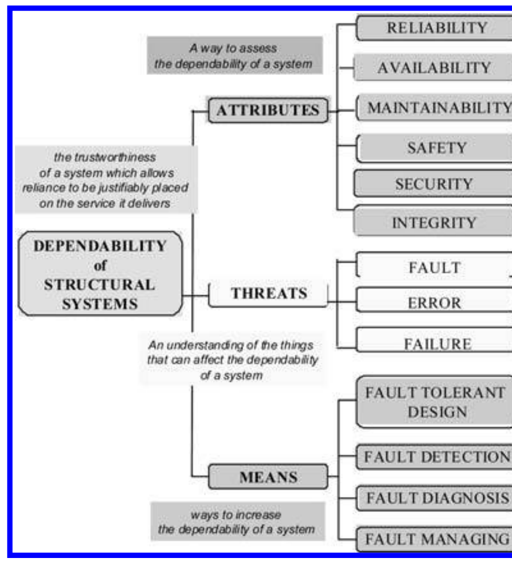

Figure 1. Main aspects of dependability.

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Multilevel structural analysis for robustness assessment of a steel truss bridge

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ABSTRACT

In recent decades, robustness and collapse-resistance have become significant aspects to consider in the design of structures. While engineers always aim to avoid them, failures do occur and may indicate a lack of robustness in a structural system. It becomes important to understand both the approach to the analysis of potential local failures and to determine their effect on the overall structure

The statically determinate trusses in actual bridges have some redundancy mainly due to continuity at connections. In the current open debate about robustness and collapse-resistant design the effect of such redundancy becomes important. In the present paper it is taken into account by a multilevel analysis going from the global to the local level. This approach is illustrated by application to the truss portion of the recently collapsed I-35W Minneapolis Bridge which has redundant reactions but is a functionally non redundant structure.

The concepts of redundancy, progressive collapse and robustness are closely intertwined. In order to assess robustness in a complex structure it is necessary to understand not only the global interaction of the structure with the surroundings but also the behavior at various local levels. Approaching the problem using a multilevel perspective, the global system has been broken down in order to perform a detailed analysis and evaluate the system performance at global and local levels.

The analyses have been performed using a 3D finite element model of the case study bridge. After bridge subjected to seasonal and daily temperature changes as well as construction loads, one built up section member is analyzed for local failure in compression. For the local model, the numerical results of a buckling analysis obtained by detailed FEM analysis are compared with results obtained from simple formulas.

The multilevel analysis is illustrated for the truss structure of the I-35W Bridge for one particular assessment: the response of one bottom chord segment of the main truss. Such a procedure can be applied to investigate not only potential sources of the collapse of that bridge but also in assessing robustness of similar structures.

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A framework for robustness assessment in the context of corroded RC structures

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ABSTRACT

Structural robustness seems to be an emergent concept related to the structural response to damage. At the present time, robustness is not well defined and much controversy still remains around this subject. Even if interest on robustness has seen growing as a consequence of catastrophic consequences due to extreme events, the fact is that the concept can also be very useful when used on more probable exposure scenarios such as deterioration. This paper intends to be a contribution to the definition of structural robustness, especially in the analysis of reinforced concrete structures subjected to corrosion. To achieve this, a deterministic measure for robustness is proposed given by the area below the curve defined by the normalized structural performance f subjected to a normalized damage d.

To illustrate the proposed concept, an example of a corroded reinforced concrete foot bridge was analyzed using nonlinear analysis numerical methods based on the continuum strong discontinuities approach and isotropic damage models for concrete. The methodology consisted on a two step analysis proposed by Sánchez et al. (2008). On the first step a cross section analysis was performed in order to capture deterioration and cracking of concrete due to expansion of corrosion products (see Figure 1).

On the second step, a 2D longitudinal model of the corroded structure was built based on the results obtained from the cross section analysis. Concrete was modeled using a composite material approach proposed by Oliver et al. (2008). Reinforcement was modeled using a slipping fiber model upgraded with the M-pull model proposed by Bhargava et al. (2007) in order to predict bond strenght deterioration. For corrosion levels from 0% to 100% load carrying capacity was predicted using the referred methodology (see Figure 2).

Finally structural robustness of the corroded structure was assessed by computing the area below the normalized load carrying capacity in Figure 2.

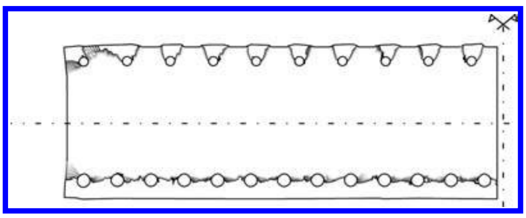

Figure 1. Isodisplacement lines showing cracks on concrete.

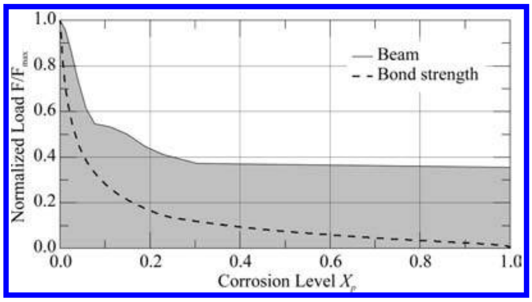

Figure 2. Normalized peak load carrying capacity as a function of the corrosion level X_p.

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Buckling of steel gusset plates

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ABSTRACT

As a result of the structural failure of the **I-35W** bridge in Minneapolis in 2007, FHWA issued guidelines for the load rating of gusset plates and recommended that the capacity of these plates on non- load-path-redundant steel truss bridges be verified. The intent of the guidelines is to be safe and simple, i.e., amenable to hand calculations. According to the guidelines, the compressive strength of a gusset plate is obtained from column strength curves in the inelastic range, which assume an initial out-of-straightness of $L/1500$. The purpose of this paper is to examine the buckling behavior of steel gusset plates in greater detail, accounting for parameters that were not explicitly included in the guidelines, such as initial deformations of the gusset plate, stiffness of the framing members, load distribution from the framing members to the plate and load eccentricity. To this end, a finite-element model of a gusset plate was developed and verified against experimental measurements. Results show that the **FHWA guidelines** for load rating are conservative and safe for larger initial out-of-plane deformations, up to one plate thickness. This is true for in-plane compressive loads with no moment and no eccentricity.

Experimental results were obtained from tests of gusset plates performed at the University of Alberta. The test parameters included thickness and size of the gusset plate, as well as angle, moments and restraint conditions of the framing members. **Finite-element results** show, as expected, that the **stiffness of the framing member** plays an important role in the buckling strength of the gusset plate. The experimental value falls between Case B, where the flange thickness of the framing member is added to that of the gusset plate, and Case C, where both the flange and the web of the framing member add to the stiffness of the gusset plate.

As **initial imperfections** increase in magnitude, the plate strength decreases. For case B and edge loading, the unfactored Load Factor Rating (LFR) or Load Resistance Factor Rating (LRFR) strength is acceptable up to an initial deformation of $1t = 13.3\,\text{mm} = 310\,\text{mm}/23.3 = L/23.3$, although the plate would have to undergo an additional out-of-plane deformation of the same order before this strength is reached. Initial imperfections in excess of $1t$ would be unsafe. For the more realistic Case C and edge loading, the LFR or LRFR value is quite conservative, even for an initial imperfection of $2t = 26.6\,\text{mm} = 310\,\text{mm}/11.65$.

The finite-element model was not refined enough to determine the load carried by individual bolts, so various **load distributions** were assumed. The assumption that the compression is carried by the first row of bolts (closest to the plate edge) produces results that are very similar to the previous case of edge loading, except for the almost perfect plates, which show a small strength increase. As initial deformations increase, the effect of a small change in column length decreases. A triangular (or linear) load distribution was also studied, with the first row of bolts (near the edge of the plate) carrying the highest load and the last row of bolts (near the center of the plate) carrying no load. Here the plate buckling strength is considerably greater than for edge loading, and the LFR or LRFR value is adequate even for an initial deformation of $2t$.

The LFR or LRFR value cannot be achieved even for an almost perfect plate loaded by a wide flange (I) section framing onto *one* side of the plate. The load is applied with an **eccentricity** equal to half of the depth of the I-section, at a location corresponding to the first row of bolts. Initial deformations of $1t$ and $2t$ produce negligible effects compared to the load eccentricity. When load eccentricity and initial imperfections act in opposite directions, load-induced deformations can reverse direction and allow the plate to achieve much greater strength than for a nearly straight plate.

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Cable-loss analyses and collapse behavior of cable-stayed bridges

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ABSTRACT

The general aim in designing structures, where the consequences of a collapse are high, must be collapse resistance. This means that no structural damage should develop that is disproportionate to the triggering event. Generally, structures can be made collapse resistant by ensuring a high level of safety against local failure or by designing for the failure of elements and thus increasing the robustness. Increasing the robustness of cable-stayed bridges is achieved by means of designing for the loss of cables.

This paper investigates the loss of any one cable by nonlinear dynamic analysis of a three-dimensional model of a cable-stayed bridge. Dynamic amplification factors for quasi-static analyses are determined. The aim of these analyses is to give advice on how to determine this dynamic amplification factor, to prove if the use of a uniform amplification factor to calculate the maximum responses to cable loss is valid and if reductions of this factor are generally possible.

The results show that a unique dynamic amplification factor cannot be specified. Instead, the dynamic amplification factor depends on the location of the ruptured cable and on the type and location of the state variable being examined. Using a factor smaller than 2.0 is only possible for the bending moments in the bridge girder. Here, an explicit calculation of the DAF can be beneficial. A dynamic amplification factor of 2.0 is necessary for the safe design of the cables. Regarding the bending moments in the pylons, large dynamic amplification factors result because of large dynamic responses resulting from a complex excitation. Dynamic time-history analyses are therefore recommended, at least for the cable loss cases which yield the highest responses. These are generally the longer cables but not necessarily the longest cable.

The dynamic amplification factor can be determined in linear dynamic analyses. Different live load positions do not have to be considered.

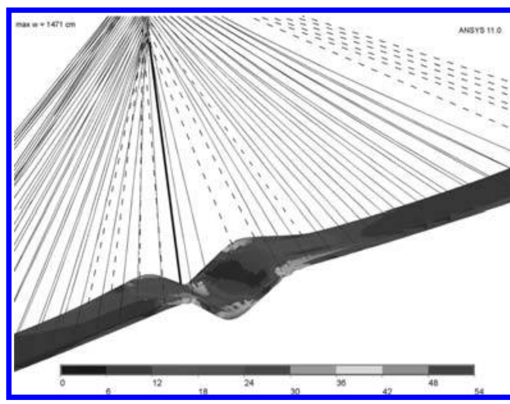

Figure 1. Instability failure of bridge deck after the loss of three short cables.

Additionally to having appropriate analysis tools for creating robust structures, it is important to know which structural properties are important to increase structural robustness. These properties are identified by investigating the collapse behavior of a cable-stayed bridge after the loss of cables. The results show that the normal forces in the bridge girder is the collapse promoting attribute of a cable-stayed bridge. For this reason, self-anchored cable-stayed react less robust to the loss of short cables where the normal force in the bridge deck is highest. For the investigated cable-stayed bridge, two adjacent short cables in one cable plane plus one cable in the second cable plane can fail without disproportionate collapse. In case of the failure of three adjacent cables, the bridge collapses due to instability failure of the bridge deck (Figure 1). Other collapse scenarios are presented, too.

The robustness of the bridge can be increased by preventing instability. This is possible by increasing the stiffness of the bridge girder or by reducing the unsupported length, which means closer cable spacing. The former is recommended here due to a higher failure probability of closely spaced cables.

SS8: Nondeterministic schemes for structural safety & reliability of bridges

Organizer: S. Arangio

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Degradation history simulation – a tool for assessment of structural lifetime

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ABSTRACT

For several years the number of reinforced concrete bridges, which show a critical state of preservation due to degradation processes, is significantly increasing. In these cases structural experts are challenged to decide whether or not a structure can definitely resist an ongoing unrestricted usage, might be upgraded by repairs, or must be sumptuously replaced by a new one.

Today, the use of the finite element method is well proven in application of deterministic structural analysis. However, reinforced concrete exhibits a strong nonlinear behavior and thus demands an application of a complex nonlinear material model.

Thereby, reinforced concrete contains many sources of uncertainties essentially affecting its mechanical properties. For nonlinear finite element simulations of structural lifetime in general, especially in the case of prediction of (residual) lifetime, these scattering properties should be considered carefully. Beneath material properties, relevant damage driving forces introduce even more uncertainty to the model. Although well founded methods of nonlinear and probabilistic structural mechanics are separately at disposal, their combination still causes great difficulties with respect to efficiency. In this context, Latin Hypercube Sampling (LHS) enables to decrease notably the number of simulations required, still ensuring an adequate representation of the underlying probability distributions of relevant input parameters. Accordingly, the spatial scatter is mapped onto the finite element mesh by random fields.

The present work transfers the above mentioned theoretical framework to a complex engineering structure – a reinforced and pre-stressed concrete arch bridge located in Germany – to demonstrate its applicability in practice. The bridge, built in 1951 two years before the first standard on pre-stressed structures has been published in Germany, has been deconstructed in 2006 owing to an expertise, which identified loss of pre-stress due to fatigue corrosion of tendons responsible for a critically reduced bearing capacity (Petryna et al. 2009).

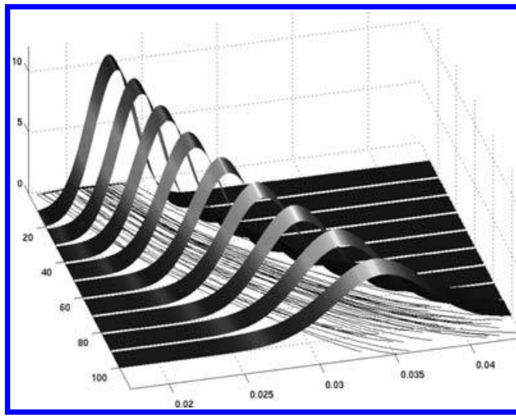

Figure 1. 3D time-deflection curves of 50 nonlinear simulations with respect to uncertain corrosion rate and geometry as well as fitted distribution of deflections each 10 years.

A finite element model of the bridge structure is simulated, until failure due to time-dependent degradation mechanisms occurs. Sensitivity of lifetime, with respect to relevant uncertain parameters, is studied (Fig. 1). The distribution of simulated lifetime response is established by hypotheses testing. Concerning structural lifetime, uncertain but irrelevant parameters are identified. Their exclusion from stochastic data base enables to reduce the variability of response. Further, the framework of RSM enables to establish a regression model, which detaches expensive nonlinear simulation from prediction. Confidence intervals of lifetime prediction capability of the proposed model as well as tests of model adequacy are performed.

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Non deterministic approaches in the current structural codes for assessing the safety and reliability of bridges

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ABSTRACT

The main goal of bridge agencies is to ensure the safety of the traveling public and the health of the economic cycle by maintaining a safe bridge network system and extending its useful life at minimal life cycle cost. To achieve this goal, bridge agencies select an appropriate bridge management approach that may include bridge instrumentation for health monitoring purposes. The implementation of such monitoring systems requires the availability of damage assessment models to evaluate the degree of structural degradation that would trigger a warning mechanism when a bridge reaches a critical state. Health monitoring systems include instrumentations for the non destructive evaluation of materials, load testing, corrosion detection, and analytical models for using the collected information for load rating. To this end, appropriate criteria are needed to identify threshold values that separate the damage state into critical and noncritical stages.

Different approaches exist for assessing the performance of bridges. The most commonly used method of bridge evaluation is the so called condition rating method. All agencies perform such condition ratings on a regular basis. Condition ratings can be either numerical ranging between 1 for very poor condition to 10 for excellent as an example or descriptive by classifying bridges as poor, acceptable, good, etc. But, using the results of the inspection in a comprehensive and objective way has been a big challenge. In fact, the same bridge, assessed by two different engineers can be rated with different grades. In the past three decades, a new measure for the assessment of existing structures has been developed within the probabilistic framework based on the reliability index. From a probabilistic point of view accepting existing structures as sufficiently safe, becomes the result of a decision-making process guided by some optimality criteria.

The condition rating and the reliability-based performance assessment techniques serve the same purpose but have different theoretical bases and completely different formulations.

In the first part of the paper, the advantages and drawbacks related to both the measures are discussed and it is shown that, under conditions of uncertainty, bridge management systems should be reliability based. In fact, it is through the probabilistic approach that the uncertainties can be reflected properly in bridge management and strategies that integrate capital and maintenance activities at the lowest expected life-cycle cost can be determined.

The second part of the paper gives an overview of existing codes and guidelines in Europe and the USA. In the USA, a unified condition assessment procedure has existed for a long time. In Europe, there are currently different condition assessment methods in each country. Harmonization of these methods is needed in the near future. Bridge authorities must adopt proven methods to assess the safety and reliability of existing bridges in order to make appropriate provisions for more refined maintenance methods.

In the last part, different methods for the evaluation of reliability indices of existing structure are discussed. It is also pointed out that redundancy and robustness of the structure should be considered.

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Basic aspects for the uncertainty in the design and analysis of bridges

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ABSTRACT

The aim of this paper is to remark basic ideas about uncertainty related to structural problems connected with the design of bridges and to devise suitable strategies that can be pragmatically applied in real problems.

The main ideas about risk, evolutive vs. innovative design (Fig.1), high probability – low consequences (HPLC) vs. low probability – high consequences (LPHC) (Fig.2) have been firstly presented, while two well-known case histories have been then pertinently reported.

Both the ideas and the examples appear simple or even trivial. Nevertheless, it seems that there is not sufficient consideration for the features connected (Handling Exceptions 2008).

One possible reason is that the strategies necessary to handle innovative design or low probability

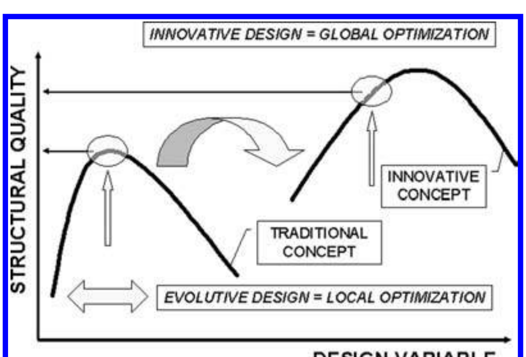

Figure 1. Evolutive/Innovative designs in the (performance – parameter) space.

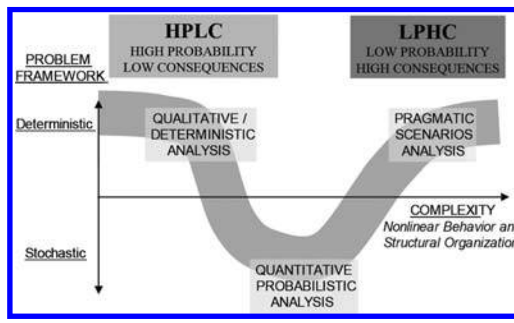

Figure 2. HPLC vs. LPHC situations and corresponding analysis strategies.

high consequences scenarios are not easily amenable of analytic treatment.

Fortunately, these situations can be handled with engineering knowledge and can be in the future eventually solved by Artificial Intelligence methodologies.

Finally, for the operative point of view, the deep consideration for robust structural systems appears indispensable (Starossek, 2009).

It is expected that this material can be useful for students and practitioners, more than for researchers and scholars.

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Block shear failure of steel gusset plates

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ABSTRACT

Following the catastrophic failure of the I-35W bridge in Minnesota in 2007, the Federal Highway Administration issued guidelines for the load rating of bolted and riveted gusset plates in truss bridges (FHWA, 2009). This paper develops finite-element models capable of predicting the behavior of gusset plates in tension, resulting in possible failure by block shear, for comparison with the guidelines. Block shear failure is a limit state that combines tension failure on one plane and shear failure on a perpendicular plane. The analysis accounts for the nonlinearity of the material and large displacements. For validation, Plate 1 simulates a physical test by the University of Alberta (UA). The material is bilinear elasto-plastic, with Young's modulus of 215 GPa, yield strength of 410 MPa and tangent modulus of 2.15 GPa. The analysis uses true stress and true strain and the tension load is applied by three point loads on the upper half of each bolt hole. Results from the present two STRAND meshes agree well with the ABAQUS model used by UA. All three finite-element results slightly underestimate the test results, especially at the onset of yielding. The Load Resistance Factor Rating (LRFR) value falls on the limit of the elastic range, whereas the unfactored FHWA value produces a small amount of yielding.

Plate 2 has a tension length L_t of 0.139 m, a shear length L_v of 0.358 m and 16 bolt holes arranged in two rows. Since net tension area $A_{tn} < 0.58\ A_{vn}$ net shear area, failure is by block shear. Assuming as failure criterion a maximum strain of 100%, results show an ultimate load of 2720 kN at a displacement of 14 mm, and thus the LRFR value provides a factor of safety of ultimate load/LRFR value of 2.15.

The bolts in Plate 3 are distributed over a narrower width. This gusset plate has a tension length L_t of 0.0462 m and a shear length L_v of 0.358 m. Again, since $A_{tn} < 0.58\ A_{vn}$, failure is by block shear. The ultimate load is 2088 kN at a displacement of 14 mm, and thus the LRFR value provides a factor of safety of 2.14.

Plate 4 has tension length L_t of 0.150 m and shear length L_v of 0.273 m. In this case, $A_{tn} > 0.58\ A_{vn}$ and failure is by tension. The ultimate load is 2190 kN at a displacement of 7.8 mm, and the LRFR value provides a factor of safety of 2.38.

Thus, Plates 2 and 3, which failed by block shear, even though they had rather different hole arrangement, behaved very similarly. Plate 4, which failed by tension, had a slightly higher factor of safety but less ductility than Plates 2 and 3. In all cases, the FHWA values are safe and adequate.

Compared to Plate 2, Plate 5 has a third row of bolts. The ultimate load is 3468 kN at a displacement of 47.3 mm, and the LRFR value provides a factor of safety of 2.73. Results show the beneficial effect of adding internal bolts, which increase the strength and ductility of the gusset plate. A simplified analysis that only accounts for the perimeter bolts would underestimate the strength and ductility of the gusset plate.

Finally we analyze one of the example plates from the guidance document. Its ultimate load is 10750 kN at a displacement of 43.2 mm, and thus the LRFR value provides a factor of safety of 2.28.

In conclusion, nonlinear finite-element analysis validated by experimental data confirms the safety and validity of the FHWA load rating formulas for the block shear strength of riveted and bolted gusset plates. For a variety of geometries, the Load and Resistance Factor Rating (LRFR) value produces factors of safety between 2.1 and 2.7. This study also provides guidance on the mesh density required around the holes, the application of bolt loads, and the approximation involved in modeling the perimeter holes only.

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Generation of modified earthquake time-histories using Hilbert-Huang transform

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ABSTRACT

Seismic Response History Analysis (SRHA) is a major analysis method for seismic qualification. Representative earthquake time-histories are required for this analysis method.

There are three types of earthquake time-histories: actual earthquake records, modified earthquake time-histories based on real earthquake records, and artificial earthquake time-histories. In eastern North America (ENA) area, there are very few actual earthquake records available.

According to existing earthquake-resistant codes (e.g. ASCE 2005, CSA 2006), modified time histories compatible with the target design spectra are required. It is thus practically important to be able to generate modified time-histories based on the few available earthquake records.

Existing modification method (ASCE 2005) is to scale the Fourier amplitudes of the actual record such that the resulting response spectrum is compatible with the target design spectrum.

However, it is difficult and not practical to use this method to generate compatible time-histories because of the deficiencies of the Fourier transform and the complexity of the procedure.

In this paper, a new modification method, which can generate a modified time-history compatible with the target design spectrum based on an actual earthquake record, is proposed. The actual earthquake record is decomposed to obtain several components of amplitude and frequency by Hilbert-Huang Transform (Huang et al. 1998). The compatible time-history is then constructed using optimization technique to minimize the difference between response spectrum of the modified time-history and the target design spectrum. The results obtained show that the proposed modification method is simple and credible in generating the required compatible time-histories.

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Influence of model parameter uncertainties on the seismic vulnerability analysis of continuous steel-concrete composite bridges exhibiting dual-load paths

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ABSTRACT

The performance of multi-span steel-concrete composite bridges in recent seismic events has shown that these structures are very sensitive to earthquake loading. Following the Performance-Based Earthquake Engineering (PBEE) framework, the vulnerability assessment of these systems must take rigorously into account all pertinent sources of uncertainty, including uncertainties in the loading and in the structural/mechanical/geometrical properties (model parameter uncertainty). The model parameter uncertainty affects not only the structural capacity, but also the Engineering Demand Parameters (EDPs). However, most of the procedures for fragility calculation focus on the variability of EDPs due to input ground motion uncertainty (record-to-record variability) but neglect model parameter uncertainty effects, or incorporate these effects only in a simplified way. This paper aims at studying the effects of model parameter uncertainty on the seismic response and on the seismic vulnerability of steel-concrete composite bridges with abutment transverse restraints. In this paper, the Extended Incremental Dynamic Analysis (EIDA) method (Vamvatsikos & Fragiadakis 2009) is used to account for all sources of aleatoric uncertainty. EIDA is applied to a benchmark SCC bridge system with dual load path (Figure 1) whose seismic response has already been investigated in Tubaldi et al. (2009). By means of EIDA, the first- and second-order statistics of the EDPs of interest are estimated, the sensitivity of the structural response to both model parameter uncertainty (i.e., uncertainty, U) and ground motion uncertainty (i.e., randomness, R) is assessed, and the seismic vulnerability accounting for all pertinent sources of uncertainty is evaluated.

In Figure 2, the system fragility curve accounting for seismic ground motion uncertainty only (denoted by R) is compared with the system fragility curve

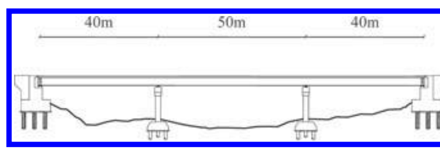

Figure 1. Bridge longitudinal profile.

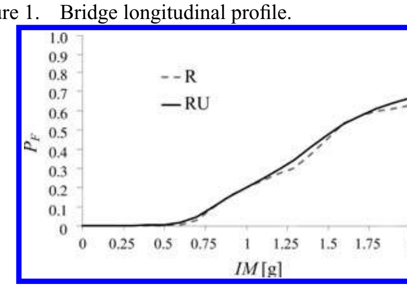

Figure 2. System fragility curve accounting for randomness only (R) and randomness and uncertainty (RU).

accounting for both randomness and model parameter uncertainty (denoted by RU).

It is observed that, for the considered benchmark structure, (a) the effects of seismic ground motion uncertainty are predominant compared to model parameter uncertainty, (b) model parameter uncertainty has a negligible effect on the system fragility, and (c) the correlation between different failure modes is significant.

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Reliability evaluation of complex bridges under multiple limit states

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ABSTRACT

Efficient maintenance, repair and rehabilitation of existing bridges require the development of a methodology that allows for an accurate assessment and prediction of bridge performance. Reliability concepts and methods have been used to quantify the structural performance when uncertainties are involved. However, there are very few successful real-life examples on the combined use of reliability theory and structural health monitoring (SHM) in bridge management. Therefore, it is a great challenge to develop an effective reliability analysis method which can be used together with SHM in bridge management systems.

In this paper, a radial basis function (RBF) is proposed to simulate the response of complex bridges, and the uniform design method is used to construct the training datasets for the RBF network (Zhu & Xiao 2007, Zhu et al. 2007). The finite element software ANSYS is used to obtain the structural response of complex bridges. The RBF neural network-based meta-model is combined with Monte Carlo Simulation (MCS) in order to improve the computational efficiency of reliability analysis of complex structures.

The main objective of this paper is to present a RBF-MCS method for reliability evaluation of complex bridges under multiple limit states. A long span steel truss arch bridge, the Guotai bridge (see Figure 1) has been analyzed in order to illustrate the efficiency of the proposed methodology for multiple limit state reliability evaluation of complex bridges. The Guotai bridge with the main span of 146 m opened very recently (May 2009), It is located in Tianjin, China. The results of the reliability evaluation can be used to establish a cost-effective health monitoring for this bridge.

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Figure 1. Guotai Bridge in Tianjin.

SS10: Life cycle bridge engineering in Korea

Organizers: H.-N. Cho & J.-S. Kong

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Study on bridge asset management in Korea based on infrastructure asset management methodology

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ABSTRACT

Recently, researchers in the field of bridge maintenance and management have tried to enhance the satisfaction of both users and agencies of social infrastructures by applying the asset management concept. Asset management of bridges is clearly a national matter that must be set up in preparation for a possible sharp increase in maintenance costs in the near future. Nevertheless, the asset management has still remained incomplete, given the lack of relevant government supports or policy in Korea. Therefore, it is urgent to carry out in-depth studies to build a bridge management system at the asset management level by establishing a support system for making decisions such as the formulation of bridge maintenance strategies and the logical budget allocation. In Korea, diverse efforts have recently been performed to build an asset management system for the entire infrastructure facilities.

This study is to develop more efficient asset management framework for bridge management. The basic concept of a bridge asset management is defined and the core technologies to establish systems are proposed by analyzing the current status and functions of the existing bridge and asset management systems both in and out of Korea. In addition, the bridge asset management process is suggested and major functions of the suggested processes are defined considering the domestic circumstances in Korea. This study also defines a diversity of LOS (level of service) of bridges with respect to the accomplishment of the local communities and customer values, and clarifies performance measures for quantitative assessment of the LOS. By dividing asset management of bridges into the establishment of strategies and that of plans, various decision-making subjects are outlined.

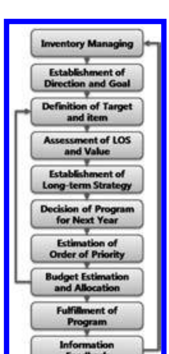

Figure 1. Bridge asset management process.

Table 1. Definition of bridge LOSs.

Customer value	LOS (Level of Service)
Sustainability	Provide sustainable services without adverse function to the environment
Approachability*	Allow users to approach bridges without restrictions
Validity of costs**	Efficiently manage bridges
Quality*	Validate bridge performance of basic functions
Health & safety*	Let users safely use bridges
Reliability and responsiveness**	Provide predictable and continuous services Minimize impact by promptly counteracting emergencies
Customer service**	Kindly respond to service demand Minimize civil complaint

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Consideration of safety in the revision of the bridge management system in Korea

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ABSTRACT

The bridge population in Korea was already large and ageing. At that time, little consideration was given to assess the bridge performance with safety. This situation may lead to loss of maintenance cost at a national wide due to no considerable strategy based on safety of the bridge. Therefore it is growing awareness that requirement of long-term bridge performance evaluation for bridge management in the future, including application of LCC in Korea. However most of Korea bridge management system has not considered performance degradation with safety to evaluate bridge condition.

Such various condition state evaluation methods as eye inspection, load carrying capacity evaluation, periodic inspection, diagnostic workup and movements monitoring analysis etc. have been developed for efficient maintenance of social infrastructures, however, these methods are mostly remained in the initial stage of research and not enough to use for the engineers in the field because the methods are too complicated or simplified to be applied.

These systems, collectively named Bridge Management Systems, have been developed with varying degrees of sophistication. Early system were mainly databases which stored information collected during inspections together with inventory data such as the location of the bridge, road category and the construction date of each structures, then, over time, more functions have been produced and systems have been developed which enable the user to undertake activities such as inspection planning, deterioration prediction, structural assessment and the economic evaluation of repairs

To derive the optimum maintenance scenario, it is needed to analyze safety of bridge caused by deterioration but standardized safety deterioration models were not available in general.

So under these circumstances, the standardization of performance degradation and BMS with consideration of safety is immediately necessary to develop.

In this study, the current status of bridges in Korea has been investigated through arrangement of safety variation with/without maintenance activity. Also we have developed practicable performance degradation model classified by some typical type of bridges and members to derive optimum maintenance scenario considering safety. The model has been developed based on analytical/computational framework for statistical regression. And the revision of Bridge Management System have considered safety model to manage bridges efficiently

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Development of life-cycle analysis based new bridge management system in Korea

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ABSTRACT

Different to other social infrastructures, bridges can be collapsed and this may cause death toll and severe social and economical damage. The occasional collapse of bridge makes aware of the importance of accurate safety inspection and the establishment of budget for performing the appropriate repair and reinforcement as well as the acknowledgement of error in design and maintenance. Therefore, it is necessary to establish the rational and scientific decision making method which can decide appropriate maintenance budget based on bridge life-cycle condition and performance. The function of bridge management can be briefly divided into three categories, performance management, asset management and decision making including life-cycle analysis through the status analysis of each country's research development.

Recently, Ministry of Land Transport and Maritime Affairs (MLTM) of Korea has re-searched on the existing bridge management system (BMS) and its improvement. The current BMS for national road managed by MLTM was developed in late 1990's. However, its computational operating system was only changed but its performance and function have limited in simple function such as inventory and inspection data storage. For developing new BMS with the analysis of research trend, the research has been developing classified into the performance evaluation, the maintenance cost analysis and the decision making technique for optimal maintenance strategy and budget plan. The contents of research for establishing the advanced BMS combining asset management concept are summarized as follows; first, it may need life-cycle performance evaluation and estimation of the effect of maintenance interventions, and asset evaluation and its calculation technique related to maintenance cost. Second, decision making technology based on these various performances and cost related information is necessary for the life-cycle management of bridge. Finally, a proper computational system and practical education for applying a new method and system should be developed.

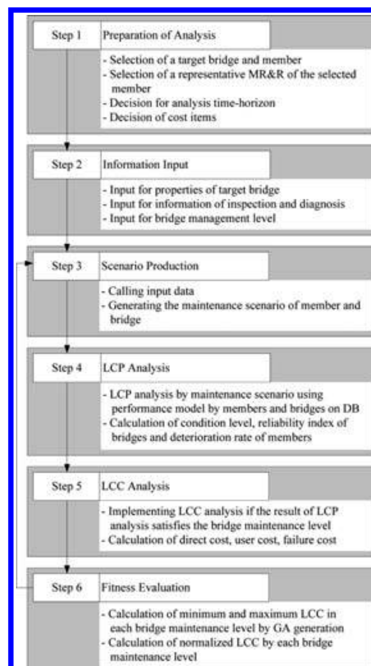

Figure 1. Analysis procedure to establish the optimal maintenance strategies.

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SS11: ARCHES: Assessment & rehabilitation of Central European highway structures

Organizers: T. Wierzbicki & J.R. Casas

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Smart cathodic protection systems

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ABSTRACT

Cathodic protection (CP) is intended to deliver corrosion protection in concrete structures exposed to aggressive environments, e.g. in deicing salt and marine climates. Although preventative application is possible (called Cathodic Prevention), most cases concern structures that already have developed corrosion and some level of damage to the concrete cover (cracking, spalling). CP involves polarizing the reinforcing steel by a low amount of direct current, originating from a conductor on the surface or in the cross section of the structure connected to a low voltage power source. Spalled areas need to be repaired using normal concrete repair methods. A large difference between CP and the conventional repair approach is that in the latter case, all chloride contaminated concrete has to be removed and new has to be reapplied. With CP, physically sound but chloride contaminated concrete can be left in place. European experience with many hundreds of CP systems shows that they perform well, provided that a minimum of (routine) maintenance is carried out. An inventory in the Netherlands of one hundred well-documented CP systems shows that working lives are at least more than 13 years and probably more than 25 years. Individual cases that needed maintenance are discussed, based on the need to replace electrical connections, reference electrodes, power units and anode materials. All such cases have occurred to a limited extent. It is felt that the quality and life of items needing replacement in the past have now been improved or have become considerably cheaper, allowing some redundancy to be installed. Maintenance is generally carried out by the contractor who installed the system, for a relatively low annual fee. This involves routine checks of power units and testing depolarisation of reinforcement at least twice a year. Regular design of CP is based on conservative assumptions. Lightweight anode materials and numerical modelling in the design phase may provide more cost effective systems, which can be termed "smart CP systems". A CP trial on a bridge substructure in Slovenia was carried out in European FP6 research project ARCHES. In the trial, various types of lightweight anodes were applied including conductive coating and titanium strip anodes in mortar dykes on the concrete surface. Numerical Finite Element modelling was applied to the trial CP systems. In the modeling a geometrical analogue is built in two dimensions, as three-dimensional finite element modeling is beyond present computer capabilities. Input parameters are either obtained from the structure (steel potentials, concrete resistivity) or from literature (electrochemical parameters). In our calculations the input parameters had to be fine-tuned to a certain extent in order to obtain realistic current levels. With the adjusted input, local polarisations were calculated in good to reasonable agreement with values measured in the trial. Consequently, it is possible to predict the performance of a CP system in the design phase, which allows for economical optimisation. For this case, life cycle costs were calculated for conventional repair (assuming 25 years life of the repairs and also taking into account that re-repairs would be needed half way the 25 year life), CP with a conductive coating (including re-application of the coating once in a 25 year period) and titanium mesh with a shotcrete overlay (that will last at least 25 years). CP proved to be more economical than conventional repair, taking into account that the life of conventional repairs is limited, as suggested by a European study, and consequently there is a significant probability that re-repairs are needed. For the trial site, it was found that up to 10% lower life cycle costs over a period of 25 years were possible with CP compared to conventional repair.

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Assessment and monitoring of existing bridges to avoid unnecessary strengthening or replacement

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ABSTRACT

The increasing volume of European transport urgently requires an effective road and rail system in New Member States (NMS) of the European Union (EU) and Central and Eastern European countries (CEEC). To bring this transport infrastructure up to modern European standards will require an immense investment – and therefore difficult to achieve in the medium term. New motorways will be required with many new bridges. Numerous existing bridges will need to be assessed, and a large portion of them improved or replaced.

The European project ARCHES (Assessment and Rehabilitation of Central European Highway Structures) had as main objective to develop ways to raise the standard of the highway structures of NMS and CEEC to the level necessary for their full economic integration into the EU and for the future development of the Union. Therefore, it becomes important to develop more appropriate tools and procedures to avoid unnecessary interventions in bridges. Rehabilitation and replacement should be as far as possible avoided by developing better safety assessment methods that may result on an extended life-time of the structure. This has been analyzed within Workpackage 2 (WP2) of ARCHES. An improved assessment needs both: first, an accurate assessment of the actual bridge capacity and, second, a good assessment of the actual loading. The last one is mainly referred to the actual traffic live load. Unfortunately, traffic loading conditions in NMS and CEEC highway structures are mostly unknown. The reason for such a situation is a lack of consistent systems that collect traffic data in a way appropriate for bridge design and assessment. The load carrying capacity of many highway structures is not known either, especially for very old bridges where the design and construction documents are not available. With that many unknowns, both on the loading and on the resistance sides, it becomes really difficult to obtain a reliable estimate of the actual bridge safety and serviceability and to propose optimal rehabilitation measures.

An accurate estimation of traffic load in the bridge can be carried out by obtaining a good estimate of the static load of vehicles and of the dynamic amplification factor (DAF). Thus, the paper is divided into 3 main parts also related to 3 of the main tasks of the project: 1) *Bridge traffic load monitoring* aimed to quantify the increased traffic loading on bridges in NMS after EU enlargement. It combines available information and newly collected traffic load data (weigh-in-motion (WIM) measurements) to propose simplified assessment traffic load models at two levels: by taking into account the measured WIM data on a bridge of concern (a site-specific model) or by applying more generalized, but measured, WIM data at the network level (road-specific). The main results are available in ARCHES-D08 (2009). 2) The task on *Load carrying capacity based on load testing results* optimized bridge capacity assessment by using load tests or computational analysis to find reserves in load carrying capacity. Results of this task are reported in ARCHES-D16 (2009). 3) The task on *Reducing dynamic loading of bridges* dealt with the assessment of dynamic impact on bridges. It is shown in ARCHES–D10 (2009) how heavy trucks and multi-presence of vehicles can lead to an important reduction of this dynamic interaction.

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Composite UHPFRC-concrete construction for rehabilitation – most recent advances and applications

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ABSTRACT

The increased volume of European transport urgently requires an effective road and rail system in Central European and Eastern countries (CEEC) with a major investment in building new and assessing and rehabilitating old structures.

Ultra-High Performance Fibre Reinforced Concretes (UHPFRC) are characterized by a unique combination of extremely low permeability, high strength and deformation capacity (tensile strain hardening), Extensive R&D works performed during EU projects SAMARIS and ARCHES have demonstrated that the UHPFRC technology is fast, efficient and price competitive for the rehabilitation or reinforcement of reinforced concrete structures.

An innovative concept of Ultra High Performance matrix with a high dosage of mineral addition has been developed that makes the application of UHPFRC technology feasible with a wide range of locally available cements and superplasticisers. This concept was validated with both Slovenian and Polish raw materials. In the next step the rheology of those mixes was adapted to enable them to accommodate 5% slopes of the substrates at fresh state. This new material was successfully applied in July 2009, for the rehabilitation of the deck and footpaths of the Log Čezsoški bridge (Slovenia) with challenging slopes of 5%, opening the way to a wider dissemination of this concept, in most demanding conditions of application, see ARCHES deliverables D06 and D14, Denarié et al. (2009a and b).

The intervention was fast (1 month instead of 3 month with traditional technique) and by means of a newly developed surfacing technique it was possible to achieve uniform textured UHPFRC footpath surfaces on which barefoot walking is possible. This application

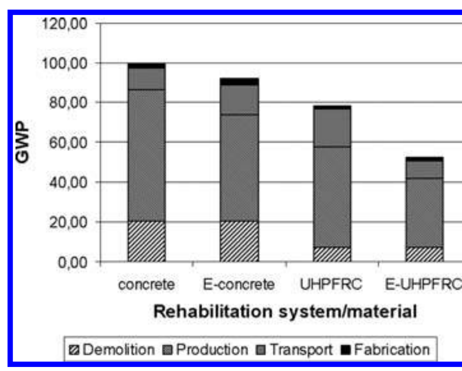

Figure 1. Global Warming potential for the Log čezsoški rehabilitation, considering the life cycle. Traditional rehabilitation system with standard concrete = 100%, Habert et al. (2009).

demonstrated at an industrial scale the ability of the newly designed UHPFRC mixes to reply to the difficult challenges of the site, without any increase of rehabilitation costs, but to the great satisfaction of the owner, user and contractor.

Further, the newly designed E-UHPFRC recipes have a dramatically reduced cement content which makes them more economical and particularly attractive from an environmental point of view.

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Corrosion resistant steels as reinforcement in concrete

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ABSTRACT

Maintenance and repair costs of concrete structures constitute a major part of the current spending in infrastructure worldwide and often overhead the costs of structures themselves. The enormous costs and safety issues associated with corrosion of steel in concrete have resulted in the development of a wide range of new technologies and materials to increase the durability of reinforced concrete structures and their repairs. One of these methods is the application of stainless steels, which are resistant to corrosion in concrete environment even when the concrete is highly contaminated with chlorides, as substitution of a part or all reinforcement in concrete. (Nürnberger, 1996).

The aim of research within working task 3.1 "Validation and application of low-alloyed steel" of the specific targeted research project ARCHES was to validate the application of alloyed steels in the concrete and finally to prepare *Recommendations for the use of corrosion resistant steels*. In this paper the overview of experimental results are presented. Experimental program was designed with the aim of evaluating corrosion behavior of alloy steel in non-corrosive and corrosive environment.

Corrosion behavior of seven different steel types was studied: two types of steel with lower content of alloying elements: TOP12 (1.4003) and 204Cu (1.4597), two grades of duplex steel: UGIGRIP 4362 (1.4362) and SAE/UNS S3 2205 (1.4462), two grades of stainless steel: AISI 304 (1.4301) and AISI 304L (1.4306) and B500B for comparison.

The research program was divided into:

- laboratory testing of steel specimens in pore solutions with different pH values and chloride contents;
- laboratory testing of steel, embedded into small cylindrical and prismatic concrete specimens and medium size concrete specimens;
- on site exposure and testing of steel embedded into larger concrete slabs and columns.

Results that were achieved and evaluated during the ARCHES program included a laboratory study of rebars in pore solution, rebar in concrete examination and two field tests of columns and a large concrete reinforced structure.

The use of corrosion resistant steel has not been widely adopted because of perceived concerns relating to initial cost penalties. Overall conclusion from the project is that the selective replacement of carbon steel with corrosion resistant steel and with tailored selection of stainless steel grade the increase of initial investment can be less than 10%, which is negligible comparing to the reduction in maintenance and life cycle costs of the structure.

ACKNOWLEDGMENTS

This research was performed within projects "The Development of New Materials and Concrete Structure Protection Systems" (082-0822161-2159), funded by Croatian Ministry of education, science and sport and ARCHES (031272), funded by the European Community under the Sixth Framework Programme.

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SS13: Current advancements in bridge technology
Organizer: A.H. Malik

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Planning and design of 1-pylon suspension bridge, Dandeung Bridge

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ABSTRACT

The Dandeung suspension bridge, the 1-pylon suspension bridge connecting Sinsido and Munyeodo in Korea, is due to be constructed in the second section of connecting road works of the Gogunsan islands in Jeollanamdo. This project is started in November, 2009 and the construction period is just 48 months. The construction of this bridge has been planned to make traffic for inhabitants of the islands convenient, to promote local development, and to improve the international marine tourism circumstance. This bridge is designed to have one pylon and asymmetric main cable configuration. The main span length of Dandeung bridge is 400 m which will be the world's longest one among suspension bridges with 1-pylon.

In the basic planning of this project, Dandeung bridge was designed as a self-anchored suspension bridge with 2 pylons, 220 m long main span and two side spans. However, the planning had to be changed to a different bridge type to be able to avoid marine works because the site for this project has the deep depth and rapid current of sea. And the basic planning was traditional, and the originality for tourism was lacked. Consequently, in the basic design, the 1-pylon suspension bridge with the only one main span of 400 m long that is the world's longest one was suggested and planned to avoid the poor marine condition, to stand for the regional character, and to build a landmark structure for tourism.

A pylon of Dandeung bridge is designed as a A-shaped rahmen concrete structure with the height of 105 m and a D-shaped additional column symbolized a sail of boat. Such a A-shaped and D-shaped pylon can secure the structural safety and stability of non-symmetric suspension bridge with 1 pylon under loading and maximize the harmony with the regional marine landscape. Main cables are planned to be installed on two planes because two planes arrangement can improve the aerodynamic stability of cables and bridge and help travelers to turn back toward opposite side easily when accidents take place on the bridge. Also, as main cables are designed to be installed with inclined arrangement, the installing space of each main cable is 20 m at anchorage 1, 3 m at the top of pylon, and 11m at anchorage 2. The key point of a stiffening girder design is to minimize the weight of girders and secure the aerodynamic stability at the same time. As a result, in this bridge, two edge box section is planned as the final design.

Figure 1. Aerial view of Dandeung bridge.

Table 1. Structural dimensions and design condition.

Component	Design
Span composition	Cable : 425 + 280 = 705 m Girder : 400 m
Girder Type	Edge box girder
Slope	
Longitudinal	2%
Transverse	2%
Design Traffic Speed	60 km/h
Width	20.0 m
Live Load	DB-24, DL-24
Design Wind Speed	34.4 m/s
Seismic Load	Seismic Grade 1

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Recent development of wireless bridge monitoring system

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ABSTRACT

The purpose of this study is to find state-of-the-art technology by the industry and research institutes which use intelligent wireless monitoring systems instead of conventional on-site bridge testing. The system could include (1) a practical plug-and-play battery-operated "wireless" data acquisition unit, and (2) state-of-the-art "wireless" data transmission and communication technologies. The second objective of this project is to investigate further and integrate the "off-the-shelf" wireless remote monitoring system with the latest technology for the project's use in the State of Maryland.

Based on the survey documented in this report, it is found that wireless communication, together with its applications and underlying technologies, as shown in Figure 1, is among today's most active areas of technology development. Figure 1 shows the requirement of hardware and supplies in (1) sensors, (2) data acquisition system (DAQ), (3) power supply, (4) data transmission, and (5) data processing and data storage.

Four case studies were presented. Case study 1 was wireless structural monitoring by Ivoncon on a newly replaced fiber reinforced plastics (FRP) bridge deck. Case study 2 was using Microstrain on the pilot study of Bridge MD140 over MD27. Case Study 3 was using ATI on the final study of Bridge MD140 over MD27. Case study 4 was conducted for a case with a portable, proto-type integrated remote (web based) bridge monitoring system which were field-tested on a typical bridge sites. The integrated system was used in the Tyding's Bridge field testing. Strain gauges were installed at four locations on the first (right) lane of the Tydings North Bound Bridge while the traffic was diverted away. Measurement was made and transmitted through the Verizon cellular networks to be viewed on a remote site. 12-minute raw data was extracted from gauge number 3 and is shown in Figure 2.

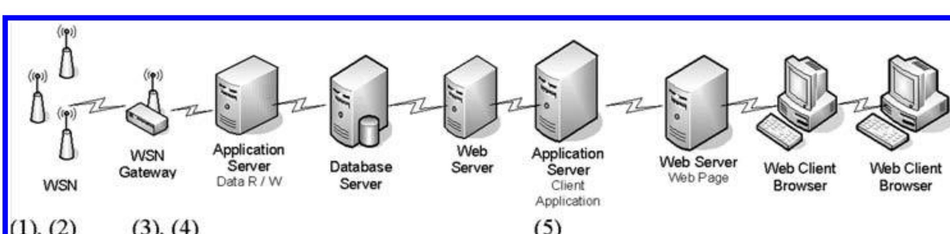

Figure 1. Remote Wireless Bridge Monitoring System.

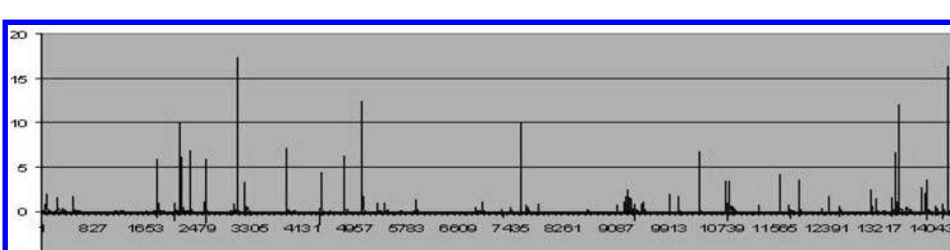

Figure 2. 12-minute raw data was extracted from Tyding Bridge gauge number 3.

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Technical advancements in bridge technology

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ABSTRACT

There have been very encouraging technical advancements in bridge technology, which are reflected in the construction of remarkable bridge structures throughout the world. This paper will briefly cover some of the aspects developed at this time. The use of high performance materials and more sophisticated design and construction specifications to meet the enhanced live load demands. The major contributor towards this advancement in bridge technology, as in many other fields, is the computer-aided software programs and the related innovations.

- ACCELERATED BRIDGE CONSTRUCTION
  The Accelerated Bridge Construction (ABC) piloted by FHWA aims at minimizing traffic delays during construction, minimizing environmental impacts and to achieve 100-year service life for new bridges.
- HIGH PERFORMANCE MATERIALS
  The introduction of high performance steel (HPS) and high performance concrete (HPC) in the bridge technology has established the confidence of the designers, the bridge owners, and the steel industry. The high strength of HPS and HPC cuts down relatively the number of girders, as well as the weight of the superstructure and overall cost of the project
- RECORD BREAKING LONGER SPAN BRIDGES
  The advancements in bridge technology are taking place in all parts of the world which is demonstrated by record breaking spans, heavy load carrying capacity and longer lifespan. The Akashi Kaikyo Bridge, Hangzhou Bay Bridge, and Sutong Bridge are the examples of record breaking cable supported bridges, which utilized the latest advancements in design, construction, and fabrication using the high performance materials.
- ENHANCED CONTRACTING and PARTNERING
  Innovative contracting methods which include 'Design Build', 'Lane Rental', 'A+B' and Design Build have saved time and project cost for the DOTs.
- DESIGN and ANALYSIS SOFTWARE
  Bridge design and analysis software has been a great asset towards the innovative approaches in the design and global analysis of complex and advanced bridge structures. Model generating techniques provide specific menu to cover all loadings, structural configurations, complex geometry and exact support conditions.
  BRIDGE DESIGN and ANALYSIS SOFTWARE
- Bridge design and analysis software has been a great asset towards the innovative approaches in the design and global analysis of complex and advanced bridge structures. Model generating techniques provide specific menu to cover all loadings, structural configurations, complex geometry and exact support conditions.

The design and analysis methodologies, construction, fabrication, and erection equipments, construction materials and inspection techniques have been developed to meet the ever increasing challenges of heavy loads, environmental changes and extreme events.

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Can nanotechnology address today's civil engineering challenges?

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ABSTRACT

A Nanotechnology has become very much the buzzword of the 21st century. While tangible benefits are being realised in industries ranging from medical to consumer products, the conservative construction sector has been slow to embrace the new technology. While products are available that claim nanotechnology-based improvements in products ranging from concrete admixtures to repair mortars, are these developments true breakthroughs in themselves – or is it that we can now see structures and interactions under the "nanoscope" for the first time?

Identifying step-change developments in concrete materials and systems due to nanotechnology is a challenge for practising engineers. Developments in civil engineering are likely to be small, iterative improvements, such as surface treatments that make concrete self cleaning. As more breakthroughs are made, so the sector may benefit from intelligent, cost effective, safer, durable and environmentally friendly materials and structures.

Nanotechnology provides access to the world of the smallest things – a world that is unfamiliar to the civil engineer, used to dealing in massive constructions; the diameter of a human hair being around 50,000 nanometres (50 micrometres).

Nanoparticles occur naturally or are man-made by-products (such as vehicle exhaust particulates), but increasingly nanoparticles are being deliberately engineered. However, the term "nano" is appearing in all manner of products and systems. The addition of a small percentage of "nanoparticles" is being claimed to impart wonder properties to a diverse range of products from face creams and car polish to concrete and repair mortars. However to the suspicious engineer, it has become obvious that in some quarters "nanotechnology" is being loosely ascribed to particles several hundred nanometres in diameter – close to the more familiar micrometre size. The impact of Nanotechnology and its potential benefit to the industry in addressing civil engineering challenges therefore still remains a subject of further research and debate.

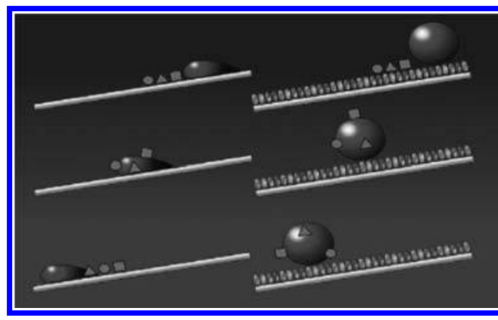

Figure 1. Lotus Leaf Effect – Developments based on Lotus Leaf effect are being applied in road signs, buildings etc.

This paper explores some of the recent developments in nanotechnology relevant to construction and studies the potential benefits on the life and performance and management of civil engineering structures such as bridges. It highlights areas of actual practical application of nanotechnology in construction and looks at products that may have great potential and benefit to address current and futures civil engineering challenges.

The challenges facing the industry at the start of the 21st century are also highlighted and areas of application for nanotechnology discussed.

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Impact of longitudinal tendons on long-term deflections of long-span concrete cantilever bridges

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ABSTRACT

Due to the advantages of higher stiffness, fewer expansion joints, saving on large-tonnage bearings, better comfort for driving, stronger anti-wind and anti-seismic performance, and more convenient maintenance during the period of service, the prestressed concrete continuous rigid-frame box girder bridge is one of the preferred design plans within the 100 m to 350 m span range in China. However, for over ten years, excessive deflections at the mid-span have commonly appeared in long-span prestressed concrete box girder bridges during their service years, especially for the segmental, cast-in-place concrete cantilever bridges. Lessons from some existing bridges, such as the Koror-Babeldaob Bridge, the Huangshi Yangtze River Bridge and the auxiliary shipping channel of Humen Bridge, the design of longitudinal prestressing tendons to avoid the excessive deflections seems to be extremely important.

This paper is initiated with the objective to improve the excessive deflections problems by designing longitudinal tendons. In the beginning, the development of design of tendons for long-span concrete cantilever bridges in China is discussed. Nowadays, the bent-down tendons bring back the designers' attention, mainly due to the web's bent-down tendons which can provide a pre-shear force and are very effective to limit the principal tensile stress. Therefore, the elimination of web's bent-down tendons is found one of the main causes for the inclined cracks which are harmful to the deflections. The comparison between the web's bent-down tendons and the straight layout of tendons is shown in Figure 1.

Moreover, two approaches used in the continuous rigid frame of Sutong Bridge to mitigate the deflections are introduced, which are the pre-setting internal tendons in the bottom slabs and the preparatory external tendons tensioned during the bridge's service. Furthermore, the impact of them on the deflections was analyzed.

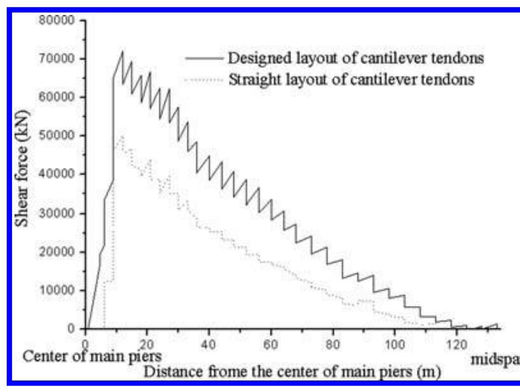

Figure 1. Shear force provided by cantilever tendons.

Table 1. Upward deformation at mid-span under the three types of layout of external tendons.

Type of tendons	Upper	Bottom	Broken-line
Deformation (mm)	11	6	11

In addition, taking account of the advantages of maintaining easily, replaceability, re-tensioning and so on, the method of external tendons accompanied with the internal tendons as the continuity tendons will have a bright future. In the design of the external tendons, we should find the optimal layout to effectively mitigate the deflections. The efficiency of three types of external tendons (upper tendons, bottom tendons and broken-line tendons) for reducing the deflections was analyzed and compared, shown in Table 1.

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Assessment of effective flexure rigidity and composite capacity of corrugated steel-concrete composite deck with I-beam welded

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ABSTRACT

The steel-concrete composite structure is applied extensively to the construction of bridges and buildings recently. This study suggested a new type of steel-concrete composite deck plate to replace the cast-in-place reinforced concrete deck of the past. The perforated plate type shear connector was useful for providing horizontal shear resistance between corrugated plate and concrete. The corrugated steel-concrete composite deck reinforced with I-beam among corrugated steel-concrete composite decks enhanced the workability and reduced the weight compared with existing RC decks by burying I-beam inside the corrugated steel-concrete deck. Currently the calculation of the effective flexural rigidity of general RC structures applies the methods recommended by the Standard Specifications for Highway Bridges and ACI.

For the effective flexural rigidity of structures, Branson suggested formula (1) by approaching it through elastic analysis of stress and strain in 1965 (Branson D.E., 1965), and Porter suggested the average of moment of inertia and the moment of inertia on non-cracking section as the effective moment of inertia.(Poter M.L. et al., 1976) Also, Lamport suggested his effective moment of inertia that provides better fitting values by adjusting constants and exponents appropriately using statistical processing of value suggested by Branson.(Lamport W.B., 1990)

This study applied the method of calculating effective flexural rigidity recommended by the Standard Specifications for Highway Bridges and ACI to the steel-concrete composite deck reinforced with I-beam to evaluate its applicability and made comparisons with reinforced concrete deck. Total 15 specimens were produced by considering 4 variables; the existence of stud, changes between spans, the shape of the section, and material connecting methods, and tested to compare and analyze the actual flexural rigidity and the effective flexural rigidity calculated with the formula suggested by ACI.

Role of groundwater in upland streamflow generation is far more complicated than previously considered and has important implications for upland water quality.

15 specimens were produced by reflecting 4 variables (use of shear connector, section shape, span length, and change of connecting method) and flexural test was performed to examine the effect of adhesion and material connecting method on effective flexural rigidity. Moment of inertia was obtained based on the load-deflection relationship obtained through experiments, and it was compared with formula suggested by Standard Specifications for Highway Bridges and ACI (ACI 318R-02, 2002) to examine changes.

And In Eurocode 4, the bending resistance of a composite beam with a partial shear connection can also be conservatively specifiyed by an interpolation method. In order to investigation the degree of shear connection of composite deck used Eurocode 4

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SS14: Using technology to manage, preserve, & renew landmark signature bridges

Organizers: D.S. Lowdermilk & F.L. Moon

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Load capacity estimation for the Burlington Bristol Bridge

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ABSTRACT

The Burlington-Bristol Bridge spans 3000 ft across the Delaware River and has a main lift span composed of a 540 ft through-truss. In 2007, this bridge was load rated using both analytical methods and traditional truck load tests, and the results were found to be inconclusive. Following the load tests, there were several models that matched the live load strains equally well and predicted significantly different rating factors. The key uncertainty was related to the distribution of dead load actions, which represented over 95% of the demand on the critical main span members and was completely unobservable during tradition truck load tests. In an attempt to estimate dead load demand, a three phased approach was developed and carried out. First, the critical top chord member was instrumented and monitored with a series of both high speed and vibrating wire strain gages to capture the variation of dead load strain due to temperature, radiation, and other seasonal effects. The second phase included detailed ambient vibration monitoring study and a subsequent physics-based structural identification to estimate total mass, mass distribution and boundary/continuity conditions. The third and final phase included the use of a portable x-ray diffraction technology to estimate the intrinsic stress at various locations around the critical upper chord member. This paper and will present the details of this study and discuss the proposal of how uncertainty can be reduced using this technology while the presentation will discuss the completion of these techniques on the Burlington Bristol Bridge and the resulting effect on the load capacity estimation.

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Design and implementation of load cell bearings to measure dead and live load effects in an aged long span bridge

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ABSTRACT

In the spring of 2010, the expansion bearings of the Burlington-Bristol Bridge were replaced due to a poor condition rating (that caused the bridge to be classified as *structurally deficient*). The replacement of these bearings represented a unique opportunity to configure the new bearings to monitor the dead load and live load actions as well as their variation with environmental conditions. Towards that end, a series of trial designs were developed with various bearing types, load cell configurations, etc. These candidate designs were then evaluated through a series of finite element analyses and a single design was selected. To validate the selected design, a prototype was developed and tested in the laboratory under various axial forces, unidirectional and bidirectional moments, and shear forces. Once validated, 14 "smart bearings" were fabricated and installed on the Burlington-Bristol Bridge. To verify their proper operation, a series of load tests were carried out following installation. This paper and presentation will detail the development and validation activities for the "smart bearings" as well as the results from the first few months of long-term monitoring. In addition, non-technical challenges associated with developing appropriate design and performance specifications for the bearings will be discussed.

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Structural health monitoring of signature bridges: An engineer's perspective

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ABSTRACT

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The aim of this paper is to detail the circumstances under which an infrastructure owner – facing difficult decisions related to bridge safety and repair for a movable bridge on a major highway link – made the judgment that a best practices application of structural identification (St-Id) was warranted. Although the integrated use of experimental and simulation technology cost the owner more than the traditional engineering inspection and load rating approaches (which were completed in parallel), from the owner's perspective the potential benefits associated with such an approach clearly outweighed the guaranteed costs. The decision hinged on the reduced uncertainty that St-Id offered, which the owner viewed as critical to making rational decisions related to posting, rehabilitation, and ultimately the replacement of the bridge in question.

The Oceanic Bridge spanning the Navesink River in Monmouth County New Jersey was constructed in 1939 and rehabilitated in 1970. The structure consists of 57 spans with a 33 meter double leaf bascule span over the navigable channel (Prader et al. 2009). The current bridge is under consideration for replacement due to the significant deterioration of the bridge superstructure and piers. Currently, the bridge (due to concerns with the bascule span) is posted for 3 tons, which severely disrupts the travel of school buses, emergency vehicles, and commerce in the surrounding area. Due to the length of the project delivery process for the proposed bridge, the existing bridge would ideally need to serve another 5 to 7 years at a higher posting with only absolutely necessary rehabilitation of the structure. The County is also planning to perform retrofit construction on corroded bridge members and a full grid deck system replacement for which the construction cost is estimated at over $3M. Another possibility the County is considering includes full replacement of the bascule span which carries a construction cost of over $8M. Both of these options are costly considering the life span of this bridge at 5 to 7 years.

The work on this project has shown that utilizing Structural Health Identification can, by reducing uncertainty give a more clear picture of current behavior of the bridge in question. Through the instrumentation and modeling intrinsic in the identification, we can get a better understanding of load paths and critical members, and can, thereby develop a rehabilitation and preservation plan that more closely matches the needs of the bridge. In the case of the Oceanic Bridge, we were analyzing a structure that must stay in service for 5 to 7 more years. By performing targeted retrofits, we can be more certain that we are placing this bridge, and the County in a position of strength to nurse this geriatric structure through the remainder of its life cycle.

Figure 1. Oceanic Bridge Bascule Span.

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Health monitoring of the Tacony-Palmyra bridge bascule span

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ABSTRACT

The Tacony-Palmyra Bridge (Figure 1) is a steel structure which spans the Delaware River between New Jersey and Pennsylvania. The bridge was constructed in 1929 by Modjeski, Masters and Chase as a replacement for the inadequate Tacony-Palmyra Ferry which operated at the same location. The design of the bridge includes viaduct approach spans, six continuous truss spans, a 550' arch span and a 260' double leaf rolling bascule span. On average the bridge, which is owned by the Burlington County Bridge Commission, sees approximately 50,000 vehicles a day. The bridge is tolled leaving New Jersey, with two lanes of traffic in that direction and one lane leaving Pennsylvania.

The Tacony-Palmyra Bridge is one of several major assets under the care of the Burlington County Bridge Commission. They are responsible for the Burlington Bristol Bridge, a vertical lift bridge, which also crosses the Delaware River and the Riverside-Delanco Bridge, a swing bridge on Rancocas Creek. In addition to one of each of the three most popular types of movable bridges, the Commission also owns several smaller fixed bridges throughout the county. All of the Commission's assets are maintained with toll revenue on the Burlington Bristol and Tacony-Palmyra Bridges. Despite the potential for delays to bridge openings, these structures still experience a substantial traffic load, partially because of lower toll rates, and partially because of the ties to the community which the Commission works to maintain.

This paper aims to discuss the methodology employed to develop the full instrumentation plan for the TPB Bascule span monitoring. This includes discussing the relationships between the owner, engineer and researcher, the goals of the monitoring project, the functionality of the movable span of the bridge and the final instrumentation design.

The proposed system will provide the following:

1. Continuous check on the balance of the bascule span
2. Immediate feedback to the bridge operator on the quality of every bascule span opening
3. A better understanding of the load carrying mechanisms of the span; in particular the distribution of load between the dead and live load trusses

Figure 1. Tacony-Palmyra Bridge.

4. Characterization of the types of live load that the bridge is subject to by way of correlation of measured responses with images
5. Tracking of transient forces like the axial force at the leaf tips resulting from the bridge operator preload the system during closings
6. Characterization of long-term effects due to environmental changes
7. Immediate feedback in relation to safety issues like icing of the roadway surface

These deliverables will aid the Commission in both the day to day operation of the structure, as well as in the development of a long-term plan for indefinite preservation of this crucial, historic structure.

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SS15: Modeling of bridge seismic response
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Application and evaluation of a performance-based methodology for the seismic assessment of multi-span simply supported deck bridges

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ABSTRACT

Nonlinear Static Methods (NSM) based on pushover analysis represent a good alternative to Nonlinear Time-History Analysis (NTHA) and they are frequently used as a practical engineering tool for the seismic assessment of existing structures. This paper focuses on the verification of a new performance-based adaptive methodology for the seismic assessment of bridges.

The proposed method, called IACSM, is based on an inverse (I), adaptive (A) application of the Capacity Spectrum Method (CSM). The capacity curve of the bridge is derived from a Displacement-based Adaptive Pushover (DAP) analysis of the structure, carried out separately in the transverse and longitudinal direction of the bridge. The bridge is modeled according to the principles of the so-called Structural Components Modeling (SCM) approach, in which, the structure is divided in a number of independent rigid diaphragms, modeling the bridge decks, mutually connected by means of a series of nonlinear springs, modeling bearing devices, piers, joint and abutments.

A number of Performance Levels (PLs), for which vulnerability and seismic risk shall be evaluated, are defined. Each PL is associated to a number of Damage States (DSs) of the critical members of the bridge (piers, abutments, bearing devices, joints), identified by a number of points on the DAP curve of the bridge. The IACSM provides the earthquake intensity level (PGA) corresponding to the attainment of the selected DSs, using high-damping elastic response spectra as Demand Curves.

The seismic vulnerability of the bridge is described by means of fragility curves associated to the PGA values previously derived. Finally, the seismic risk is evaluated as convolution integral of the product between the seismic vulnerability of the bridge (expressed by the fragility curve) and the seismic hazard of the bridge site (expressed by a proper hazard curve).

In this paper, the proposed procedure has been applied to a set of nine multi-span simply supported deck bridges of the Italian A16 Napoli-Canosa highway, characterized by different types of piers (i.e. single shaft, simple frame and single wall) and different types of bearing devices (i.e. neoprene pads, steel hinges, sliding/roller bearings, pendulum systems).

The IACSM predictions have been compared to the results of NTHA, carried out using a set of 7 accelerograms, compatible with the response spectrum provided by Eurocode 8 for soil type B, scaled to the PGA values provided by IACSM for each DS.

The comparison has been made in terms of maximum deformed shape of the deck and maximum pier displacements. Three indexes have been also computed to measure the precision of IACSM in capturing the 'exact' maximum deformed shape of the deck, maximum pier displacement profiles and seismic demand to the critical element (pier, bearing device, abutment or joint) of the bridge. The target value of the three indexes is always 1.

Results show that IASCM is able to predict with good accuracy the 'actual' response of the bridge derived from NTHA, even when the bridge response is strongly nonlinear. Moreover the critical members predicted by IACSM coincide with those resulting from NTHA. In approximately 75% of the totality of DSs considered, the three indexes result between 0.9 and 1.1, both in the longitudinal and transverse direction. Results clearly indicate the good accuracy of IACSM in predicting the PGA values associated to slight-to-severe damage states, for bridges representative of the Italian bridge inventory.

The results of this study prove that the proposed methodology can be confidently used by the network managers for screening and prioritization of a wide bridge inventory, to take responsibly decisions concerning possible repair or seismic retrofit measures.

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Probabilistic seismic response and retrofit assessment of aging bridges

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ABSTRACT

Extreme events such as earthquakes continue to threaten the security of bridge infrastructure. However, current approaches to seismic retrofit address the enhanced seismic protection of bridges without explicit consideration of other threats such as aging and deterioration of the structure. Presently, more than 26% of the nation's bridges are estimated to be structurally deficient and it is evident that aging bridges are more vulnerable to natural hazards, such as earthquakes. Hence, it is important to identify retrofit strategies that simultaneously target both aging, such as corrosion protection, as well as reduction of seismic vulnerability of aging reinforced concrete members.

This paper presents a probabilistic performance evaluation of aging bridges under seismic loading using bridge fragility curves, which are statements of the conditional probability of bridge failure. Along the service life of the bridge, mechanisms such as the loss of cross sectional area of the reinforcement bars and degradation of steel bridge bearings due to corrosion leads to significantly deteriorated seismic performance. In the present study, fragility curves are developed for typical degraded highway bridges by incorporating probabilistic time dependent corrosion models for key bridge components, specifically reinforced concrete columns. Other effects of corrosion deterioration like concrete spalling are not considered in the present model due to their lack of influence on bridge fragility. Performance of carbon fiber-reinforced polymer (CFRP) column wraps as a potential retrofit measure for both mitigating corrosion deterioration and improving seismic performance is presented and analytically assessed for its influence on bridge fragility. The effectiveness of conducting this retrofit at different points in time along the service life of the bridge is evaluated through comparison of the fragility curves for the deteriorated and retrofitted bridge.

Table 1. Improvements in median values of system fragility after the bridge is retrofitted with CFRP jackets at $t = 25$ years.

Years	25	50	75
% Increase in median values	15.4	21.2	28.0

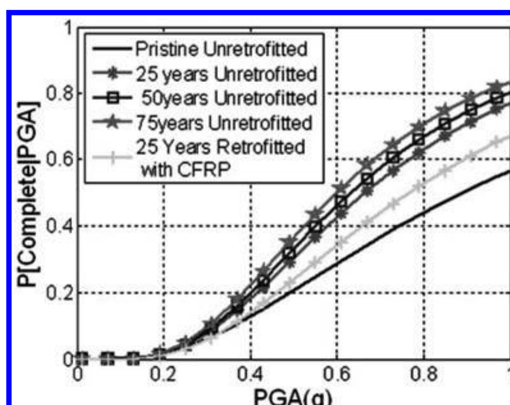

Figure 1. System level fragility curves at different points in time for the retrofitted and unretrofitted bridge.

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Seismic response of repaired bridges by pseudodynamic tests

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ABSTRACT

Eight bridge circular pier specimens (1:6 model of the most stressed piers) severely damaged after previous pseudodynamic and cyclic tests until failure, were repaired and/or upgraded by means of epoxy adhesive, stainless steel rebars, self compacting concrete and C-FRP wrapping and tested by pseudodynamic, under the seismic action used for first tests on undamaged specimen. The undamaged pier specimens were representative of tall and squat circular r.c. piers designed according to Eurocode 8 or Italian Code before 1986.

The damaged piers have been retrofitted by removing the damaged cover at the base substituting longitudinal rebars with inox bars making the new cover with SCC concrete and wrapping the pier with C-FRP to increase shear strength and ductility.

In the pseudodynamic test, the repaired and/or retrofitted piers specimens (one column per bridge) are physically tested in the lab (Department of Structures, DIS, University of Roma Tre) while the rest of the bridge is numerically simulated. After pseudodynamic tests on the bridge the single piers are brought to failure by cyclic tests. Damage states after the previous tests are assessed and description of the repairing and retrofitting of the piers as well as the description of the In-House pseudodynamic apparatus and dedicated software implemented using LabVIEW and MATLAB are given and finally a comparison between the seismic behavior of the original and the retrofitted bridge piers are shown in terms of seismic response. Tests results are compared to nonlinear analyses using the computer program OpenSees. Results are discussed.

Future development will be the inclusion in pseudodynamic test of the soil-structure interaction and deck torsional stiffness, while inox rebars of new type will be considered.

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Response of curved steel bridges to seismic loading

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INRODUCTION

Given the lack of research related to curved, steel, bridge seismic response and the continued increase in curved bridge design and construction, there continues to be a need to investigate seismic behavior. Two projects that studied curved, steel, plate girder bridge seismic response are summarized. The first examined the influence of certain parameters on forces and reactions generated in representative curved bridges. The second studied seismic response of a group of curved bridges to generate fragility curves.

Findings for the first study indicated that, for simply-supported curved plate girder bridges, varying girder radius of curvature had a slightly more pronounced effect on reactions reported at the bridge bearings for the interior (lowest radius) girder (Figure 1), especially under vertical excitation, than varying cross frame spacing. Variations in cross frame spacing had marginal effect on maximum loads sustained in a select cross frame member during a seismic event. Vertical seismic excitation was shown to have more influence than horizontal excitations for the cases that are presented.

The second study detailed methods used to generate seismic fragility curves for a select group of horizontally curved steel bridges using RSMs. Procedures used to: (1) select and screen important input and output parameters, generate finite element models and synthetic ground motions used to create the RSMs and; (2) apply the RSMs to efficiently generate select fragility curves for a group of 99 curved steel bridges were presented. One representative fragility curve, related to the likelihood of bearing damage from tangential deformations as a function of increasing PGA, was shown (Figure 2).

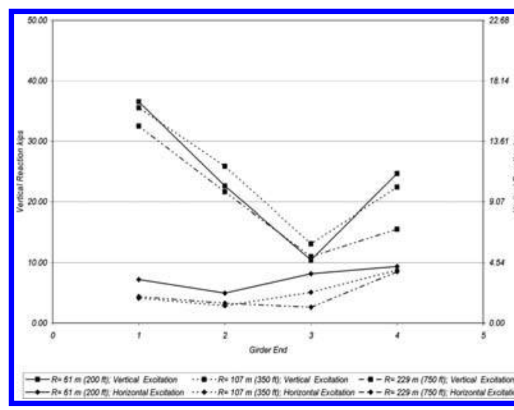

Figure 1. Effect of radius of curvature on maximum vertical reactions under vertical (red) and horizontal (blue) excitation.

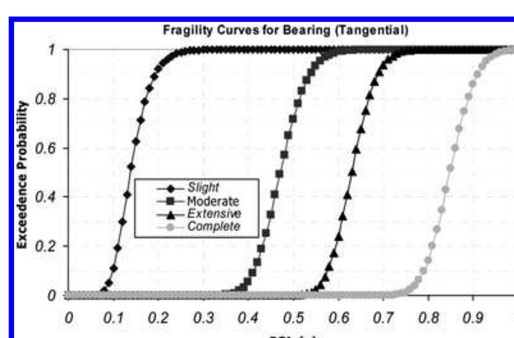

Figure 2. Representative Seismic Fragility Curve, Tangential Bearing Displacement.

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The design of r.c. bridge deck subjected to horizontal actions by strut-and-tie models

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ABSTRACT

Strut-and-Tie Model (STM), conceived by the French builder François Hennebique as a simple representation of a reinforced concrete element subjected to shear and bending, has been later developed by Ritter and Mörsch. In the twentieth century several studies have been carried out about STM and the results constitute the fundamentals of some prescriptions in many international codes. Schlaich et al. (1987) proposed a global approach to the structural design by means of STM.

The Strut-and-Tie method implies that the structure is designed according to the lower bound theorem of plasticity (Schlaich et al. 1987). Since concrete permits only limited plastic deformations, the STM has to be chosen in a way that the deformation limit (i.e. the capacity of rotation) is not exceeded at any point within the structure before the assumed state of stress is reached in the rest of the structure. This ductility requirement is fulfilled by adapting each element of the model to both the direction and size of the internal forces as they would come from the theory of elasticity (Schlaich et al. 1987).

It is often not necessary a deep knowledge of the Strut-and-Tie method to find truss models that best fit the regions under study. This is also due to the fact that often it is possible to adapt well known pre-solved examples to the analysed case.

In non-standard cases the development of the 'optimum' truss model can require not only an expert designer but also it can be extremely time consuming. This is the reason why many procedures (e.g., Load Path Method, optimization criteria), that aim at finding the most 'accurate' solution with the minimum 'effort', have been proposed in the last decades.

In this paper, taking as example the application of the Strut-and-Tie Method to a typical r.c. bridge deck subjected to horizontal transversal action, the need of a step-by-step procedure that goes from the comparison with pre-solved similar cases to the use of the Load Path Method (Schlaich et al. 1987, Palmisano et al. 2002) and optimization criteria (e.g. Bi-directional Evolutionary Structural Optimization method, Yang et al. 1999) has been highlighted.

Moreover, for the case under study, the influence on the model of both the geometry of the bridge slab and the distribution of longitudinal and transversal deck beams has been investigated.

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Hybrid seismic isolation design of Sakarya-II Viaduct in the proximity of the North Anatolian Fault

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ABSTRACT

In this paper, the seismic isolation (SI) design of the Sakarya-II viaduct located within 300 meters of the North Anatolian fault in Turkey is studied. Sakarya-II is a 385-m long, 9 span, slab-on-steel-girder viaduct. A view of the bridge is presented in Figure 1.

Strong near-field characteristics such as directivity and liquefaction are expected due to soft soil conditions under the scenario earthquake having a magnitude of $M_w = 7.5$.

Preliminary analyses indicated that using a conventional SI-system results in very large isolator displacements and forces due to strong directivity effects expected at the bridge site. Consequently, a hybrid SI-system is chosen for the bridge to minimize the magnitude of the isolator displacements and forces. The hybrid SI-system consists of four natural and two lead-rubber bearings supporting the six steel-girders at each substructure.

Lead cores in the lead-rubber bearings are designed to have an overall characteristic strength equal to 10% of the superstructure weight. This large characteristic strength is intentionally chosen to limit the isolator displacements under near-field effects. Furthermore, the supplementary four natural rubber bearings over each substructure provided additional stiffness to achieve a small post elastic period to deflect the earthquake input energy transferred through the underlying soft soil and to limit the isolator displacements under near-field effects.

Nonlinear time history analyses of the viaduct with hybrid SI system have revealed that the hybrid SI-system reduced the sensitivity of the bridge response to the characteristics of the near-fault earthquake and liquefaction. Superstructure with steel girders resulted in lower earthquake forces due to its lower weight compared to a concrete superstructure.

Prototype testing of lead rubber bearings utilized in the viaduct is a challenging task due to dimensions and test velocities of the bearings. Lead rubber bearings have 1100 mm and 1400 mm diameter. Lateral design displacement of the bearings is ±850 mm. Test velocity is calculated as 2.2 m/sec. Static prototype and dynamic prototype tests were performed. Static tests were performed at bearing manufacturer Alga Spa, Milano laboratory. Dynamic prototype tests were performed at EUCentre laboratory, Pavia, Italy. Very high demands in dynamic prototype tests resulted in peculiar problems in the test equipment. Tested bearings were not damaged during dynamic prototype tests although a very stringent test procedure was applied. View of a lead rubber bearing after dynamic prototype test is presented in Figure 2.

Figure 1. View of the bridge.

Figure 2. Lead rubber bearing after dynamic prototype test.

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Seismic abutment-deck interaction of a four-span R/C bridge model

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ABSTRACT

A quarter-scale, four-span, 33.5 m (110 ft) long asymmetric conventional reinforced concrete bridge model was tested using the shake table system at the University of Nevada, Reno. The bridge received biaxial horizontal motions at the bases of the three two-column bents from three separate shake tables. In addition, the bridge was simultaneously subjected to longitudinal motions from actuators attached to the abutment seats. The ground motions, based on a 1994 Northridge earthquake record, were applied in seven test runs with increasing amplitudes. This paper focuses on the nonlinear computer modeling of the bridge including the interaction between the bridge deck and the abutments, the in-plane rotation of the bridge, and the corresponding transverse bent displacements.

A three-dimensional finite element model of the bridge was developed in OpenSees (references are given in the main paper). A view of the computer model is shown in Figure 1(a). Appropriate masses were assigned to the bridge deck nodes as shown by the solid circles in Figure 1(a). The elevated nodes above the bridge deck correspond to the masses of the additional blocks that were placed on the bridge. The bridge columns were modeled with the OpenSees' *nonlinearBeamColumn* element with a fiber-discretized cross section composed of *Concrete01* and *Hysteretic* uniaxial materials for concrete and steel, respectively. Column bond-slip moment-rotation behavior was represented by *zeroLength* elements with *Hysteretic* uniaxial material placed at the column ends. The superstructure was assumed to stay within the linear-elastic range during the seven experimental tests; thus, *elasticBeamColumn* elements were used for all the bridge deck. The biaxial earthquake motions were imposed to the column bases as shown by the double-headed arrows in Figure 1. The longitudinal actuator motions were imposed on both the North and the South abutments. These motions were obtained from a pre-test

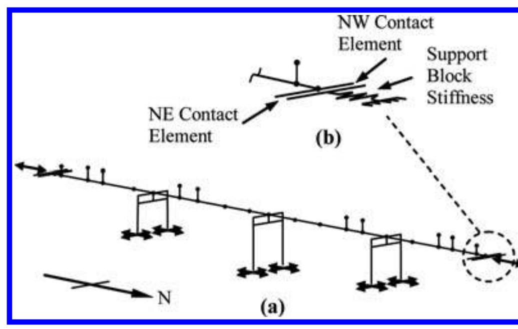

Figure 1. OpenSees model of the four-span bridge: (a) overall model, and (b) North abutment.

response history analysis of the bridge that included an abutment spring representing the force-displacement relationship of a typical seat-type abutment.

The abutment-deck interaction was modeled by a set of two contact elements at each abutment. Furthermore, realistic abutment gap openings were included that were based on the start and the end of each test run. Several sensitivity analyses were conducted. The results of the friction sensitivity analysis gave high variability in the calculated transverse residual displacements. In some cases, a small perturbation in one of the contact element friction coefficients produced substantially different results. However, using the average of the residual displacement square root of the sum of the squares (SRSS) within each of the seven cases considered, it appears that imposing friction at the NE or the SW or a combination of the NE and SW produced transverse bent displacements that were closer to the experimental results. More abutment contact with the NE and the SW corners are consistent with the observed deformed shape of the bridge deck.

Given the variability of the predicted responses, obtaining a set of optimal friction coefficients is difficult. Through a large number of simulations, it may be possible to obtain such an optimum set.

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Influence of shear key modeling on the performance of bridges under simulated seismic loads

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ABSTRACT

Exterior shear keys are commonly used at the abutments of small to medium span bridges to provide transverse support for the bridge superstructure under small earthquake. When subjected to large magnitude earthquakes, these shear keys have been assumed in design and analysis of bridges to perform as structural fuses. Previous experimental research results clearly show that shear keys at the abutments do not perform as structural fuse. In fact and according to current reinforcing details, shear keys possess significant post damage residual capacity. Analytical results show that the residual capacity of shear key can have a considerable impact on the seismic response of medium span bridges.

Another modelling aspect of these shear keys, is the use of a single finite element or a series of elements that can represent the spatial distribution of the shear keys along the abutment length. Results have shown that using multiple elements the bridge superstructure develops rotational restrains at the abutments, which are not properly replicated when using a single element. This restraining of the superstructure at the abutments can be shown analytically to decrease substantially the displacement ductility demand on the supporting columns. Furthermore and in conjunction with the modelling of the shear key gaps, this paper presents results from a series of finite element analysis that were conducted using the open source finite element program OpenSees. This paper provides detailed results from multiple modelling cases that consider the shear keys hysteretic response, spatial distribution modelling and gapping at the abutments. These analyses are presented and discussed in further detail in the paper.

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A case study of analysis techniques for precast segmental bridges subjected to vertical seismic excitation

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ABSTRACT

Precast segmental construction of bridges can accelerate construction and minimize the cost of medium span bridges in environmentally sensitive and difficult to access locations. Consequently, the use of precast segmental construction is on the rise worldwide. Numerous analyses techniques exist to estimate seismic demands on precast segmental bridges. These techniques range from a simple linear elastic response spectra analysis to a complex nonlinear time history analysis using truss elements to model the segment joint behavior. Each technique has benefits and drawbacks and every bridge may have a different set of performance requirements, thus it can be difficult to determine the most appropriate modeling technique to accurately and efficiently assess the seismic demands.

This paper presents a case study of analysis and modeling techniques to determine the most appropriate modeling method to estimate the vertical seismic demands on precast segmental bridges. Three separate two-dimensional analytical models were developed for a 91 m (300 foot) span bridge. The first model simulated the superstructure using only linear elastic members. The second model simulated the superstructure using simple non-linear elastic lumped plasticity members. The third model utilized numerous axial only gap-hook elements to explicitly model the behavior of concrete and post-tensioning tendons at critical superstructure segment joints. These models were subjected to five different earthquake hazard levels. The results were compared to determine the most appropriate modeling method for 'Ordinary' and 'Important' segmental bridges and were used in the development of seismic design guidelines for precast segmental bridges for the California Department of Transportation.

The results indicated that variations in the damping value of different modes due to Rayleigh damping caused the difference between elastic time history and modal analyses. The Rayleigh damping for vertical analyses should be based on the dominant vertical modes, not the dominant longitudinal mode and 10Hz. It is recommended that Rayleigh damping be defined at frequencies where (i) the cumulative vertical modal mass exceeds 20% and (ii) the cumulative vertical modal mass exceeds 80%.

The results from the linear elastic time history were consistent with the results from the nonlinear inelastic model. Both models indicate that the superstructure segments will exceed the decompression limit state at select segments, primarily the midspan segments under negative bending and only due to the 2500 year hazard. The nonlinear elastic model predicted segment joint rotations that were up to five times larger than the rotations predicted by the nonlinear inelastic model. This is because the NEM used a bi-linear curve that assumed that the segment joint was already fully cracked.

Based on the analyses performed and the results presented in this paper it is recommended that a modal analysis using a linear elastic model should be used for the functional evaluation of 'Ordinary' bridges. For the safety evaluation of 'Ordinary' bridges, it is recommended that designers satisfy the no collapse criteria by checking the capacity of all vertical collapse mechanisms relative to the vertical design spectrum.

For both the functional and safety evaluation of 'Important' bridges, a time history analysis using a nonlinear elastic model is recommended.

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Compressive stress-strain model for high-strength concrete confined with spirals

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ABSTRACT

The design of high-strength concrete columns under seismic loading is not only based on the maximum strength of the column but also the ductility of the column. A combination of the axial stress and strain of the column can be represented as the total amount of energy that the column can absorb before failure. An increase in the strength of the concrete core or increase in maximum strain before failure will increase the amount of energy that the specimen can absorb. For the design of circular column, the method to improve the amount of energy the column can absorb is based on the spacing of the transverse reinforcement used. This paper presents a stress-strain model to predict the axial behavior of high strength concrete confined with both single spiral and two opposing spirals. Data from twenty-one of high-strength concrete columns along with ten normal-strength concrete columns is presented and used to derive the model. To have an understanding of the correlation between the amount of transverse reinforcement and the strength of the concrete core, the range from the minimum spacing to the maximum spacing as specified by ACI-318 was considered. Also, the effect of the amount of longitudinal reinforcement was considered in deriving the model.

The data from the columns that were constructed by Marvel (2008) along with Al-Qattawi (2004) in their research was used to derive equations to find the f'cc for a column using cross-spiral configuration.

In order to find the full trend of cross-spiral reinforcement, both normal-strength concrete and high-strength concrete are used to predict the strength increase caused by the spiral reinforcement. In the case with normal-strength concrete, the increase is derived from previous equations that are based on the concrete being able to fully engage the steel reinforcement at maximum loading. With high-strength concrete being more brittle, the maximum strength of the concrete is sooner than when the spirals have been fully engaged which causes the high-strength concrete to follow a different trend. While the equation for a single-spiral column using high-strength concrete has already been determined, the cross-spiral column does not follow the same equation, and this paper offers an equation that predicts the cross-spiral column strength. In either the case of high-strength concrete or normal-strength concrete, the base equations, such as the core effectiveness and the confinement pressure, need to be modified to encompass the cross-spiral configuration in order to obtain the final equation.

Several conclusions can be obtained from the analysis of the 21 high-strength concrete columns along with the 10 normal-strength columns.

- Columns using normal-strength concrete using single and cross-spiral column configuration can be described by the same equation that Mander, Priestly and Park (1988) predicted.
- Columns using high-strength concrete follow a different trend when dealing with single-spiral verses cross-spiral configuration.
- The confinement pressure can be predicted for cross-spiral columns with the same method Mander, Priestly and Park (1988) used for hoops or single-spirals in which the average percentage error from the test samples was 4.68 for normal-strength and 3.28 for high-strength concrete.
- The method Mander, Priestly and Park (1988) used to predict the core effective can be modified to include cross-spiral columns as well.

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SS16: Recent challenging bridge structures
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Assessing and extending the service life of bridges

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ABSTRACT

Traditional bridge management practice is reactive to the needs of specific bridges. When routine inspection identifies that the condition of structures has degraded sufficiently, a more detailed inspection and study is performed to develop a repair approach to address the observed deficiencies. This approach is generally insufficient to assess the remaining service life of a structure and provide the information necessary to develop cost effective rehabilitation plans and/or evaluate the feasibility of reusing components of an existing structure.

Service life-based evaluation techniques historically adapted by the authors for concrete bridge elements, adopt life cycle cost analyses together with durability modeling. This approach provides an effective tool for the development of rehabilitation planning and can also reduce the overall life cycle cost of a structure by preventing the onset of more severe degradation mechanisms.

Effective service life analysis integrates inspection and testing methods to assess existing condition and identify causes of degradation observed along with quantitative analysis of anticipated structural vulnerabilities and a statistical evaluation of the likely progression of deterioration. Often the analysis will consider a rehabilitated condition.

Over the last decade, the authors have developed and implemented this approach for evaluating service life of high-value bridge assets. Three case studies involving the condition evaluation of concrete substructures will be described and presented, to illustrate the tools and methodologies available for evaluation of structures, to determine existing conditions, assess remaining service life, and to develop appropriate and timely rehabilitation schemes. These case studies are gathered from significant structures with conditions typical nationwide, including the northern climate, deicer-affected and marine climate-influenced environments.

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

The role of finite element analysis in bridge assessment and design

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ABSTRACT

Finite element (FE) analysis is being used more and more these days for bridge engineering because of the more economical and accurate assessments and designs its use produces.

This paper describes the basic development of FE analysis and illustrates the role that it can play in just some areas of bridge analysis, assessment and design with reference to a number of real-life FE bridge case studies carried out by US and UK consultants.

FE analysis allows for a more rigorous design or analysis approach to be adopted which is often significantly more accurate and economical than codified methods. To illustrate this mention is made of a study by Catbas & Gokee into the use of AASHTO distribution factors which produced results found to be typically 25–40% over-conservative when compared to FE solutions. A similar load capacity study by Atkins for a pair of braced steel beams showed that a nonlinear FE analysis can provide a significantly larger collapse load factor than that obtained from a design code.

When the diaphragms of a steel box bridge do not comply with assessment code criteria, FE analysis will allow a detailed analysis to be performed in order to help prove the integrity of the design. Raith Bridge and the Midlands Link Viaducts, both in the UK are given as examples.

Two, US steel truss bridges – the US-2 over Cut River, and the M-55 over Pine River, and a typical UK steel 'through' bridge at Hackbridge (which was analysed for a proposed increase in vehicle axle weight), are given as examples of how load capacity can be proved on structures even if the latter was 'seen' to fail according to a design code.

Bridge assessments requiring retrofit solutions are another area where FE analysis can assist greatly with what-if scenarios. A two span deck truss bridge with a lower lateral bracing system that vibrated excessively under truck crossings and required remedial action is mentioned along with mention of solutions for the Milford-Montague bridge retrofit.

The role that FE plays in creating innovative new designs is illustrated by the Gateshead Millennium Bridge in the UK, and the initial proposed New Mississippi River crossing by Modjeski & Masters These two boundary-pushing structures show the real benefits of using FE analysis.

When a re-design is undertaken using FE analysis there can be occasions when great savings can be made and the Estero Parkway Flyover in Fort Myers, Florida, is cited as an example of this.

For erection engineering two cable stayed footbridges, the Wichita Riverfront Footbridges and the Bob Kerrey Pedestrian Footbridge show how fabrication geometry can be checked and cable tensioning can be optimized to speed-up construction.

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Challenges in design, rehabilitation and construction of bridge structures

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ABSTRACT

Engineering challenges may appear in the form of situations or constraints that require engineers to respond in a creative way to them. They offer opportunities to engineers to demonstrate their technical skills and creativity.

This paper refers to four case studies in design, rehabilitation and construction where innovative approaches were used to solve engineering challenges. In each case, engineers analyzed the situations, explored various possibilities and selected appropriate solutions.

The key to addressing any challenge is to understand the issues. TQE® is Alfred Benesch & Company's unique approach for understanding and addressing these challenges. Challenges to design, construction and rehabilitation require consideration of constructability, integrity of existing structures during rehabilitation, cost and schedule. Constructability includes construction staging, construction method, field conditions and maintenance of traffic.

The first case study is the shortening of a truss which had never been tried before. The design challenge was to widen the roadway within the limits of the truss bridge. This project evolved due to the need to improve an interchange at the north end of the truss. The interchange was built in the late 1950s and had numerous design deviations from current standards, including several sub-standard safety features. Several places within the interchange had accident rates that were more than ten times higher than statewide averages. Thus, the interchange needed improvement, but the proximity of the truss posed a significant site constraint. Shortening the truss was proposed as a solution to rebuild the end with a wider cross-section. Shortening part of the anchor arm has a major impact to the cantilever and suspended span. Benesch engineers devised a load-transfer system to cut the truss in a safe way.

The second case study describes the design and construction of a bridge in eight days. A major fire destroyed a Metra rail bridge impacting the daily commute of thousands of riders. Temporary and permanent solutions were needed immediately. Metra had never before faced a service disruption of this magnitude. They quickly responded by assembling a team to assess existing conditions, develop a design-build plan and execute the reconstruction of this critical commuter line. Planning and design of a temporary structure started immediately and proceeded concurrently with the removal of the remains of the double track bridge. The team designed and rebuilt the structure in eight days.

The third case study is the staged deck replacement of a tied arch bridge where the solution was carefully implemented in stages. The two-way bridge was supported by a single arch structure. Part-width construction caused an asymmetrical loading condition. The challenge to the designer and the contractor was to ensure that the behavior using the Finite Element Model was replicated in the field and that the appropriate measures during the different stages of construction were taken in order to produce the desired deck profile. Some of these measures included surveying the top of the ties before and after each stage and making sure that the elevations are within an acceptable tolerance of the elevations shown on the construction drawings. The project was completed successfully without changes to the design procedure.

The fourth case study details the construction of a curved bridge where local materials were utilized to reduce the construction cost. The girders had to be held with temporary supports to restrict its vertical and lateral movements. Available beam sections were stacked over to provide the needed section modules and concrete blocks to stabilize the support system. The temporary structures were designed, fabricated, constructed and the decks poured within three months. All deflections and lateral movements were well within the allowed amounts.

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Innovative design approach to a GFRP pedestrian bridge: Structural aspects, engineering optimization and maintenance

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ABSTRACT

This paper deals with a pedestrian bridge made with pultruded GFRP (Glass Fiber Reinforced Polymer) profiles. Here are highlighted solutions from a structural point of view and from a maintenance standpoint. The footbridge is now under construction and it is entirely designed with GFRP Profiles.

Design criteria of short-span bridges made with pultruded profiles have been analyzed focusing on the most relevant challenges, especially those related to global instability and dynamic performance, and finally comparing with similar items for steel bridges.

The relatively low modulus of glass fibre FRPs also requires that buckling characteristics be assessed a little more critically than would be the case with a steel component, which generally have an abundance of stiffness relative to their strength.

The high strength, high fatigue resistance, lightweight, and corrosion resistance of composites are highly desirable characteristics for bridge applications and the maintenance program in the design of the bridge demands only a biennial inspection.

At present the construction of such a bridge certainly is a fascinating matter in that, despite several studies been carried out, the actual application has proven difficult. One of the reasons is the innovation being related to the use of new materials; another one is linked to a likewise pioneer construction management.

Furthermore – from a point of view of public construction – the use of a non-conventional material has to deal with a lack of normative and of a well consolidated price history.

A detailed numerical analysis of the dynamic structural answer of the All-GFR pedestrian bridge strictly related to the low weight has been carried out, and the evolution of construction design of lightweight structures as pedestrian footbridges has been pointed out.

In the all-GFRP construction's field it is necessary to pay attention particularly to the dynamic performance expected in function also to the evaluation of the fundamental frequency value, both for traditional frame structures, made by beam-column connection, and vehicular or pedestrian bridge. In fact, the major benefit due to the construction made by GFRP material

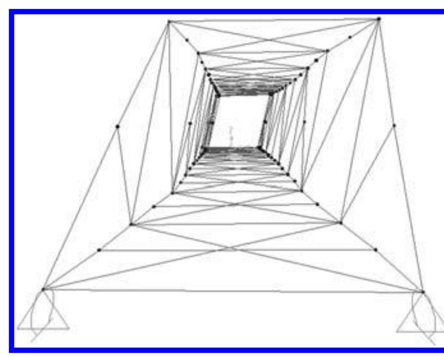

Figure 1. Mode 1.

Table 1. Mechanical characteristics of GFRP material.

Items	Properties
Density	1800 kg/m^3
Long. Tensile strength	200–500 MPa
Modulus elasticity (tensile)	20.000–30.000 MPa

is linked to the very low weight (for structural employment it become from 1800 to 2000 kg/m^3 and 60% of fibers) but it could be considered as a weakness aspect as to the risk of the resonance phenomenon. Besides, a possibility due to the very light and durable structural material characterized also by very high value of strength (but also with not negligible deformability level), represents anyhow an extraordinary opportunity in term of innovative design approach.

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SS17: Life-cycle design of structural systems
Organizers: F. Biondini & D.M. Frangopol

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Structural health monitoring role in bridge life cycle analysis

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ABSTRACT

The primary objective of asset/bridge management is ensuring safety of users while maintaining or reducing costs of ownership. Decision making tools are often used by bridge owners to assist in reaching these objectives and one such tool is life cycle analysis. In particular, the Bridge Life Cycle Cost Analysis (BLCCA) technique is extensively advocated in the infrastructure arena. But decision making capabilities can be enhanced by an accurate estimation of the benefits and life span of the structure. Thus, Bridge Life Cycle Analysis (BLCA) is a metric that requires consideration on all three components and includes BLCCA, BLCBA (Bridge Life Cycle Benefit Analysis) and BLSA (Bridge Life Span Analysis).

Since accuracy of BLCA depends mainly on the quality of information and assumptions that are used, it is clear that SHM can play an essential role in all of the BLCA components (see Figure 1). One major issue in this type of analysis is lack of reliable data that is needed for accurate results. Structural Health Monitoring (SHM) is one of the tools that have the potential to assist in BLCA. The paper will discuss this by

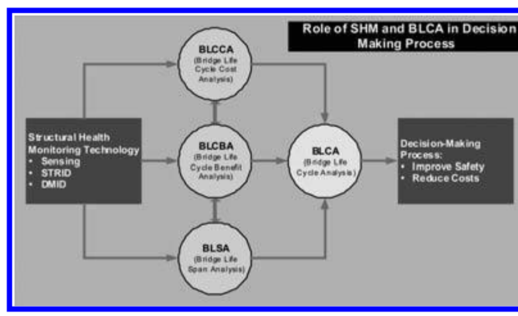

Figure 1. SHM Role in BLCA and Decision Making.

exploring two modes where SHM can provide advantage to the BLCA process. Following these modes can help decision makers in achieving their safety and cost optimizations goals. (Ettouney and Alampalli 2010).

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Lifetime seismic performance of concrete bridges

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ABSTRACT

The seismic performance of concrete bridge structures under material degradation induced by the diffusive attack of aggressive agents over the structural lifetime is investigated. In a traditional seismic ductile design philosophy, the inelastic response of bridge occurs in the bridge substructure with development of plastic hinge regions at the bottom and/or top of the piers (Paulay & Priestley, 1992). As a consequence, if aggressive agents act on bridge piers, the structural behavior varies during lifetime and hence the seismic performance is generally reduced in terms of both ductility and strength (Biondini *et al.*, 2004). The interaction between the diffusion process and the corresponding mechanical damage can be complicated if different geometrical parameters, as distribution of pier heights and deck stiffness, are considered. In fact, these aspects can considerably affect the overall seismic performance of bridge systems.

In this paper geometrically regular and irregular continuous bridge structures with piers having box cross-section are analyzed. The time-variant structural performance of the critical cross-sections of the piers is evaluated in terms of bending moment vs. curvature relationships. Time-variant pushover analyses are then carried out over the structural lifetime to assess the global performance of the system in terms of base shear force and displacement ductility (Biondini *et al.*, 2010). Seismic strength is compared with design actions corresponding to prescribed peak ground accelerations, and seismic indicators, such as overstrength and global behavior factor, are investigated over the structural lifetime.

The results confirm that the lifetime seismic performance of concrete bridges is affected by the diffusive attack of aggressive agents that induce the deterioration of concrete and corrosion of reinforcement (CEB, 1992). Moreover, the material deterioration has different influence on the bridge systems analyzed. Bridges for which the ductility demand tends to be equally distributed along the piers are less affected by the effects of material deterioration. Therefore, in order to mitigate the effects of material deterioration on bridge systems, a suitable configuration of bridge pier profiles has to be achieved. In particular, change of section profile or reinforcement along bridge pier should be implemented in order to equally distribute the ductility along bridge piers. Finally, the results show that the behavior factor, assumed as constant value in the seismic design codes (CEN-EN 1998-1, 2004), varies over time depending on the environmental exposure of the structure. Further investigations are then needed to properly identify the structural bridge typologies more vulnerable to lifetime material degradation effects.

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Structural robustness and redundancy of deteriorating concrete bridges

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ABSTRACT

In structural design the terms robustness and redundancy are often used as synonymous. However, they denote different properties of the structural system. In fact, structural robustness can be viewed as the ability of the system to suffer an amount of damage not disproportionate with respect to the causes of the damage itself. Structural redundancy can instead be defined as the ability of the system to redistribute among its members the load which can no longer be sustained by some other damaged members after the occurrence of a local failure.

Moreover, structural robustness and redundancy are usually investigated with respect to damage suddenly provoked by accidental actions and abnormal loads. However, damage could also arise gradually in time from aging of structures. Depending on the damage propagation mechanism, such kind of damage may also involve disproportionate effects and alternate load redistribution paths. These effects are particularly relevant for bridge structures due to their environmental exposure. Notable events of bridge collapses due to the environmental aggressiveness and related phenomena, such as corrosion and fatigue, include for example the Silver Bridge in 1967, and the Mianus River Bridge in 1994. For these reasons, structural robustness and redundancy should be considered as key factors for a rational approach to life-cycle design of deteriorating structure and infrastructure systems. It is therefore of great interest to investigate the relationship between robustness and redundancy and its evolution in time under a progressive deterioration of the structural performance.

Structural systems under progressive damage have been investigated to identify suitable measures for structural redundancy (Frangopol and Curley 1987) and structural robustness (Biondini and Restelli 2008). More recently, the time factor has been explicitly included in a lifetime scale for a time-variant measure of structural robustness (Biondini 2009) and structural redundancy (Okasha and Frangopol 2010).

Based on these approaches, the time-variant robustness and redundancy of deteriorating concrete bridges are investigated in this paper by taking uncertainties into account. The effects of the damage process are evaluated by using a proper methodology for life-cycle assessment of concrete structures exposed to diffusive attacks from environmental aggressive agents (Biondini *et al.* 2004). Time-variant measures of structural robustness and structural redundancy are developed at the global level with respect to the loads associated to the first local failure and to the structural collapse (Biondini and Frangopol 2008).

The proposed approach is illustrated through the application to the assessment of time-variant structural robustness and structural redundancy of a concrete arch bridge. The results demonstrate that robustness and redundancy are different structural properties, which may exhibit opposite trends over time, both in deterministic and probabilistic terms, depending on the damage scenario.

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Life-cycle analysis of a new composite material for bridge pavement waterproofing

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ABSTRACT

One of the most critical issues related to bridge management and preventive maintenance is deck waterproofing.

Infiltration of meteoric and rain-wash water into the bridge asphalt pavement through unsealed cracks could produce a series of chain events that leads towards an accelerated decay of the bridge structural adequacy. Actions like crack sealing and general waterproofing are very important; they can in fact slow down the deterioration and reduce the life-cycle cost.

An effective protection system against water is one of the key factors for a functional and efficient bridge management plan. However, benefits associated with such maintenance activities are seldom correctly accounted and, in many instances, they are not even considered at all. Consequently many road bridge authorities do not include preventive maintenance activities in their management infrastructures programs. Recent research has resulted in the development of new technologies in the field of crack sealing and pavement waterproofing. In this context, a new composite material for waterproofing interlayer membranes, to be "cold applied" (with great benefit for environment and workers) was studied and also tested in full scale experiments by Politecnico of Milano researchers. This paper presents the main characteristics and performances of this new technology, applied to bridge pavements.

The Authors carried out a Life Cycle Cost Analysis in order to estimate the long term benefits of "*Preventive Maintenance*" strategies (*PM*), making use of a waterproofing interlayer membrane vs a "*Do-Nothing*" alternative (*DN*). The analysis was developed considering a sample road bridge (with unitary length), with a transversal section constituted by two traffic lanes for each direction. Agency and User costs were defined considering reconstruction, preventive maintenance operations and residual service life value.

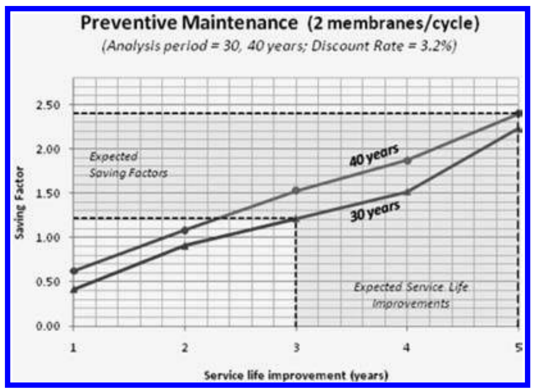

Figure 1. Savings due to preventive maintenance with waterproofing membrane (2 membranes/cycle).

Costs were discounted and summed in order to obtain the Life Cycle Cost for each alternative; results are expressed through a "*Saving Factor*" (Fig. 1). The result, derived from both economical analysis and practical experience, showed significant benefits due to the adoption, on bridge deck pavement, of preventive maintenance treatment with waterproofing membrane, allowing a remarkable overall saving.

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The fatigue limit states of the AASHTO LRFD Bridge Design Specifications

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ABSTRACT

Recently, a second fatigue limit state was included in the AASHTO *LRFD Bridge Design Specifications* (AASHTO, 2010). This limit state, termed the Fatigue I limit state, was added to explicitly differentiate infinite-life fatigue design from finite-life fatigue design. The original fatigue limit state is now termed the Fatigue II limit state and provides finite-life designs. The live-load load factors of the two fatigue-and-fracture limit-state load combinations excerpted from LRFD Table 3.4.1-1 are given in Table1 below.

The load factors of the fatigue-and-fracture limit-state load combinations represent multipliers which transform the fatigue load of LRFD Article 3.6.1.4, the HL-93 design truck with a fixed rear-axle spacing of 30 feet, into nominal fatigue loads for use in the two fatigue-and-fracture limit-state load combinations. They do not represent uncertainty of the nominal fatigue loads.

The Fatigue I load factor of 1.50, from Table 1, when multiplied by the stress range due to the passage of the LRFD fatigue truck produces the maximum stress range considered for fatigue design. This maximum stress range for fatigue design has been characterized as the stress range with a 1 in 10,000 probability of being exceeded.

Table 1. *LRFD Specifications* fatigue live-load load factors.

limit-state load combination	live-load load factor
Fatigue I	1.50
Fatigue II	0.75

The Fatigue II live-load load factor of 0.75, from Table 1, times the fatigue load yields an effective truck which replicates fatigue damage of the entire distribution of truck traffic.

The additional fatigue limit state has no impact on the design of steel and concrete highway bridges. The original fatigue resistance equation with its lower bound has been replaced with two separate fatigue resistances equations. The real impact is on the perception of bridge designers. Now when they are providing an infinite-life design, they explicitly know it.

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Reliability assessment of reinforced concrete beams rehabilitated with CFRP sheets

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ABSTRACT

In a time of greater demands posed by a deteriorated infrastructure in both developed and developing countries, it is clear that the design of retrofit/ strengthening systems for bridges is of utmost importance. Nowadays, the utilization of externally bonded carbon fiber-reinforced polymers (CFRP) sheets has found a large acceptance in the rehabilitation of reinforced concrete (RC) structures. This fact is easily understood since these materials present a number of favorable characteristics such as lightweight, relatively easy to install, noncorrosive, and high tensile strength that make them suitable for use as retrofit/ strengthening systems.

Accordingly, guidelines for the design and construction of externally bonded FRP systems for strengthening concrete structures have been proposed, e.g. FIB Bulletin 14 (2001), ACI 440.2R (2008), and JSCE 23 (1997). Design codes for new structures have been calibrated via probabilistic methods; similarly, design recommendations for the rehabilitation of existing structures shall be based on such methods. The implicit reliability levels attained must be assessed; additionally, these levels shall satisfy a prescribed target reliability index.

In this study, the reliability assessment of RC beams, retrofitted by externally bonded CFRP sheets is performed. The flexural retrofit has been designed according to the ACI 440.2R (2008) guidelines. For comparative purposes, the resulting reliability levels for undamaged and damaged beams are also evaluated.

An important task in the reliability analysis process is the realistic description of the beam resistance. It is well known that the debonding of CFRP sheets plays an important role in the performance of retrofitted beams. As such, a nonlinear finite element model, capable to deal with the debonding process and coupled to a Monte Carlo simulation procedure, is used in the determination of the statistics of the beam failure loads (Paliga 2008). Failure loads have also been calculated through a simplified procedure. Two different procedures are used in the reliability analysis.

Reliability indexes (and corresponding failure probabilities) are obtained for undamaged, damaged, and retrofitted beams (Table 1). These results are compared and discussed in the light of: (i) the reliability procedure used in the analysis, (ii) the adequacy of the design scheme used in the repair of the beam, and (iii) the safety evaluation of existing versus new structures. It is shown that reliability levels can be significantly overestimated depending on the selected reliability procedure.

Table 1. Probability of failure and reliability index.

	Undamaged		Damaged		Retrofitted	
r	P_f	β	P_f	β	P_f	β
3	2×10^{-6}	4.61	6.9×10^{-3}	2.46	1.0×10^{-6}	4.75
1	1.1×10^{-4}	3.70	1.8×10^{-2}	2.10	7.5×10^{-5}	3.79
1/3	1.2×10^{-3}	3.04	3.9×10^{-2}	1.76	1.1×10^{-3}	3.07

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Time varying risk modeling of deteriorating bridge infrastructure for sustainable infrastructure design

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ABSTRACT

Time varying risk assessment models represent a new approach for characterizing risk due to multi-hazard environments, such as extreme seismic events and long-term structural deterioration, that vary independently over time. The proposed method quantifies the impact of time-dependent deterioration of reinforced concrete elements on seismic risk for a single infrastructure element over time through shifting structural fragility curves. A more comprehensive, probabilistic reinforcing steel deterioration model is presented that accounts for the effects of uncracked and cracked concrete covers through the inclusion of pitting corrosion in cracked concrete. When coupled with a probabilistic seismic response model for reinforced concrete structures, the result is a comprehensive assessment tool for the structural fragility of infrastructure systems exposed to multi-hazard environments over long periods of time.

The effects of including pitting corrosion increase the probability of failure of RC elements throughout their service life, most notably after 25 years. At a moderate age for bridge structures (50 years) the probability of failure of an RC bridge column during a seismic event with $S_a = 0.8$ g increases by 2.4 times as a result of accounting for pitting corrosion over previous models as seen comparing Figures 1 and 2. Such results may point toward an existing systemic underestimation of the fragility of infrastructure stock potentially exposed to chloride-induced corrosion in seismic areas.

Results also show that designers can have a significant effect on reducing the multi-hazard risk profile investigated in this study by altering basic design parameters such as concrete cover. As seen comparing Figures 2 and 3, the increase of cover thickness from 51 mm to 127 mm virtually negated the effects of chloride-induced corrosion thereby preserving the initial structural fragility curve throughout the expected 100-year service life.

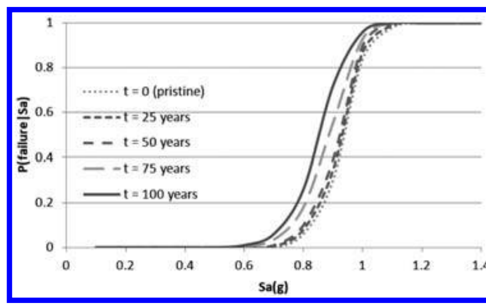

Figure 1. Column failure fragility with uniform corrosion, (Concrete cover = 51 mm).

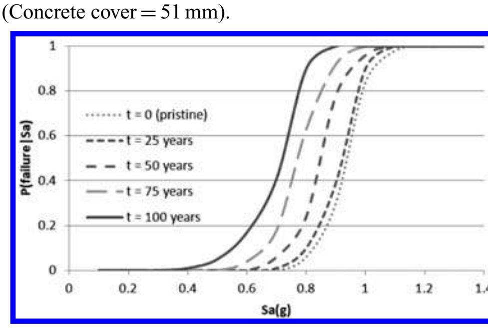

Figure 2. Column failure fragility with pitting corrosion (Concrete cover = 51 mm).

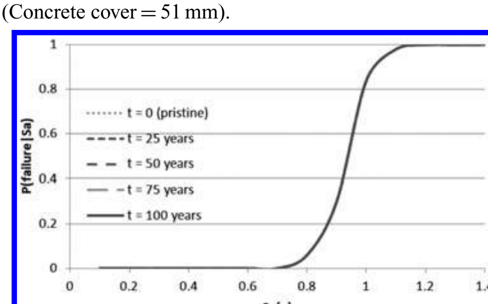

Figure 3. Column failure fragility with pitting corrosion (Concrete cover = 127 mm).

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Study of remaining fatigue life of Brazilian concrete bridges

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ABSTRACT

In this paper a parametric study on the fatigue life time of short-span reinforced concrete roadway bridges is performed. For this study a simulation model has been developed, which includes the most significant sources of uncertainty in the definition of traffic action on bridges as well as of structural response.

From experimental data collected on dynamic tests, curves for natural frequencies and dynamic amplification factors were provided, (Fig. 1). Impact factors obtained from the tests with higher speeds were found larger than corresponding values recommended by bridge codes. In addition to dynamic tests, concrete samples were extracted from bridges under investigation. Modulus of elasticity and compressive strengths of existing concrete were obtained from specimens extracted from bridges. Dimensional checking of bridges under investigation was also carried out. Values measured on sites were compared with dimensions specified on bridge design, and respective differences were registered.

For the parametric study, a common structural type of bridge built on Brazilian roadway net since 1950 were adopted, a two girder-slab reinforced concrete bridge, with 3 spans, ratio of 0.25 : 1 : 0.25, and free edges on extremity spans.

To represent traffic action, real traffic data registered on Brazilian roadways were considered. Natural uncertainties inherent to the definition of traffic action, as type of vehicles, intervals between vehicles' arrivals, its total weight and its velocity, were adopted as random variables.

Monte Carlo simulations were performed with main spans of 7, 10, and 13 meters, within the range of investigated bridges, and reinforcement ratios of 1, 1.5 and 2%. Geometrical and mechanical characteristics of bridges were considered as random variables based on site survey.

A statistical definition of the S-N curves and the Miner summation at failure was adopted to deal with the uncertainty in the prediction of the number of cycles to failure.

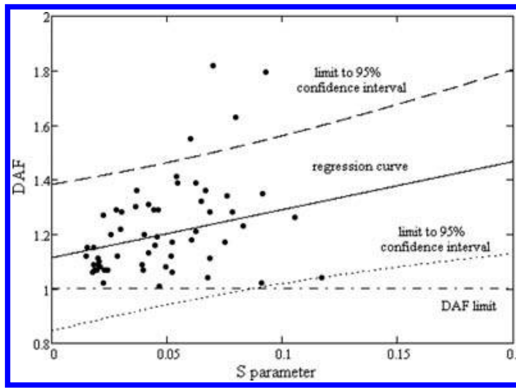

Figure 1. Experimental DAF-S parameter relationship.

To fatigue analysis, resistance and solicitation were coupled in the Limit State Function, and reliability indexes were derived using FORM (First Order Reliability Method).

The results of the current investigation show that fatigue reliability index can be lower than target reliability indexes, mainly in shorter spans bridges, between 7 and 10 meters. Besides that, a substantial increase in the reliability index is observed with increasing reinforcement ratio. For new structures design this fact suggests a border line of cross-section stiffness to be applied on these kind of structures on the aim of reach 100 year lifetime required by codes. This border can be investigated with the procedures developed in this study.

Due to low values of fatigue reliability obtained in this study, simulations to analyze Ultimate Limit State should be performed.

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SS18: Optical monitoring techniques for bridge maintenance & safety

Organizers: S. Sumitro & H. Matsuda

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Real-life applications and experiences with fiber optic bridge monitoring installations

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1 ABSTRACT

We present a brief overview of real-life bridge structural health monitoring (SHM) installations using fiber Bragg grating sensing technology. We review several project installations and describe the associated successes, challenges and lessons learned for each application. In general, project successes are coupled to improved sensing tools: better sensor packages, simpler and less expensive instrumentation, improved installation techniques, and more efficient data analysis tools. Some shortcomings are the direct result or poor project planning and communications. Particular attention is given to the benefits and economics of instrumenting civil structures – when and how it pays.

Over the last few years, fiber optic sensors (FOS) have seen increased acceptance and widespread use for industrial sensing and structural health monitoring (SHM) applications in composites, civil engineering, aerospace, marine, oil & gas, and defense. Given their EM immunity, intrinsic safety, small size & weight, and capability to perform multi-point and multi-parameter sensing remotely make of FOS an attractive, flexible, reliable and unique sensing solution. One of the most common applications is for strain/stress sensing, but a variety of other parameters such as temperature, pressure, magnetic field, voltage, chemical species and others, can also be measured using them. And, nowadays, a variety of rugged sensors and interrogation instruments are commercially available—offering attractive performance, ease of installation, reliability and reasonable pricing.

A variety of discrete fiber strain sensors based on Fabry-Perot (FP) cavities and fiber Bragg gratings (FBGs), as well as distributed techniques based on Raman or Brillouin scattering methods have been developed along with pertinent interrogation electronics and instruments, many of which are already commercially available.

The technical requirements imposed by many SHM applications combined with the increased demand for practical, reliable, field-portable and low cost equipment to perform on-line measurements, has led to a new wave of commercial product and a push for the development of new optical fiber sensor solutions. However, can fiber optic sensing address the application needs and provide an economical, effective and reliable bridge monitoring alternative? Owners must manage and ensure the safety of their bridges—even as their use might extend well beyond their original design lifetime. Traditionally, most structures rely on strict maintenance procedures, visual inspections, and very few sensors. Furthermore, maintenance operations are expensive; visual inspections can miss critical problems; and conventional sensors often fail in harsh environments. Hence, how can this new technology be satisfactorily proven in the field?

This and other questions are aimed to be answered by way of reviewing a number of real-life bridge installation projects and their associated practical lessons learned, challenges and successes from each bridge installation.

In general, many successes are coupled to improved FOS tools: better sensor packages, simpler and less expensive instrumentation, improved installation techniques, as well as more efficient data analysis tools. Some shortcomings are the direct result or poor project planning and communications. Particular attention is given to the benefits and economics of instrumenting civil structures – when and how it pays.

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Base study on measure of transformation of concrete structures used by digital image correlation method

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ABSTRACT

This study is the basic research for proposing a maintenance method for the structures using full visual field non-contact measurement technique. By using digital image correlation method, bending behavior of a steel and concrete composite girder was measured. The digital image correlation method is to take a surface pattern before and after the deformation by a digital camera and to obtain amount and direction of deformation of the specimen from the brightness distribution of the digital image obtained. The specimens are steel-concrete composite girders using the perfobond-rib shear connector (PBL: Perfo-Bond Leisten) as shear connector. A scope of 1000 mm × 600 mm was photographed by a digital camera. In order to reduce variations in the image caused by factors other than deformation, multiple images were overlaid. In addition, deformations in the vicinity of the target point to be measured were averaged.

It is found that about the same level of accuracy as that by contact type measurement was obtained by the measurement of displacements such as deflection or relative displacement. With regard to the strain measurement, although it has larger variations than the measurement by gauge, its qualitative behaviors were able to be evaluated.

Picture 1. Specimen and camera.

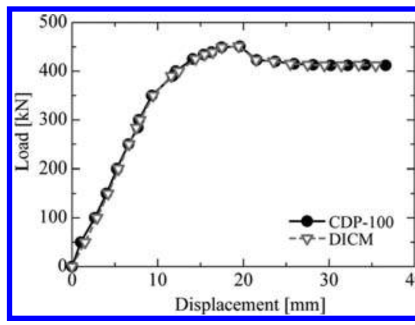

Figure 1. Load-deflection curve.

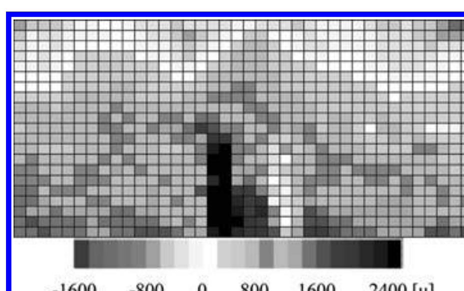

Figure 2. Strain distribution of composite girder by DICM.

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Development of approximation process of existing action stress of pre-stressed concrete by stress relief technique

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1 INTRODUCTION

A concrete structure has high durability and it has been believed that a maintenance is unnecessary. However, in some concrete structures, by degradation of damage from chloride attack, carbonation, an alkali aggregate reaction, etc., it cracks and rebar corrosion etc. has appeared notably in recent years.

Especially the pre-stress concrete structure (following PC structure) built over varieties, such as a highway bridge, a railroad bridge, and a building, has introduced pre stress into the structure with PC tendon. PC bridge in which it is located near the seashore shows the example which PC tendon fractured by damage from chloride attack to photo 1.1.

Degradation, such as carbonation, chloride attack and an alkali aggregate reaction, reduces the durability ability of a concrete structure. A degradation factor permeates into concrete and it is begun to corrode rebar and PC tendon in concrete. If degradation advances further, by breaking PC tendon, pre-stress will decrease and a load-proof performance will also decrease.

Therefore, the suitable measure against PC structure is made by presuming and predicting reduction of pre-stress. And the safety of PC structure is intentionally maintainable.

However, the technology that adequately measures of pre-stress of the PC structure under the present situation has not been established. Therefore, authors combined the stress relief technique and the full-field measurement method and developed the measuring method of the existing action stress.

Photo 1. PC Tendon fractured by chloride attack[1].

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Development of on-board image measurement system for actual running and application to wall surface surveys of structures

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ABSTRACT

Many inspections of linear structures such as rail-road tunnels and monorail track beams have been carried out by close visual inspections, but the surveys spend a lot of time, and weigh on the original repair schedule. As an alternative technique for these close visual inspections, a method using video cameras has been reported whereby video images of surfaces of structures are converted into still images to create developed images while the vehicle is in motion. However, this technique inevitably causes image quality degradation. In addition, it cannot synchronize several cameras, and takes time for post-processing. In contrast, industrial cameras used in imaging tests at manufacturing lines have smaller sizes and higher precision and frame rates, and can be connected directly to PCs.

However, to apply these industrial cameras to inspections of public structures, it is necessary to solve their adverse problems in photographing conditions (Table 1).

This time, the authors revealed the characteristics of the industrial cameras through laboratory experiments to solve these problems, and developed an "on-board image measurement system for actual running" that does not cause image quality degradation.

We verified the system on monorail track beams and at road tunnels, and confirmed that it can acquire clearer images than those acquired from conventional video cameras, and report the results in the post-processing in about a half period of time.

- **Acquisition of high-definition images**
  The authors compared the images acquired from this system with those acquired from conventional video cameras by verifying the system at monorail track beams and tunnels, and confirmed that the images acquired from the system were clearer.
- **Centralized management of a huge amount of images using time axes**
  The authors synchronized more than one CCD camera, and managed hundreds of thousands of high-resolution images acquired using time axes.
  We calculated the travel distance that changed successively using the optical encoder installed to the moving vehicle, had the images segmented automatically, and synthesized them.
- **Developed image creation system (image correction, segmentation, and connection)**
  The authors constructed a geometric correction system to create developed images that maintain uniform quality from images having different photographic resolutions, and created precise developed images of curved surfaces and cross sections.
  By changing the system configuration according to each subject, photographing keeping a designated quality is possible.
- **Extraction of cracks**
  There is still room for improvement in post-processing including crack extraction, hence we will continue to develop new systems.

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Table 1.

	Outside light conditions	Camera-to-subject distance	Travel speed	Number of equipment
Manufacturing line	Can be controlled to constant conditions	Almost constant	Can be controlled at a constant speed	Single camera
Infrastructure survey	Change between day and night, and depending on places	Distance changes depending on cross sections.	Travel speed changes.	More than one camera is used. Vibration-proof and small-size cameras are desirable.

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Development of a remote laser scanning system for continuous monitoring of cable-stayed bridges

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ABSTRACT

In Japan, continuous investments in infrastructures have formed a huge stock. However, since the infrastructures were constructed rapidly and emphatically in high economic growth period, the numbers of the degraded infrastructures increase rapidly. Therefore, the development of quantitative and efficient monitoring techniques is strongly required (Fujino et al. 2007).

This situation hold true for existing bridges. Moreover, according to geographical features of Japan, there are many long-span bridges such as the Tatara bridge, the Akashi Kaikyo bridge and so on. For these large structures, a large amount of labor is required for maintenance. The expensive cost of its inspection will become a big burden in the future.

In order to reduce the cost, vibration-based structural health monitoring is quite promising. In fact, tensile forces of cables in long-span bridges have been measured using vibrations. In vibration-based structural health monitoring, accelerometers have been usually utilized. They are attached to selected measurement positions of a structure and wired to other equipments such as amplifiers and PC. At long-span bridges, measurement works involve the installation of sensors at high ground levels. Therefore, it is strongly needed to reduce the dangerous works and improve their efficiency.

In this research, to improve the safety for work of vibration measurement in a long-span bridge, a non-contact and remote laser scanning system, which is the combination of a Laser Doppler vibrometer (LDV) and a total station (TS), was developed.

Although this system is simple because the LDV is simply attached on the TS, it makes possible to conduct automated remote vibration measurement continuously due to internal storage of the coordinates of measurement points and mechanical rotation of the LDV together with the TS. Furthermore, by using the rotating motor of the TS, the scanning angle of the system becomes ±90 degrees.

Measurement procedure using the developed system follows below.

1) Reflection tapes or prisms for surveying are attached on measurement points.
2) LDV laser beam is adjusted so that the reflection levels of the laser at each point attain the highest level.
3) TS measures three-dimensional coordinates of measurement points and stores their information.
4) Measurement starts. TS rotates to a memorized point, and then LDV conducts vibration measurement automatically.
5) When LDV measurement finishes, TS rotates to the next measurement point. 4) and 5) are repeated for all measurement points.

In this study, at first, the developed system was evaluated at selected bridges. Then, in order to examine the practicality of the system, it was applied into measuring vibration of the Tatara bridge. As a result, the developed system is expected to apply continuous vibration monitoring of the whole long-span bridges. By using this system, most of cables, girder, and pylon in the long-span bridge can be monitored quickly, easily and safely.

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3D profile measurement and buckling simulation on thin-walled cylindrical shells under compression by utilizing 3D-digital image correlation method

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ABSTRACT

Cylindrical shells are widely used in the most diverse branches of civil engineering technologies because of they combine light weight with high strength. However the buckling strength and post-buckling behavior prediction are extremely complex especially for thin-walled ones. Initial geometric imperfections are normally known to have a strong detrimental on the buckling strength and to be the reason of large discrepancy between experimental buckling loads and theoretical predictions. Moreover random imperfections in real shells are not known at the stage of design and may take a complex form. In order to investigate the effects of geometric imperfections on the buckling strength and post-buckling phenomena of thin-walled cylinders, these investigations were carried out both experimentally and numerically.

In present paper, a general and simple approach is proposed for modeling the geometric imperfection of thin-walled shell specimens by utilizing 3-D digital image correlation method (DICM), an optical non-contact and full-field measurement. Realistic geometric imperfection of thin-walled shell specimens were obtained and applied in numerical simulation, the thickness of specimens used in this experiment is 0.131 mm and the ratio of external diameter (R) to thickness (H) R/H is 250. Procedures of scanning on the geometric imperfection of thin-walled shell specimen and its application in numerical simulation are described detailed in this paper. The experimental investigation agrees well with the numerical simulation on both the strength and buckling behaviors. Therefore by using this effective approach the potential of strength capacity can be tapped in real thin-walled shell structures and it is demonstrated the visualization of buckling behaviors with initial random geometric imperfections by using this digital image correlation technique.

This investigation shows the sensitivity of buckling behaviors of thin-walled shell on the geometric imperfection clarify the quantized classification on the effect of geometric imperfection as follow figure.

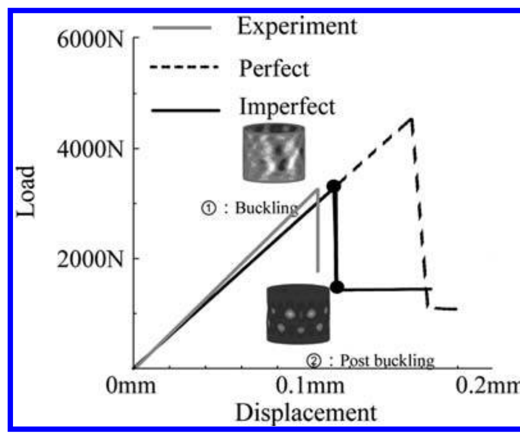

Figure 1. Load-displacement curve of specimen No.2.

This approach for the 3d profile measurement of thin-walled shell structure is an efficient and accurate method and would be conduced to the maintenance work of realistic thin-walled structures. By which, the realistic imperfection information can be employed to form the numerical mesh, therefore the simulation results agree very well with the experimental simulation. As to the realistic structures, combining this dialog and the 3d profile measure approach, the real imperfection information of structures can be obtained, thereafter by employing the numerical simulation; rationalization proposal can be obtained for the maintenance work of realistic thin-walled structures.

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Strain analysis method using multi-rosette analysis by digital image correlation method

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ABSTRACT

A new analysis method of strain distribution using the digital image correlation method (DICM) will be developed in order to estimate stress concentration and residual stress (Uchino, M. et al., 2009). We will present the relationship between the stress or strain concentration around the hole and the distance change of the hole diameter. As shown in figure 1, for the one-dimensional tension of an infinite thin plate with a through hole, the change ratio of the hole diameter along the x-axis equals the tangential strain at the border of the hole on the y-axis and then the change ratio of the hole diameter along the y-axis equals the tangential strain at the border of the hole on the x-axis. The changes of the hole diameter and the ring geometry around the hole were used for the new analysis method of stress concentration and residual stress. Specifically, the distance change between two points that is symmetry with respect to a center point of the hole will be measured by using the DICM. The stress distribution

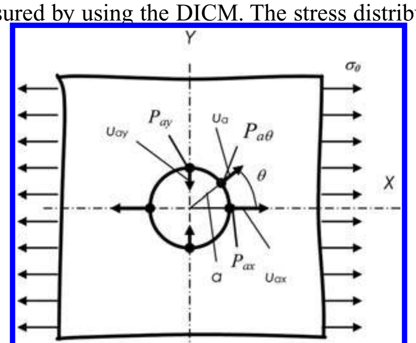

Figure 1. Definition of deformation state at the border of the through hole.

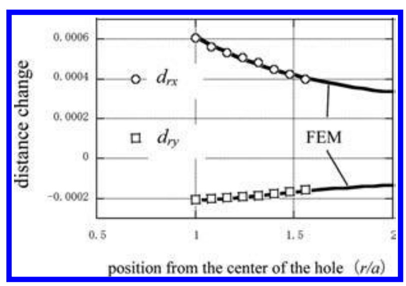

Figure 2. Results of tensile test of the aluminum plate with a 2 mm diameter hole in the stress of 15 MPa.

around the hole is estimated by this distance change (d_{rx}: along x-axis, d_{ry}: along y-axis). For an application of this method, the one dimensional tension experiments in the aluminium plate with the center hole and the stress release experiment in the concrete block by a core drilling method were performed to evaluate the proposed method. Here, the results of tensile test in the case of the center hole diameter was 2 mm using the multi rosette analysis at the tensile stress of about 15 MPa is shown in figure 2. The analysis result using the finite element method (FEM) is also shown in this figure. The experimental results (d_{rx}, d_{ry}) of the multi rosette analysis correspond with the analysis results of the FEM.

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Deflection measurement for bridges with frequency-shifted feedback laser

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ABSTRACT

Full-field optical measurement technology, which is the focus of the authors' efforts, is one field of Structural Health Monitoring technology. Full-field optical measurement technology is a technology which utilizes the properties of light to grasp information on objects of interest in 2- or 3-dimensions, and has several important advantages, including 1) remote/noncontact measurement, 2) collection of information on numerous points within a short period of time, and 3) visualization of that information. The frequency-shifted feedback laser (hereinafter, FSF laser) developed by Tohoku University, which is one such technology, is an ultra-high accuracy measurement technology (Fig.1), and because of its principle of measurement, it has the distinctive features that noncontact distance measurement is possible and measurement accuracy is not dependent on the measurement distance. Furthermore, because the maximum sampling frequency of the FSF laser is 1000 Hz, measurement of dynamic displacement behavior is possible. The authors have carried out various verification studies with the aim of applying the FSF laser to Structural Health Monitoring, and made improvements in the FSF laser at Tohoku University reflecting the results. This paper reports the results of verification of the distance and displacement measurement accuracy of the new FSF laser in long distance measurements, and also presents examples of application of the FSF laser to measurements of bridge deflection in service.

The results of verification of the distance and displacement measurement accuracy of the new FSF laser at measurement distance up to 1000 m are summarized in Table 1. And the results of deflection measurement for actual bridges are shown in Figure 2. Based on these results, it is possible to apply the FSF laser to actual bridges. In particular, the FSF laser is expected to demonstrate its effectiveness in measurements of long bridges and bridges with high piers.

Figure 1. The external appearance of the FSF Laser.

Table 1. Results of verification.

Measurement Distance	Standard Deviation σ (mm)	Measurement for 50 mm disp. (mm)	Deviation (mm)
50 m	0.128∼0.156	49.929	0.071
300 m	0.132∼0.158	49.976	0.024
500 m	0.137∼0.169	50.119	0.119
1000 m	0.149∼0.181	49.986	0.014

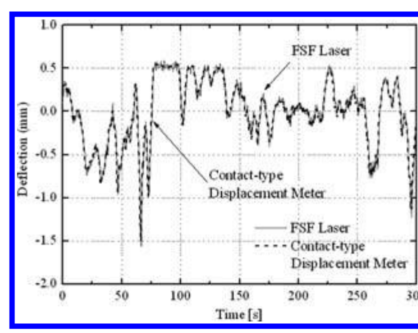

Figure 2. Result of deflection measurement (Truss Bridge).

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SS19: Implementation of bridge management administration in Japan
Organizers: H. Furuta & E. Watanabe

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Asset management of bridge structures in Osaka prefectural government

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1 INTRODUCTION

The Osaka Prefectural Government has been building an asset management system for bridge structures. This system attempts to maintain existing bridges in good conditions and extend the remaining lives by introducing the concept of Life-Cycle Cost (LCC). It is, then, necessary to achieve a rational plan of resource allocation through the normalization of annual budget. In this paper, the asset management system developed by the Osaka Prefectural Government is described.

2 PRESENT STATE OF BRIDGES IN OSAKA PREFECTURE

There are 811 bridges (more than 15 m span) in Osaka prefecture. The age of most bridges is 40 years, and it will become 60 years after 20 years. The ratio of bridges with more than 50 years will become 56% from 9%. Then, it is necessary to equalize the annual budget and change from the essentail maintenance to the preventive maintenance in order to establish a rational maintenance program under the limited amount of budget.

3 BRIDGE MANAGEMENT SYSTEM IN OSAKA PREFECTURE

This system attempts to maintain existing bridges in good conditions and extend the remaining lives by introducing the concept of Life-Cycle Cost (LCC). It is, then, necessary to achieve a rational plan of resource allocation through the normalization of annual budget.

4 CONCLUSIONS

In 1999, around 900 bridges with more than 15 m span length had been inspected in Osaka prefecture. Using the inspection data, the integrity level of each bridge was calculated, which was used to make the ordering of repair actions. In the determination of the repair order, the importance of bridges was taken into account as well as the damage states. The importance was given by the significance level of the road involving the bridge. The importance level was classified into three categories.

LCC calculation presents 3.5 million US dollars/year necessary for maintenance. Because of the present financial difficulty of Osaka prefecture, it is difficult to secure the budget for the maintenance. Thus, the necessary resource may be partly appropriated from other budget of repair works such as pavement, painting or other repair actions or may be managed to be secured through the more efficient utilization of road infrastructures.

The improvement of painting has started to extend the interval of repainting. Various actions have been performed to obtain the understanding of tax payers for the importanece of maintence works throught the site explanation meetings. The repair strategy and methods have been integrated and standardized to realize a rational allocation of financial and human resources.

The movement of asset management of bridge structures has spread all over Japan. Many local govermnents established a forum of asset management engineers, in which active discussions and experience and knowledge have been exchanged. The appeal for the necessity of preventive maintenance has started and the movement has grown up.

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Evaluation of maintenance cost estimation and feedback to the BMS

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ABSTRACT

The Aomori Bridge Management System (ABMS) has been developed in 2004 and 2005, and the Five Year Bridge Management Plan was established using ABMS.

The accuracy of maintenance cost estimation has been evaluated in order to feedback the results to ABMS. The accuracy of maintenance cost estimation is vital, because the Budgetary Plan which is the main part of Bridge Management Plan is solely dependent on the maintenance cost estimation. If the accuracy of maintenance cost estimation derived from BMS is poor, the Bridge Management Plan will lose dependability and it becomes difficult to keep PDCA management cycle of BMS rolling.

The investigation on the actual cost of the maintenance works against the estimated cost calculated by ABMS was made on 52 bridges whose maintenance works were completed in the year of 2007.

Among 52 bridges, there were 9 bridges which were not originally in the list of maintenance works but were included in the 5-year action plan because the seismic retro-fit was planned. In such cases, some maintenance works were conducted along with seismic retrofit. In other cases, the originally planned maintenance action was cancelled after detailed investigation. Therefore, cost of newly added maintenance works was subtracted from the actual cost, and the maintenance works which was planned but not conducted were also subtracted from the estimated cost. After making above mentioned modification, estimated cost and actual cost were compared.

As shown in Figure-1, there were 17 bridges whose actual cost was less than 50% of originally estimated cost. The main reason for the big decrease in cost was the change of maintenance measures. For example, original plan was to take off concrete with high concentration of chloride ion by water-jetting, but in the actual rehabilitation work the water-jetting was not used because the area of high concentration of chloride ion was limited. In other case, original plan was to replace the bridge bearings, but in the actual rehabilitation work, refreshing method was used to reduce long-term maintenance cost instead of replacing bridge bearings.

There were 5 bridges whose actual cost was more than 150% of estimated cost. Two of them were steel girder bridge, whose original maintenance measure, partial painting, was changed to whole painting because the old paint had PCB and it was necessary to take all PCB off.

Taking those two groups out, the remaining group of 30 bridges had actual cost with 50% to 150% of originally estimated cost. The group of 18 bridges, which is 35% of 52 bridges, had actual cost with 75 to 125% of estimated cost. The mean value of cost ratio (actual cost/estimated cost) was 74%.

Considering the fact that the cost estimation is based only on the visual inspection data from periodic inspection, the accuracy of cost estimation is in the satisfactory level.

After three years of using ABMS, we are satisfied with the accuracy of the LCC estimation, and we do not feel that we need to change various data including cost data at this moment. The feedback to ABMS will be made after the completion of 5-year bridge management plan.

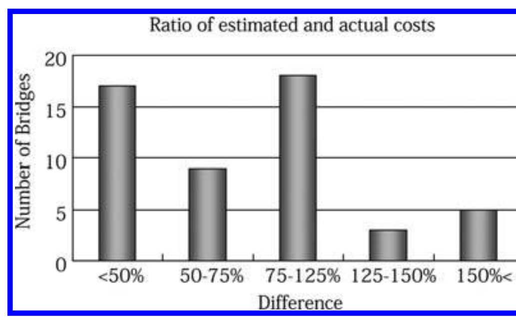

Figure 1. Ratio of estimated and actual costs.

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Risk based inspection strategy considering structural redundancy of long span bridges

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ABSTRACT

Generally, long-span bridges are large in scale and contain numerous inspection points. Most of them are also marine bridges with poor accessibility, which makes maintenance, especially inspection, more difficult.

Some of the existing inspection gantries on long-span bridges are approaching a time for costly renewal on which decisions need to be made in a very near future. For the purpose of rationalizing long-span bridge inspections, this study investigated how to determine optimum inspection intervals for long-span bridges utilizing risk assessment, with a focus placed on whether to use the inspection gantries or not. Figure 1 shows an example result of 60-year LCC analysis for a single-plane cable-stayed bridge in relation to the average expected risk obtained. Both LCC and risk were lower than the current inspection policy (inspection at 7-year intervals using inspection gantries) at the inspection intervals of 3 or 5 years with no-use of inspection gantries.

Since long-span bridges tend to have a complicated structure with high redundancies, it is not necessarily reasonable to weigh all members equally in inspections. In an attempt to identify locations and members to which higher inspection priority should be given, the authors carried out an entire bridge analysis using a network arch bridge model with corrosion caused to various members and joints. Figure 2 shows vertical displacements of the arch crown, respectively, with live load applied to the center of the span. As shown in this figure, when corrosion was caused in the arch springings ((1)-a) or in the entire arch rib ((1)-c), limits were exceeded and displacement rapidly increased at a remaining section rate of 40%. Members with higher risk of leading to overall deterioration were identified.

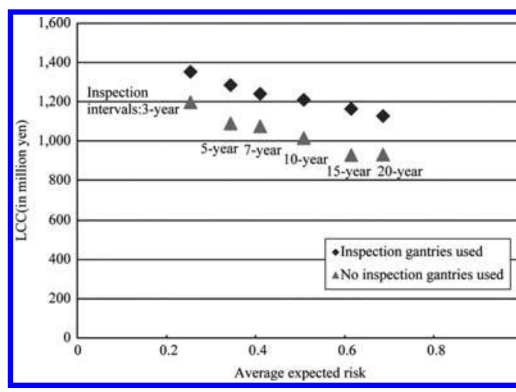

Figure 1. LCC vs. risk on a single-plane cable-stayed bridge.

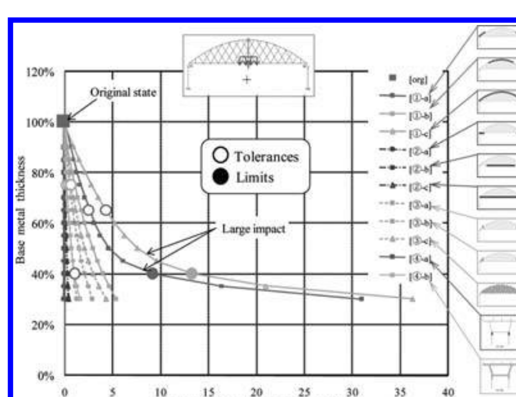

Figure 2. Arch crown displacements by member corrosion pattern.

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Approach for bridge management using BMS in West Nippon Expressway Company Limited

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ABSTRACT

Under the circumstance that expressway bridges should be deteriorating in the near future, the Nippon Expressway Company Limited (NEXCO) developed the bridge management system (NEXCO-BMS) in 2003 for the preventive bridge maintenance and the system has been applied to bridge management in NEXCO since then to keep the safety and security of the expressway. This system is expected to support the well-planned maintenance by determining bridge condition, predicting future deterioration, and selecting optimal timing and method for repair and/or reinforcement. Since the integrated maintenance database was built such as for inventory data and the inspection records and repair/improvement history evaluation data, we know the transition number of deteriorated bridges which need repair and/or reinforcement by using several kinds of prediction formula based on engineering knowledge. So far the function of this system has been partially improved for supporting the well plan and maintenance work enforcement of the actual project that suited the practical work in 2008. In this paper, outline of NEXCO-BMS and actual simulation case in West Nippon Expressway Company Limited are described.

By using improved NEXCO-BMS, several cases of simulation might be calculated accurately. The finding obtained from the simulation is as follows (Table 1).

– As for scenario of the corrective maintenance Case A, if calibration ratio is not high, same deterioration tendency might be occurred based on similar environmental factors, deterioration curves and so on.

Table 1. Simulation cases in West NEXCO.

Case	Case A	Case B
Service life (year)	45	23
Number of member	445	90
Start year of simulation	2009	
Term of simulation (year)	45	
Maintenance level	Grade 1 to 4	

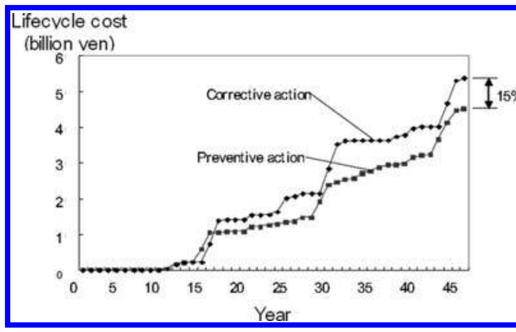

Figure 1. Lifecycle cost compared Preventive and Corrective actions (Case B).

Table 2. Scenarios of optimizing annual budget (Case A).

Items	Scenario 1	Scenario 2	Scenario 3
Budget limit per year	2 billion yen	3 billion yen	–
Lifecycle cost (45 years)	51.7 billion yen	48.8 billion yen	44.7 billion yen
Ratio of LCC	1.16	1.09	1.0
Ratio of project concentration*	1.7	2.8	15.6

*Ratio of maximum annual budget and average annual budget

– As for scenario of the preventive maintenance Case B, preventive actions is superior to corrective one in the view point of lifecycle cost, also calibration such as deterioration curve, deterioration mechanism is needed (Figure 1).
– Result when optimizing annual budget applied to Case A, total lifecycle cost is the most economical on available budget free case, nevertheless "Ratio of project concentration" is highest. It indicates that budgeting case increases both total lifecycle cost and ratio of deteriorated members because of changing strategic optimal planning; on the other hand, project concentration will be distributed. (Table 2).

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Preventive maintenance and technical development on long-span bridges

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ABSTRACT

This paper describes the preventive maintenance and the technical development on the Honshu-Shikoku Bridges (HSB) in order to inspect and maintain the long-span bridges in healthy conditions for more than 200 years as a long-term program, continuing and developing the bridge engineering in Japan.

The HSB, including 17 long-span bridges, were completed and opened to traffic by 1999 as the three permanent links between Honshu and Shikoku, i.e., two of four major islands in Japan. One of the most famous bridges is the Akashi Kaikyo Bridge, which was completed in 1998 with the world's longest span of 1991 meters. As the toll road project, the HSB were designed and constructed by the advanced bridge technologies, considering typhoons, earthquakes, tidal currents and corrosive environments.

For the preventive maintenance of long-span bridges, the dehumidification system for the main cable, called as the dry-air injection system (Figure 1), has been applied to all 10 suspension bridges since 1998. The dry-air injection system is operated effectively in order to enhance the durability of main cables. In addition, the coating management system is under way in order to reduce the life-cycle costs of steel structures. The coating thickness, especially the surface coat and the intermediate coat, is periodically checked at designated positions on the long-span bridges, evaluating the coating durability and determining the overall repainting.

In the technical development of long-span bridges, the automatic coating machine and the magnet-wheeled gondola (Figure 2) were developed for steel box girders and steel towers in order to inspect the coating durability and to repaint the deteriorated coats safely and efficiently. On the other hand, the vacuum-wheeled gondola was also developed for concrete structures in order to inspect the surface durability with no scaffolding. In addition, a non-destructive test, called as the electromagnetic flux method, was developed and applied to the investigation of existing suspenders on suspension bridges, checking the area reduction of suspenders and revising the maintenance program.

By continuing the preventive maintenance and technical development on long-span bridges, the HSB shall be operated and managed in sound conditions for a long time even though Japanese road policy will be changed in the future.

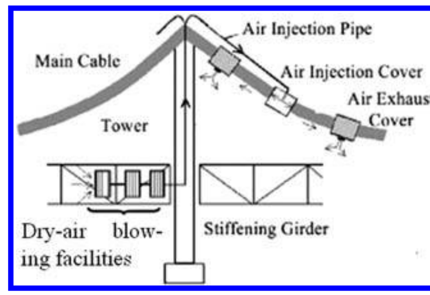

Figure 1. Dry-air injection system for Akashi Kaikyo Bridges.

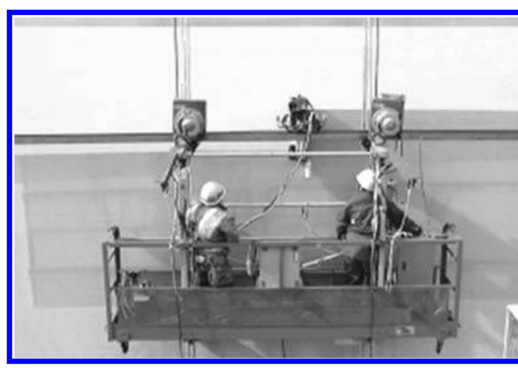

Figure 2. Automatic repainting on main tower, using automatic coating machine and magnet-wheeled gondola.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Risk evaluation and financial analysis for road maintenance on urban expressway based on H-BMS

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ABSTRACT

Hanshin Expressway Public Corporation had been privatized in 2005 and has responsibility for maintaining service level of civil structures at sound condition and completely paying off its debt within 45 years after privatization. Therefore, it is an important issue to establish maintenance and rehabilitation plan considering the payment of the debt.

This study addresses on Hanshin Expressway asset management system considering internal control and risk management system. Concretely, first of all, business process of Hanshin Expressway is constructed in terms of internal control and risk management.

On the other hand, uncertainty of deterioration process is treated as risk, and the relationship between this deterioration risk and maintenance/rehabilitation cost is verified. Based on the risk evaluation result, this study tries to calculate financial information which can be used to making debt payment plan.

Finally, for continuous improvement of the business process, extraction of the high priority monitored structures is discussed based on relative evaluation method of deterioration rate of each structure using benchmarking analysis.

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Study for bridge renewal and repair by Osaka municipal government

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ABSTRACT

The Osaka Municipal Office has been operating the Bridge Management System (OBMS) since 2005. OBMS executes the optimum planning for the preventive bridge maintenance through the periodical inspection, efficient data acquisition and the priority rule taking into the historic and civic importance. According to a trial calculation, the total budget in coming 30 years can be reduced by 50% if the preventive maintenance is executed on 100 old bridges by implementing the renewal of limited number of bridges, for example, only 10 bridges in comparison with the renewal of every one of these 100 old bridges.

Recently, Osaka City is in a serious financial situation. Although it is still desirable to build new bridges, renew old bridges or retrofit seismically vulnerable bridges, the cost reduction by means of new maintenance technologies is above all essential since the City of Osaka has a limited bridge budget.

Osaka Municipal Office is anxious to enthusiastically develop the project of preventive maintenance; it found out that the renewal or replacing old bridges should be kept minimum. Therefore, the countermeasure for the old bridges by the Osaka Municipal Office may be classified into the preventive maintenance for the majority of old bridges but only a limited number of renewal or replacement.

Osaka city had made the first selection of one hundred old bridges that have been in service for more than 70 years since before the World War II (excluding those bridges which are targets of special replacement or removal budget plan), to perform the detailed analysis through the "Replacement judgment matrix". Through this screening process, only a limited number of bridges were selected for the LCC analysis to see if they are worth being considered for preventive maintenance or the replacement (Yokota 2008).

It is a general practice to decide the maintenance plan based on the field inspection data and the health assessment of bridge taking into account the deterioration rate of the bridges. However, in view of the fact that the old bridges built before the Second World War were designed using different codes from now with respect to loads, earthquake-resistant design and river conditions and do not correspond to the present design codes. Thus, the judgment on the bridge replacement only on the basis of the bridge health assessment may be misleading. From this standpoint, the final decision was proposed by adding the functionality in addition to the health assessment.

In conclusion, as a result of the Replacement judgment matrix and the LCC analysis, 10 of bridges were selected for the replacement in addition to those bridges which were to be renewed that are budgeted by different budget plans.

This paper explains the basic concepts regarding the project on the replacement of old and historic bridge (OCPEWF, PWBOMG). Thanks to the well-preserved drawings, documents of summary of works and records of works on the bridges built before the Second World War, the project has been promoted quite smoothly. Thus, the effort and tradition of the bridge engineers of the Osaka Municipal Office must be highly respected. The importance of the succession of the important documents and information to the next generations is becoming more and more important.

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SS20: Challenges for enhancing bridge security

Organizer: S.R. Duwadi

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Recognizing and reducing vulnerabilities of transportation infrastructure

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ABSTRACT

Incorporating robustness and resiliency into highway infrastructure design has evolved over time through lessons learned from each hazard event. As a result of hurricanes, earthquakes, flooding and scour there have been a greater understanding of the impact of these on the infrastructure and its vulnerability. Solutions have been developed over years through research and development effort, although there are still challenges that remain. The understanding & designing for security however is relatively new, and therefore multitude of challenges lay ahead. Unlike natural hazards where effort has been spent over decades to understand behavior, effort to understand behavior of structures to explosive loadings or other security risks is relatively new and still evolving. The importance of recognizing and defining vulnerabilities to each hazard event is critical for developing solutions to keep these structures functional after any event. This paper will discuss the importance of recognizing vulnerabilities of highway bridges to natural and manmade hazards and present a summary of solutions that have been developed to mitigate these hazards.

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Bridge design – relevance and efficiency of protective measures for bridge structures under severe loading

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ABSTRACT

The research project SKRIBT (Protection of critical bridges and tunnels in the course of roads) is concerned with the development and demonstration of a comprehensive methodology for the evaluation of infrastructure subjected to severe natural and man-made hazards. The presented paper is focused on the resilience of bridge structures as a part of this general analysis and the relevance and efficiency of protective measures under severe loading conditions.

Starting with a thorough investigation of potential and available measures and an identification of relevant scenarios the extraordinary loading cases are integrated in the semi-probabilistic design approach provided in the current Eurocode standards. The subsequent analysis of those actions and structural resistance is demonstrated by means of a stay-cable bridge example.

Albeit the loss of a single cable has to be taken into account in regular bridge design neither the loss of a cable group nor the significant additional dynamical effects possibly caused by man-made hazard are covered in European standards. The presented analysis shows that this scenario does not necessarily lead to an abrupt failure or progressive collapse of the global structure as detailed analysis of maximized dynamical internal forces shows. The analysis is based on safety factors analog to the extraordinary load combination in Eurocode 1 and compared to the materials' yield strength. Against this background the components remain linear elastic. It can be shown that damage stays localized and only local damages caused by the explosion have to be reconstructed. The resulting bearing capacity limits the allowable traffic loads to 60% of the original loads until the bridge is fully restructured.

The described damages are classified in a five step categorization of damage and evaluated in a comprehensive approach together with all relevant affected aspects for the user and the environment.

Once the comprehensive evaluation system for the criticality and vulnerability is established, the variety of measures (see table 1) can be evaluated using the same methodology. Thus the procedure enables for the evaluation of the relevance and efficiency of protective measures for bridge structures under severe loading conditions.

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Mitigation of terrorist threats to structural components

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Permission to publish was granted by the Director, Geotechnical & Structures Laboratory, ERDC

ABSTRACT

Many significant structures, including buildings, bridges, tunnels, etc. are threatened by terrorist attack. There are unlimited possibilities as to the types of terrorist threats that could be brought against these structures. Based upon past history, the predominant threat of concern has been vehicular bombs, or using the current term, "vehicle-borne improvised explosive devices" (VBIEDs).

There are many approaches to terrorist threat mitigation, including the four "D's" that are commonly defined as: Deterrence, Detection, Denial, and physical Defeat. A complete mitigation plan for any structure will incorporate all four of these components. However, with a vehicle bomb threat, the first three D's are often of limited benefit in that many of our nation's critical structures, such as bridges and tunnels, demand unlimited vehicular access by their very nature, leaving Defeat, or more accurately, "hardening", as the only option.

Even worse, critical structural components are often directly adjacent to and within inches of high-traffic areas, making them extremely vulnerable to detonations from VBIEDs. Further compounding the problem, these structural components are often very weak in the direction of applied loading from a VBIED. For example, columns are predominantly designed for vertical axial loadings. Yet, a VBIED would generally detonate at such a location that its extreme pressure loadings will be applied laterally on the column, in its weakest structural direction.

Previous research and mitigation efforts for buildings (where most efforts were concentrated prior to 9–11) always assumed some reasonable degree of bomb standoff, and distance, even a small distance, is by far the most effective means of blast mitigation. Unfortunately, many structure owners do not have this luxury as enforcement of any reasonable standoff will essentially require closure of the asset. Thus, much more effort must be given to blast hardening of these critical structures and understanding the loading and response phenomena associated with these severe blast pressure environments very close to the explosive.

Currently, the engineering approaches to structural hardening against blast are very widespread, with some being sound and others being poor or improperly applied for the given situation. Much of the problem arises from inexperience in the arena of explosion effects and blast loading as many engineers are just being introduced to this unique field. As a result, there is much confusion and mis-information. It is the purpose of this paper to provide a basic overview of explosive blast mitigation techniques for basic structural components that are applicable to any structure type. Mitigation techniques are categorized and discussed in terms of the manner in which they affect the most basic law of motion, Newton's second law. Results from recent high explosive tests conducted by the authors are presented and used to show the effectiveness of the varied concepts.

SS21: Structural monitoring of bridges:
Hong Kong's experience
Organizers: Y.-L. Xu & M.C.H. Hui

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Structural health monitoring and safety evaluation of Stonecutters Bridge under the in-service condition

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ABSTRACT

A structural health monitoring and safety evaluation system (SHM&SES) is deployed by Highways Department (HyD) to monitor the structural health condition of Stonecutters Bridge under its in-service condition and to evaluate its structural safety when exceedance of monitoring criteria are identified in certain locations and/or components. The SHM&SES is composed of 4 integrated systems, namely, structural health monitoring system (SHMS), structural health rating system (SHRS), structural health evaluation system (SHES) and structural health data management system (SHDMS). This paper outlines the functional and architectural requirements of the SHMS, SHRS, SHES and SHDMS. The modification works done on previous SHMS in HyD for enhancing the operational efficiency of SHM&SES are also described.

A summary of structural health monitoring and safety evaluation for long-span bridges is outlined as follows:

(1) Codified performance limits – Structural health monitoring refers to the usage of on-structure instrumentation system to monitor the performance of the bridge structural system; whereas structural safety evaluation refers to the usage of analytical tools such as finite element software to evaluate the stability of the bridge structural system when the monitoring criteria are exceeded. The structural performance limits at the serviceability limit state should be used as the base-line of structural health monitoring.

(2) Performance limit for monitoring and evaluation – The performance of the bridge structural system to be monitored by the SHMS is devised in accordance with the structural performance limits defined by BS5400 at the serviceability limit state; whereas the structural safety to be evaluated by SHES is devised in accordance with the structural performance limits defined by BS5400 at the ultimate limit state.

(3) Codified requirements in monitoring and evaluation – Apart from structural health motoring and safety evaluation, the key functions of development and deployment of the SHMS&SES are: (a) to improve/enhance the current practice of bridge inspection and maintenance from local and subjective condition to global and objective condition; and (b) to provide data and information for updating/amending the contemporary bridge design manuals, standards and codes which are the standards for future works of: (i) design and construction of new bridges and (ii) maintenance and rehabilitation of old bridges.

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Structural health monitoring of Stonecutters Bridge under the construction stage

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ABSTRACT

The Stonecutters Bridge is a two-cable-plane cable-stayed bridge and carries dual-3 highway traffics. It is currently the world's second cable-stayed bridge with the main/central span length exceeding 1 km. In the bridge construction stage, Highways Department deployed a structural health monitoring system to investigate the variation of structural health conditions of the bridge under various stages of construction, with particular reference to the erection stages of steel deck segments at main/central span. This paper presents the monitoring works in six aspects: (i) objectives and scope of monitoring scope, (ii) monitoring equipment and facilities, (iii) monitoring base-lines, (iv) bridge geometry profiles monitoring, (v) stay cable force and damp-ing ratio monitoring and (vi) steel deck segment strain/stress monitoring.

Finally the monitoring works of Stonecutters Bridge under the construction stage are summarized as follows:

1. The as-built bridge geometry profile basically satisfies the upper limit as stated in the Construction Speci-fication for Stonecutters Bridge.
2. The stress monitoring in key locations of bridge showed that the strength limits had not been exceeded during the construction stage.
3. The stay cable force monitoring shows that 92.4% (207/224) of the stay cables are within ±10% of the analysis/design results.
4. The Scruton Number of most stay cables, basing on damping ratios measurement on selected samples (1 out of 4) of stay cables, are less than 10. This supports the necessity for installation of dampers in stay cables for reducing the potential risk of rain-wind-induced vibrations on stay cables.

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Thermal behaviors of Tsing Ma Suspension Bridge

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ABSTRACT

The Wind And Structural Health Monitoring System (WASHMS) for Tsing Ma Bridge has been operated since 1997. The WASHMS is devised to carry out the monitoring of four categories of parameters, namely, environmental status, traffic loads, bridge features, and bridge responses. This paper studies on temperature distribution of the Tsing Ma Suspension Bridge via both field monitoring and thermodynamic analysis.

With appropriate simplification, the entire large-scale bridge can be divided into several thermal fields and each component can be modeled with fine finite element models. With appropriate thermal boundary conditions, transient thermal analysis is conducted in the present study to predict the temperature distribution of the deck plate, bridge section, and bridge tower at different time and at different seasons. Variation of the temperature data at different time is obtained and compared with the measurement data from the WASHMS. The comparison shows a very good agreement between the model prediction and field measurement. Consequently the FE models are verified.

The numerical results provide thorough information regarding the temperature distribution of the bridge. The temperature data can be employed to calculate the thermal responses of the bridge, which can be used to evaluate the environmental effect on bridge's performance quantitatively.

The Tsing Ma Bridge with a total span of 2132 m is a suspension bridge as shown in Fig. 1.

Fig. 2 shows a clear and fairly stable cycle of monthly ambient temperature variation.

Fig. 3 illustrates the temperature distribution across the tower section at 13:00 pm. It is clearly shown that the structural temperature in the region near the surface is much higher than those inside the tower.

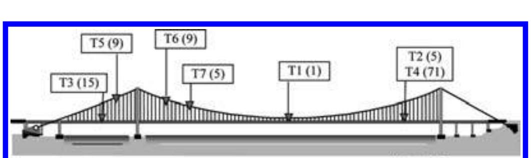

Figure 1. Position of temperature sensors in Tsing Ma Suspension Bridge.

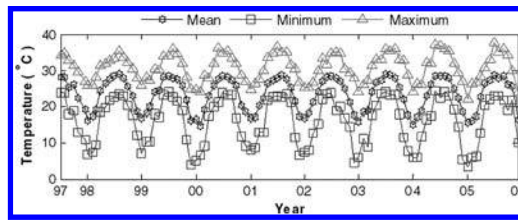

Figure 2. Monthly ambient temperature in 1997–2005 (T1).

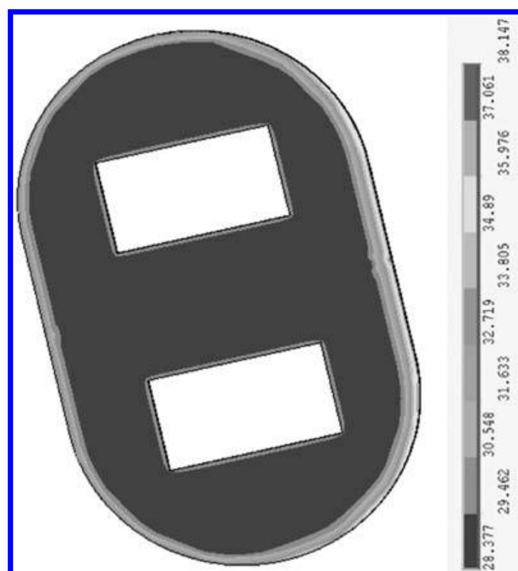

Figure 3. Temperature fields of tower section at 13:00 pm.

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SHMS-based bridge rating method for long span cable-supported bridges

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ABSTRACT

Long span cable-supported bridges begin to deteriorate once they are built and continuously accumulate damage during their service life due to natural hazard and harsh environment such as typhoons, earthquakes, vehicles, temperature and corrosion. To ensure the serviceability and safety of long span bridges, bridge rating systems are often adopted by bridge management authorities as guidance in determining the time intervals for inspection and the actions to be taken in the event of defects being identified. Most of the currently-used bridge rating methods are based partly on engineering analysis and partly on practical experience. Kushida et al. (1993) proposed a membership function to quantify the subjective uncertainty included in empirical knowledge on bridge rating. Aktan et al. (1996) integrated analytical and experimental research to assess global conditions and evaluate the serviceability and safety of bridges. Liang et al. (2001) set up an evaluation multiple layer fuzzy method for evaluating the damage stage of existing bridges. Sasmal et al. (2006) developed a systematic procedure and formulations for rating existing bridges using fuzzy mathematics.

On other hand, structural health monitoring technology gains a rapid development recently. Structural health monitoring systems (SHMS) have been designed and installed in a number of long span bridges to monitor their serviceability and safety. However, there is insufficient link between the bridge rating method and the SHMS to fulfill the common goals.

In this regard, this paper presents a SHMS-based bridge rating method for long span cable-supported bridges. The fuzzy based analytic hierarchy process (F-AHP) is first employed, and the hierarchical structure for synthetic rating of each structural component of the bridge is proposed. The criticality and vulnerability analyses are performed largely based on the field measurement data from the SHMS to offer relatively accurate condition evaluation of the bridge and to reduce uncertainties involved in the existing rating method. The procedures for determining relative weights and fuzzy synthetic ratings for both criticality and vulnerability are then suggested. The fuzzy synthetic decisions for inspection are made in consideration of the synthetic ratings of all structural components. The SHMS-based F-AHP bridge rating method is finally applied to the Tsing Ma suspension bridge in Hong Kong as a case study. The field measurement data recorded by the SHMS installed in the bridge for the past ten years are used to update the existing bridge rating system. The effects of different comparison matrices on the inspection decision are investigated. The results show that the effects of relative weights from different comparison matrices are small. For the bridge components concerned, the time intervals for inspection are either 1 year or 2 years. The results from the case study indicate that the proposed bridge rating method is feasible and can be used in practice for long span cable-supported bridges with SHMS.

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A method for stress concentration factor determination of welded steel bridge T-joints under moving load

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ABSTRACT

Fatigue is one of the most important failure modes of metallic components of aircraft, civil structures, and mechanical systems. Fatigue life prediction for structural components subject to oscillating stresses with constant amplitude is given by the well known Wöhler curve, also called *S-N* curve. In the real applications, however, the nominal stress approach is not suitable for structural joints if the object detail is complicated and incomparable to any classified joints, or the loadings are complex to make it difficult or impossible to determine the nominal stress. An alternative method for the fatigue analysis of complicated welded steel joints is the hot spot stress approach which is more accurate and reliable than the nominal stress approach.

When using the hot spot stress method for fatigue design and fatigue life prediction of welded structures, a critical issue is focusing on how to determine the stress concentration factor (SCF) for the welded details. In general, the determination of SCF can be achieved by means of finite element analyses, laboratory experiments, or field tests. Research efforts have been devoted to investigating the SCF determination (Karamanos et al. 2000, Fung et al. 2002, Gho et al. 2003, Chan et al. 2005, Gao et al. 2007), and most of them are related on the welded tubular joints and seldom are devoted to investigating the SCF properties for the welded plate joints of large-scale civil engineering structures, especially for the cable-supported steel bridges. Furthermore, because of the considerable effects and sources of uncertainties during the determination of SCF either by finite element analyses or by experimental measurements, an investigation into the stochastic properties of SCF is of vital necessity.

The purpose of this research is to seek a method for determining the SCF and its stochastic characteristics for a typical welded steel bridge T-joint composed of two perpendicular steel plates, which is perceived by conducting full-scale model experiments of a railway beam section of the suspension Tsing Ma Bridge (TMB). The strain data of the pre-allocated measuring points are acquired and the hot spot strain at the weld toe is determined by a linear regression method. The SCF is then calculated as the ratio between the hot spot strain and the nominal strain which is derived from the measured data from desired strain gauge. To take full account of the effect of predominant factors on the scatter of SCF, the experiments are carried out under different moving load conditions. The statistical properties and probability distribution of SCF are achieved, which reveal that SCF for the welded steel bridge T-joint conforms to a normal distribution. The proposed method will benefit the fatigue life and reliability assessment using measurement strain data from a long-term structural health monitoring (SHM) system since the sensors for strain measurement are usually not installed at the most critical locations due to the limitations of implementation technology and specific field conditions.

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Long-term performance of structural health monitoring system in bridges

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ABSTRACT

As a cutting-edge technology for infrastructural hazard mitigation, structural health monitoring systems (SHMS) have gained worldwide acceptance which is witnessed by a number of long-term SHMS installed on those recently-built long-span bridges over the past two decades. Being a basic component in SHMS, sensor systems play an important role in data acquisition and transmission. Although SHMS systems are generally intended for long-term service, very rare attention has been paid to the long-term performance of sensor systems. It has been observed recently that the lifetime of many installed sensors may not be as long as service life of bridge structures, which requires proper inspection, maintenance or complete replacement of sensor systems during the lifetime of bridges. The traditional optimal design approaches based on initial performance and cost may be greatly compromised by neglecting these factors.

In light of this limitation, this study focuses on the "health" of SHMS in bridges. To assess the functionality of sensor systems, a performance index is defined based on the observation errors of modal response. Using Monte Carlo simulation, the time-variant performance index of sensor systems can be obtained based on the lognormal life distribution of individual sensors. The optimal placement of a vibration sensor system composed 100 sensors is first conducted for Tsing Ma Bridge located in Hong Kong, and such a system is employed in the case study of long-performance sensor system performance. The simulation results clearly demonstrate the deterioration trend of the sensor system performance. A performance index threshold is adopted as the failure criterion for the sensor system. Subsequently, two important characteristics, the median-time-to-fail (MTTF) and the average maintenance period (AMP), are discussed in detail. The effects of two key variables, the dispersion in sensor life and failure threshold of performance index, are also examined within the procedure through parametric studies. The MTTF and AMP of the sensor system are generally shorter than the median life of individual sensors, and both are greatly affected by the prescribed performance threshold in failure criterion. It is also found that a large dispersion of sensor life adversely affects the median life and the reliability of the sensor system with no maintenance actions.

The discussion of the MTTF and AMP of the sensor system will shed light on more rational design strategy of SHMS in future—an optimal design method of sensor systems in terms of their life-cycle cost instead of the initial cost.

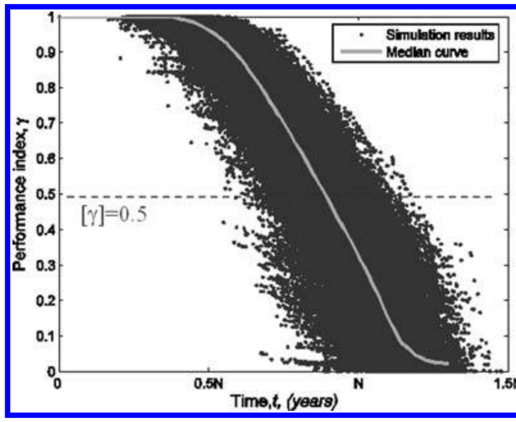

Figure 1. Time-variant performance index ($\zeta = 0.5$).

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SS23: Performance-based design for steel structures
Organizers: S.-H. Kim & J.-S. Kong

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Determination of load actions for performance-based design

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1 INTRODUCTION

Recently, performance-based design has gained attention as a design method that can provide more rational structural designs. In the performance-based design, it is possible to allow any design as long as it satisfies the requirements of performance. This means that designers can try to produce new innovative structures under more flexible environments. Needless to say, it is necessary for practical design cases to solve such issues as how to prescribe, guarantee, and evaluate the required performance, and prove the rationality and optimality of the design obtained.

In establishing process of performance-based design, design loads have not been sufficiently discussed from the viewpoint of performance-based design. Task committee on Actions on Civil Engineering Structures for Performance-Based Design Codes (Guideline for Code Writers), Japan Society of Civil Engineers has completed the report on load actions for performance-based structural design, which consists of two parts; Part I is related to the objectives, framework, classification of load actions, and technical terms, and Part II describes the detail explanation of various load actions. In this paper, the outline of its content will be introduced and some problems will be discussed for its practical use.

2 PART I – GENERAL FRAMEWORK

In Part I, general framework of actions is described. In Chapter 1 the objective of the guideline is made clear, and Chapter 2 describes basic policy which covers description principle, description scope, and description policy. Chapter 3 is dedicated to application range and past treatment in the past design codes. In Chapter 4, framework of actions is given by covering fundamental condition of design, action factor, and effect of action. Chapter 5 is the analysis of actions and their combination, and Chapter 6 is associated with the classification of actions, and the definition of terminology is given in Chapter 7. Part I has five appendices, first of which is the combination of load actions and the second is uncertainties involved in load actions and design loads. The third one is associated with the establishment of action model based on statistical approachs. The fourth and fifth are accidental load actions and the present status of actions in some international design codes.

3 CONCLUSIONS

The outline of the content of the guideline for Actions on Civil Engineering Structures for Performance Based Design Codes (Guideline for Code Writers) published in March, 2008 from Japan Society of Civil Engineers was summarized in this paper. In the establishment of performance-based design, design loads have not been sufficiently discussed from the viewpoint of performance-based design. A new concept of "Action" was introduced to determine the design loads, which can be available for the design load modeling.

It is believed that this guideline could contribute to the achievement of performance-based design. Furthermore, the guideline could provide useful information for various structures such as steel, concrete, composite and geotechnical structures, in order to make a common consensus for their mutual understanding. In addition, code writers can refer some references and accumulated data to specify the loads.

The guideline was completed by the joint task committee for guideline of design load for civil engineering structures. The authors express their sincere gratitude for all committee members for their great contributions.

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A study on vibration performance estimation of footbridge using human body model

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ABSTRACT

In recent decades there has been a trend towards to improve mechanical characteristics of materials used in footbridge construction. Engineers are now designing lighter, more slender and more aesthetic structures. As a result of these construction trends however, many of these footbridges are becoming more susceptible to vibration when subjected to dynamic loads. In most cases the vibrations of footbridges lead to serviceability problems, i.e. the inconvenience of the pedestrians or in some extreme cases a bridge may no longer be used and has to be closed. The human-induced dynamic loading occurs frequently and it is often regarded as dominant load for footbridges because it sometimes obstructs pedestrians to walk conveniently. Therefore, a three dimensional analysis program is developed in this study using the human body model. This program can effectively generate pedestrian excitation for the dynamic behavior of a footbridge under human-induced load.

A new human body model is proposed in this study. It is comparatively easier to consider these effects in the human body model. Also, it has advantages to apply complicated loading conditions such as loads from crowded pedestrians.

Therefore, dynamic analysis is performed for the crowded pedestrians using human body model. Three different walking models are compared, namely, the randomly walking human body model (human body model R), the synchronized model (human body model S) and the classical randomly walking model (classical random walking model $\sqrt{n}$) which was proposed by the Matsumoto et al. (1978).

Pedestrian density is set to be 1.5 pedestrians/m^2 as it represents crowded walking (Schlaich, 2002). Figure 1 indicates the example of responses from human body model R. As shown in table 1, larger displacement and acceleration responses can be found in both human body model R and S compared to the classical random walking model $\sqrt{n}$. Such results indicate that the human-structure interaction force and the inertial forces of human masses somehow affected the responses whole bridge.

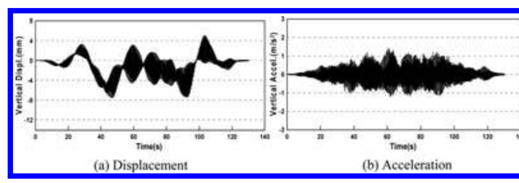

Figure 1. Time histories of response from human body model R.

Table 1. Maximum responses for each analysis model.

	Human body model S	Human body model R	Classical random walking model $\sqrt{n}$
Maximum displacement (mm)	18.249	12.81	4.87
Maximum acceleration (m/s^2)	3.70	1.84	1.00

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I-girder with discrete torsional bracings: Lateral-torsional buckling and torsional vibration

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ABSTRACT

The lateral-torsional buckling (LTB) strength of an I-girder can be increased by using an adequate bracing system. Torsional bracing can be divided into two categories: continuous and discrete torsional bracing. For the LTB strength of beams with continuous torsional bracing, Taylor & Ojalvo (1966) developed the exact solution for the critical moment of beams. Their solution is adapted for discrete torsional bracing by summing the stiffness of each bracing along the span and dividing by the beam length to obtain an equivalent continuous bracing stiffness (Trahair 1993, Yura 2001). Trahair (1993) proposed a stiffness requirement for beams with a mid-span torsional restraint based on a numerical approximation. Valentino & Trahair (1998) performed several solutions for beams with midspan torsional bracing under various loading conditions. Thus, it seems that a significant amount of study is still required for torsionally braced beams, especially for those with multiple bracing points.

The analytical solutions for LTB strength and the torsional stiffness requirement are successfully developed (Nguyen et al. 2009). This paper presents equations to determine the critical moment, torsional natural frequency and stiffness requirements of I-girders with discrete torsional bracings under uniform bending moment. For the I-girders subjected to linear moment gradient, the equivalent moment factor C_b in terms of the number of bracing points is proposed. The critical moment and the torsional natural frequency are derived for the arbitrary number of bracing points using Rayleigh-Ritz method (Chen & Lui 1987, Bazant & Cedolin 1991) and Lagrange's equation (Meirovitch 1991), respectively.

The proposed equations are then compared with the results of finite element analyses using ABAQUS (Hibbit, Karson & Sorensen 2001) and those obtained by previous researchers (Trahair 1993, Yura 2001). From the results, it is found that the proposed equations show good correlation with finite element analyses regardless of the number of bracing points and are successfully verified. Finally, a reduced formula for the total stiffness requirement is proposed for the design purpose.

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Reliability evaluation of RC piers of highway bridge using time series multi-state system approach

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1 INTRODUCTION

The reliability-based design method has been introduced as a design methodology of structures in Japan after the Hanshin-Awaji Big Earthquake Disaster. In the conventional reliability-based design method, however, a structure has only two performance states whether normally operating or completely break down. In a lot of systems, the change of state happens with not only two states but also multiple states including a partial failure according to the passage of times. The system that considers the multiple states changing with time is known as a time series MSS (Multi-State System) (Lisnianski, A & Levitin,G. 2003). If such a time series MSS is introduced in evaluation of structural reliability, we may consider the change of multiple performance levels by the occurrence of crack, progress of corrosion, deterioration of strength, so on.

In this study, the reliability evaluation of with multiple performance levels is performed using a time series MSS approach. Concretely, multiple performance states of piers due to salt damage such as the occurrence of crack of concrete, progress of corrosion of reinforcement, deterioration of carrying capacity of member are modeled by the time series MSS, and then the reliability of piers subjected to seismic load is evaluated. Further, their results are compared with the results obtained by a conventional reliability method with considering only binary performance states such as normally operating or completely break down. The advantage of the reliability evaluation using a time series MSS approach is verified.

The time series MSS approach makes it possible to evaluate the reliability of RC bridge pier RC bridge pier considering a cycle of performance deterioration such as initial state, corrosion initiation, craze initiation and lost carrying capacity. The conventional reliability evaluation method cans only two performance deterioration such as initial state and lost carrying capacity state. Also, as the MSS approach has three reliability indices such as availability, expected performance and performance deficiency, diversity evaluation of reliability can be possible. These merits take advantage for maintenance problem of structures.

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Reliability based design optimization of steel box girder bridge under corrosion attack

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ABSTRACT

Recently, it is reported that the construction cost of steel bridge lower than concrete bridge cost. And the steel bridge structures are developed rapidly in the world. This trend created a motivation for finding the optimal solution which minimizes the total construction cost and maximizes the structural performance. Reliability-based design optimization (RBDO) is considered as the integrated analysis of reliability analysis and optimization algorithms. Total cost of bridge is expected to minimize under predefined target reliability index that related to uncertainties and RBDO can be satisfied these conditions. During the service lifetime, uniform corrosion can occur on the surface of steel box girder bridge and lead to the structural strength reduction and damage (Czarnecki & Nowak, 2008). This paper presents a method to solve RBDO of steel box girder bridges using a computer program which integrates the Matlab optimization toolbox and a reliability analysis subroutine. Moreover, the effect of environmental agents is considered in terms of modeling the uniform corrosion phenomena which occur on the box surface during the structural lifetime.

An simply supported PC box girder bridge is considered with span length L = 40–70 m, subjected to self-weight dead load and truck live load HL93 follows AASHTO LRFD code. The results of design variables are shown in Table 1 which X_1, X_2, X_3 are thickness of top, bottom flange and the thickness of web respectively, X_4 is the depth of web. The total costs with respect to span length from 40 m to 70 m are plotted versus the allowable reliability index for ultimate strength in the range from 3.5 to 5 in Figure 1. Considering this Figure, the optimal cost steadily increased according to the span lengths and target reliability indices. It can be seen that this results is appropriated with the target reliability index that suggested by AASHTO code. Moreover, the proposed method to solve RBDO of PC box girder bridge by using optimization toolbox and code development is simple, flexible, practical and appropriate with the bridge engineering design and other field.

Table 1. Results of optimum design variables.

Design variables	L = 40 m	L = 50 m	L = 60 m	L = 70 m
X_1	18.10	18.23	18.40	18.68
X_2	25.15	28.28	31.26	34.14
X_3	18.24	18.35	18.52	18.77
X_4	1704.3	2113.5	2537.7	2976.5
Total cost	390.96	436.7	483.22	530.6

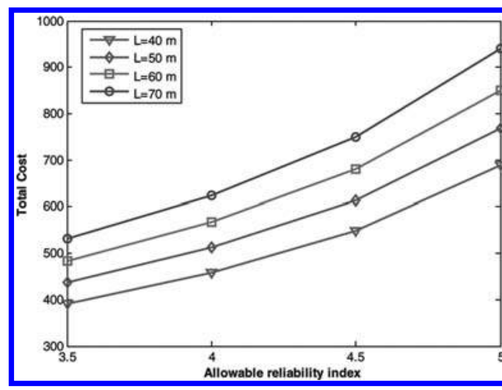

Figure 1. Optimum total cost versus reliability index for ultimate and span length.

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Performance-based durability design of weathering steel bridge

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ABSTRACT

In the recent climate of the budge cut for infrastructure investment, the weathering steel bridge is getting popular in Japan, and in fiscal 2006, the weathering steel bridge accounts for over 30 mass% of steel consumed for the construction of steel bridges. However, the development of abnormal rust in the weathering steel bridge is reported occasionally. The understanding of the durability design of the weathering steel bridge among bridge designers and bridge owners does not seem adequate, and the durability design may need be tuned up as well.

For the durability design of the weathering steel bridge, it is essential to see if the atmospheric environment in which to construct the bridge is not very corrosive. To this end, the study based on the long-term exposure test has indicated that such a mild environment is defined by the value of the air-born-salt-deposition rate less than 0.05 mdd (Public Work Research Institute 1993) and has further specified the areas of those environments where the weathering steel is applicable. To that end, Japan is divided into 5 zones: for each zone, the distance from the sea is specified individually, beyond which the unpainted JIS-SMA weathering steel highway bridge can be constructed without conducting the investigation of the air-born-salt-deposition rate. Design Specification for Highway Bridges in Japan (Japan Road Association 2002) has employed the distance requirement from the sea in the commentary. The distance requirement appears satisfactory, yet there is room for improvement.

A problem of the distance requirement is its sole dependence on the air-born salt while it is not the only factor that influences the rust development. Taking the other influential factors into consideration, more sophisticated corrosion-prediction methods have been proposed in the JSSC activity.

The corrosion loss of steel can be computed if two coefficients, the first-year corrosion loss A (mm) and the index of corrosion loss rate diminution B, are given. The JSSC activity has provided formulas of evaluating B from A so that the corrosion loss can be estimated once A is obtained. The evaluation of A is then the key to the assessment of atmospheric corrosiveness.

Two formulas have been proposed for evaluating A. The variables of those formulas include not only the effect of the air-born salt but also the other influential factors such as wetness and temperature. But those meteorological data at the specific construction site are not always available: the data at a nearby meteorological observatory may have to be used instead.

Alternatively, a simple means of evaluating A at the construction site has been proposed. The method has been termed the "button test".

The button test is essentially an exposure test, using button-shaped weathering steel test specimens. After one-year exposure, the weight losses of the test specimens will be measured to yield the site-specific value of A. The cost for conducting the button test could be even lower than that of the air-born-salt-deposition-rate measurement.

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SS24: Steel bridge rehabilitation
Organizer: M. Sakano

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Rehabilitative design/build of a railroad bridge

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ABSTRACT

The Roebling Steel Mill located in Florence Township, New Jersey is part of an EPA Superfund site and is in the mist of decontamination operations. However, a creek bisects the property and the only viable access to the other side is an abandoned railroad bridge. In October 2008, W.J. Castle, P.E. & Associates, P.C. (CASTLE) was retained to perform an in-depth inspection of the bridge to determine the overall conditions. The EPA wanted to convert the railroad bridge into a roadway bridge able to support vehicles up to 110,000 lbs. Our analysis of the site determined that the bridge could be utilized in the proposed design with minor repairs required to the components.

CASTLE was awarded the contract for the design/build of the bridge. Upon review of the proposed cost of rehabilitation versus the cost of rebuilding from scratch, it was determined that repairing the existing bridge would significantly save both time and money. Our initial inspection revealed several small voids or spalls on the abutments above water and one large void below water. Also, the existing steel superstructure exhibited areas of extensive deterioration and corrosion.

Using both our engineering and construction personnel, we ensured that the designs for the repairs would not only exceed all safety factors and capacities but would be easily implemented in the field.

In the original configuration, the six steel stringers were closely spaced at the center of the abutments, however the client required a minimum horizontal clearance of 13′–6″ to accommodate their vehicles. This change in layout prohibited the reuse of the existing diaphragms and bearing plates. While the bridge design was being finalized, the field crew was removing the steel superstructure to begin cleaning and repairing of the steel.

New steel bent plates were specially designed and fabricated to reinforce the deteriorated ends of the steel stringers. The voids and/or spalls above water in the concrete abutments were filled with standard epoxy concrete while the void below water was first covered with composite material before concrete was poured to fill the voided area.

For both speed and ease of construction, the guiderail posts and all connection hardware were attached to the fascia beams prior to be lifted in place. Also the channel diaphragms were attached to one beam and once the second beam was in place, was connected to both preventing any potential rolling or displacement of the beam.

A full length bearing plate anchored into the concrete abutments was utilized due to the minimal maneuverability. Also, portions of the cheekwalls were removed to accommodate the new wider spacing of the beams.

All existing steel components were painted with black epoxy paint except the galvanized repair plates and guiderails. Construction of the reinforced concrete deck was completed the first week in January and the bridge was opened by January 23, 2009.

The total cost of the Project including underwater inspection with repair to the abutment was $170,000.00 and the Project was completed within 3 months.

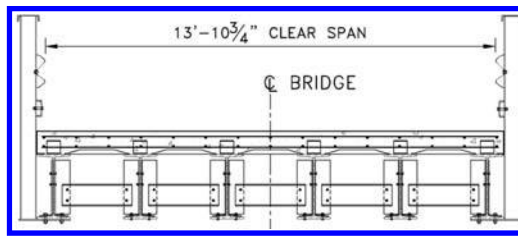

Figure 1. Typical section.

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Strengthening design for two-span steel-concrete composite bridges strengthened by external tendons

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ABSTRACT

In general, existing bridge strengthening as an alternative to complete replacement or construct a new one can provide an effective and economic solution (Conner et al., 2005). The post-tensioning with external tendons has been considered an effective method of strengthening or rehabilitating existing bridges (Troisky, 1989). The advantages of this technique are to enlarge the elastic range of bridge behavior, to increase the ultimate load capacity of bridges, and to improve the fatigue and fracture strength of bridge components (Saadatmanesh et al. 1989a; 1989b; 1989c). In addition, this technique is easy to perform and convenient to maintain because the tendons are exposed outside of bridges. For this reason, the post-tensioning with external tendons has been widely applied to various types of bridges as a means of strengthening existing bridges (Harajli 1993, Ng 2003).

The main objective of this paper is to propose a new rating equation considering the increment of the tendon force due to live loads of bridges in order to determine optimum numbers of strands in external tendons and the initial tendon force. The post-tensioning with external tendons is used for strengthening of steel-concrete composite bridges. Analytic expressions considering the increment of the initial tendon force are derived using the virtual work principle for configurations of straight and draped tendons under external loads. Based on these analytical expressions, a new rating factor equation is introduced considering the initial tendon force and its increment under external loads. A systematic procedure is illustrated to determine the number of strands in external tendons and the initial tendon force using the proposed rating equation. A design example is also given to demonstrate the effect of the proposed equation on increasing the load-carrying capacity of existing steel-concrete composite bridges. Eq. (1) and Eq. (2) represent a new rating factor and initial tendon force proposed in this paper. Eq. (3) represents the number of required strand (N_t).

$$RF = \frac{f_a - (f_{DL} - f_T)}{(f_{LL} - f_{\Delta T})(1+i)} \tag{1}$$

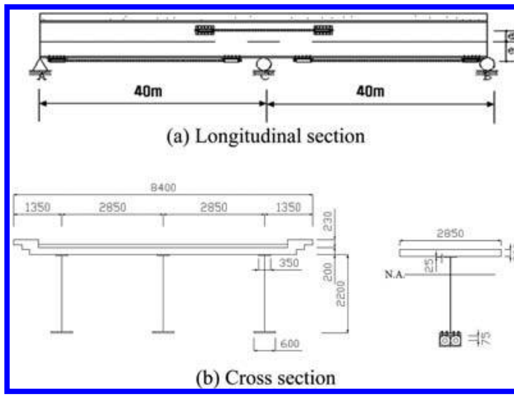

Figure 1. A plate girder bridge strengthened with external tendons (unit: mm).

$$T = \frac{f_a - f_{DL} - RF \cdot f_{LL}(1+i)}{\left(\frac{1}{A_{cp}} + \frac{e_{cp}}{I_{cp}} y_{sb}\right)} - RF \cdot \Delta T(1+i) \tag{2}$$

$$N_t = \frac{T}{\phi_t F_u} \tag{3}$$

Figure 1 shows longitudinal and cross section of design example for plate girder bridge strengthened with external tendons.

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Assessment and corrosion performance enhancement of weathering steel highway structures

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ABSTRACT

Herein, three studies undertaken to investigate several issues related to the assessment and corrosion performance enhancement of weathering steel highway structures are presented. The first examines the effects of corrosion product and local pitting on plate thickness measurements obtained using a standard ultrasonic gauge. The second evaluates the potential of metallizing, as a means for protecting elements of existing corroded structures. The third study comprises a series of reliability analyses of weathering steel composite box girder overpasses with bottom flanges corroding uniformly at various rates. Several of these analyses simulate plate thickness measurement or metallizing application events occurring part way through the service life.

In Figure 1, mass gain/loss results are presented for uncoated and metallized weathering steel specimens subjected to laboratory corrosion testing according to the SAE J2334 regimen (SAE 2002). It is estimated that these tests simulated at least 15 years of service in a marine environment for the uncoated specimens and 40 years for the metalized specimens. Base on results such as this, it is concluded that metallizing is an effective way of improving the corrosion performance of weathering steel structures.

Figure 2 shows the results of a structural reliability analysis, performed using limit state functions based on (CSA 2006), wherein the bottom flange thickness of a 40 m span, simply-supported box girder overpass is reduced over time using an assumed corrosion model for urban service conditions. Based on analysis results such as this, it is concluded that the structural reliability over time is highly dependent on the assumed corrosion rate. This assumed rate has a very high level of uncertainty associated with it, making in-situ thickness measurement a vital part of any structural assessment.

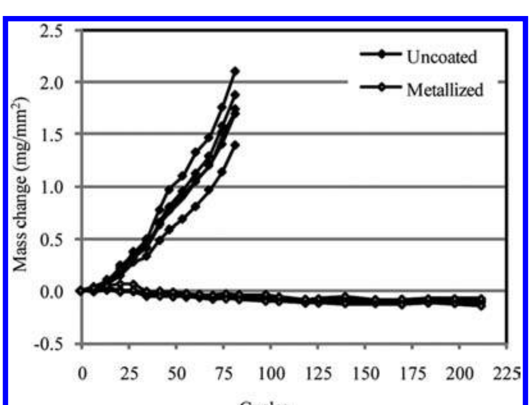

Figure 1. Weathering steel corrosion test results.

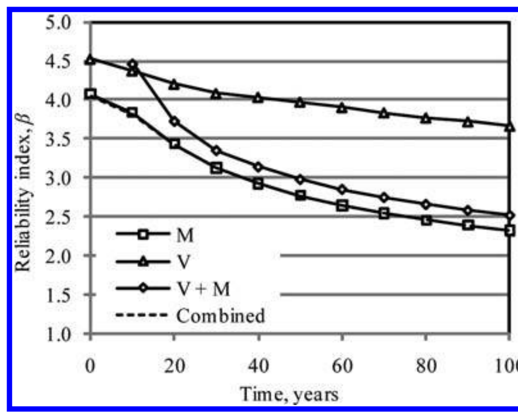

Figure 2. Reliability analysis results for urban corrosion.

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Analytical study of fatigue repair of coped beams using carbon fibre reinforced polymers

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ABSTRACT

Fatigue and fracture behaviour are important considerations in determining the condition of steel bridge girders. The existing microscopic cracks in girders and stringers can propagate into large cracks due to repeated loading. The presence of large cracks will reduce the load-carrying capacity of the girders. If left unattended, unstable crack growth can occur, causing catastrophic failure of the structure.

A fibre reinforced polymer (FRP) crack bridging repair eliminates fastener holes or weld stress risers that are produced using traditional repair methods. The FRP repair also has good fatigue resistance, high strength-to-weight ratio, good durability, corrosion resistance, and formability to complex contours.

This paper presents an analytical study of the effect of an FRP repair on the stress intensity factor at the crack tip, and it's applicability to use on bridges.

A finite element analysis study was carried out to determine the effect of an FRP repair on the stress intensity factor at the crack tip of a fatigue crack. For this study, a three layer technique developed by Naboulsi and Mall (1996).

When compared to the unrepaired specimens, the average reduction of stress intensity factor (SIF) on the patched side was about 61% for specimens with effective stress ratio ($ETR = E_{frp}t_{frp}/E_s t_s$) of 0.16 and 53% for specimens with ETR of 0.107. On the unpatched side, SIF showed a maximum increase of 14%.

An analytical case study was undertaken to evaluate the effectiveness of an FRP patch applied to a fatigue crack in the web of a coped beam. A coped beam has a complex combination of bending and shear stresses at the cope location. The specimens tested by Yam and Cheng (1988) were modelled using the SIF data from the finite element study.

Three parameters were considered. These were: stress range ($\Delta\sigma$), ETR, and orientation of FRP repair. The parameters were chosen to simulate field conditions. They take into account bridge loadings and accessibility of the repaired members.

The linear elastic fracture mechanics was employed to predict the increase in fatigue life of a repaired member based on the repair based change in stress intensity factor at the crack tip.

Table 1 shows the results of the analytical study. The percentage increase in fatigue life was identical for the high and low stress ranges. The study showed that a longitudinal repair is significantly less effective than a perpendicular repair. Provided there is clearance to apply the repair perpendicular to the fatigue crack, FRP repairs show to be a very effective means of extending the fatigue life of an aging structure.

Table 1. Results of case study.

Case	Repair	σ_n	ETR	ΔSIF	Fatigue Life	% Increase
NH	None	80	N/A	1	86,200	–
NL	None	40	N/A	1	689,500	–
SHL1	Long.	80	0.132	0.93	108,100	25%
SHL2	Long.	80	0.264	0.90	117,600	36%
SHA1	Angled	80	0.132	0.78	179,200	108%
SHA2	Angled	80	0.264	0.71	244,000	183%
SLL1	Long.	40	0.132	0.93	864,800	25%
SLL2	Long.	40	0.264	0.90	941,000	36%
SLA1	Angled	40	0.132	0.78	1,433,800	108%
SLA2	Angled	40	0.264	0.71	1,952,000	183%

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CFRP repair of fatigue cracks and bonding behavior subjected to cyclic load during curing

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ABSTRACT

Recently, many fatigue damages have occurred in steel bridges because of the influence of the increase in traffic volume and passing of heavy vehicles. It may be difficult to repair and reinforce the fatigue cracks because they are often initiating in narrow locations, where some welded members are crossing each other.

The Carbon Fiber Reinforced Plastic strip (it is hereafter called CFRP) is recognized to be a useful material for repair and reinforcement of existing steel structures, because of its lightweight, high strength and excellent corrosion resistance. In addition, the boning method for CFRP strips is very simple and is also easy to use in situ. However, In the repair of steel bridges in service using CFRP strips, not only steel members, but also the CFRP strips and epoxy resin adhesives on repair work are subjected to cyclic stress and vibration generated by traffic load. The aim of this study is to investigate for the adhesion characteristics and the effects of the fatigue crack repair using CFRP strips and epoxy resin adhesives under dynamic load during curing.

In this study, investigation was carried out experimentally. First, adhesion behavior under cyclic load during curing was investigated using coupon specimens of flat steel plates with bonded CFRP strips. Next, Effects of the fatigue crack repair under the similar condition were examined using coupon specimens of flat steel plates with a through crack. The sinusoidally varying loads were applied to the specimens at frequencies of 0.01 and 3.0 Hz for 24 hours. The curing temperatures were 40°C.

As a result, first, as shown in Figure 1, when the specimens were subjected to cyclic load during curing, it was observed that epoxy resin adhesives have cured in several hours and CFRP strips have been bonded at mean load. It means that the strains on CFRP strips were nearly zero at mean load. After unloading, tensile stresses were also introduced to the steel plate. It was found that the tensile stress did not influence

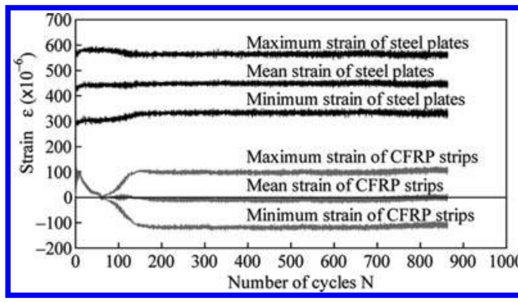

Figure 1. The relationship between strain and number of cycles.

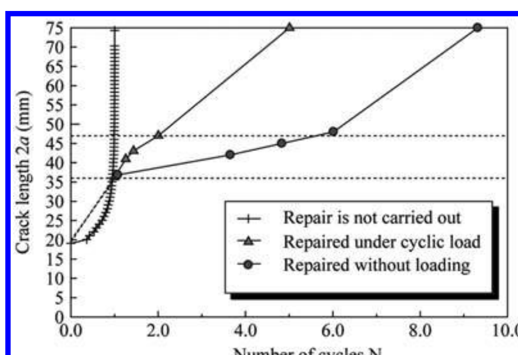

Figure 2. The relationship between crack length and number of cycles.

on static bond strength by tensile testing. Next, as shown in Figure 2, in the effects of fatigue crack repair under cyclic load, while the epoxy resin adhesive was curing, fatigue crack propagated slightly. However, it was found that the fatigue life after curing could be sufficiently improved by high cycle fatigue testing. Therefore, the CFRP repair in service conditions was confirmed feasible for practical use.

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An analytical study on the fatigue cracking of the top flange in steel railway through truss stringers

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INTRODUCTION

In steel railway through truss stringers, the fatigue cracks have been detected along the corner of angle steel in the top flange (see Photo 1). In the previous study, it is reported that the compressive stress is high at the both inside and outside of the top flange through measuring the transverse stress on the back of the top flange under the sleepers in the stringer. By the results of previous study, we could expect that both inside and outside of the top flange were displaced downward, and we considered that the behaviour of the top flange was controlled by the contact condition between the sleeper and the top flange. Although it can be understood that one side of the top flange is displaced downward, it has not yet been verified the mechanism of deformation that both inside and outside of the top flange are displaced downward at the same time.

In this study, the deformation down the both sides of the top flange at the same time is simulated through finite element analysis.

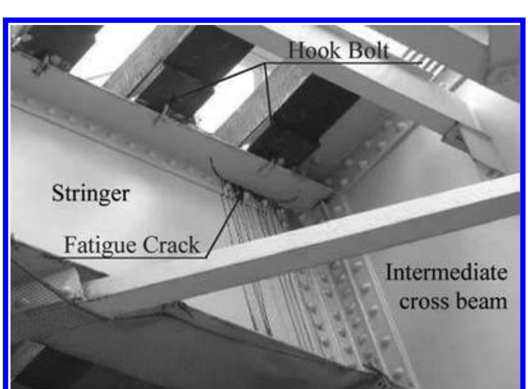

Photo 1. Fatigue crack.

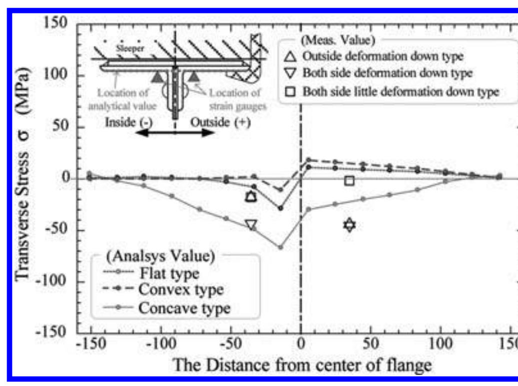

Figure 1. Transverse distribution of Transverse stress on top flange.

The principal results obtained through this study are as follows;

(1) In the case for the flat type or the convex type in the face shape of the top flange, the inside of the top flange was deformed down and outside of the top flange was deformed up. In the concave type, the both sides of the top flange were deformed down at the same time.
(2) The mechanism of the behaviour that the both sides of the top flange were deformed down at the same time is inferred the sleeper contacted strongly with the edge of the top flange because the both edges are higher than the center in the face shape of the top flange (see Fig. 1).
(3) Stress condition on top flange in stringer can be simulated by changing the face shape of top flange.

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A study of longer service life of a multispan simple steel railway bridge

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ABSTRACT

The average use years of steel railway bridges in Japan exceeds 60 years, and there are some bridges that is used for 100 years or more. In the future, concentration of renewal of steel bridges will come caused by corrosion, fatigue and so on. Given this situation, bridges its total length is long (for example it strides over a wide river) need large cost of operation and maintenance. It tends to be also difficult to rebuild those bridges by some restrictions. Therefore, it is valuable to make maintenance scenario of bridge for longer service life. We selected a bridge striding over a wide river in an actual important railway route as a model case and make several examinations focusing on corrosion, function of bearing and fatigue. The outlines of the examinations are as follows.

Concerning corrosion, we examined the present condition of films and steel corrosion, and the level of corrosive environment (the amount of salt adhered to members). By the results of the examinations, it is considered that the environment where the model bridge locates is not corrosive. While it is possible that films peel off widely where total film thickness is relatively thick or mill scale of steel member is fragile. Therefore, we have decided to examine the condition of former films and determine surface cleaning level before every repainting.

Concerning function of bearing, we marked lines to upper bearings and lower bearings of reformed bearings in 6 spans, and measured movement of bearings by variations in temperature for a year. By the result of the examination, it is considered that reformed bearings (sliding bearings) move smoothly at present. However, its durability is unknown. Therefore, we have decided to continue to measure movement of bearings by variations in temperature twice a year.

Concerning fatigue, a lot of cracks have occurred along angle corner of stringer upper flange in the model bridge recently. We analyzed tendency of cracks, measured stress at angle corner of stringer upper flange, and examined touch condition between sleeper and upper flange. By the results of the examinations, it is known that out-of-plane bending occurs at upper flange angle of stringer. And by the tendency of cracks, it is considered that stringer is a weak point on fatigue. So, it is possible that the amount of fatigue crack increases continually. Therefore, we have decided to strengthen inspection especially aiming at fatigue and exchange stringer members on a medium-range plan. And, we practiced a test construction.

In the past, the thought of *Scrap & Build* was common and a lot of bridges were rebuilt. However, recently, thought of *Longer Service Life* of infrastructures such as bridge etc. has become major gradually. In such a situation, we think this work can contribute to longer service life of the model bridge.

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Development of XFEM analysis code for simulation of fatigue crack propagation in steel structure

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ABSTRACT

Recently, the fatigue damages of the steel floor systems of the aged bridges have been reported. It is therefore urgently required to clarify the cause of the damages and to rationalize the maintenance such as inspection, repair and reinforcement.

The numerical simulation of the behaviors of fatigue crack in the steel floor systems is effective for the clarification of the mechanism of fatigue crack propagation and the rationalization of maintenance for the aged steel bridge.

The XFEM proposed Belytschko and his coworkers (Belytschko & Black 1999) is a numerical method employs the local enrichment function, which enables the approximation allowing the reproduction of singularity or discontinuity in the local parts of the domain, though it is formulated in the framework of the FEM. As a feature of the XFEM, the crack can be modeled independently of finite element meshes. Therefore, the XEFM has a potential ability to solve the problem of vexatious complication in the modeling of crack propagation using the standard FEM.

In the present study, in order to evaluate the behavior of a fatigue crack in the local part of large-scale civil engineering structure, we develop a fatigue crack simulation code based on the general-purpose FEM analysis software ABAQUS through the implementation of the PU-XFEM approximation (Shibanuma & Utsunomiya 2009). In particular, multiple-nodes are introduced to increase nodal degrees of freedom.

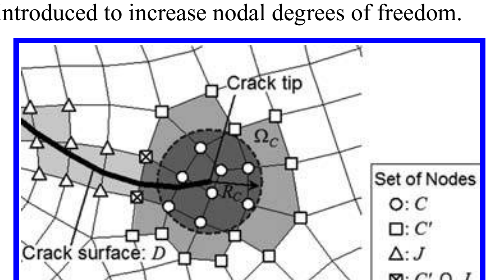

Figure 1. Modeling of crack in the PU-XFEM.

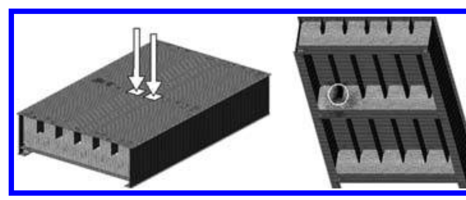

Figure 2. Numerical model of the orthotropic steel deck specimen.

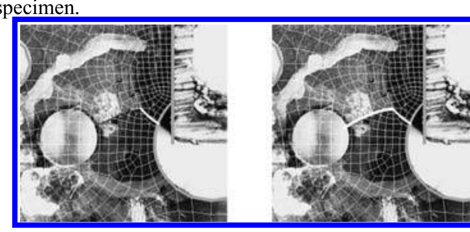

Figure 3. Comparison of the results of fatigue propagation path in the numerical simulation and fatigue test.

Using this simulation code, the behavior of a fatigue crack through thickness of a plate in a three dimensional structure can be quantitatively evaluated. The performance of the developed PU- XFEM code is evaluated through its applications to the numerical simulations of the fatigue crack propagation in the intermediate floor beam of I-girder bridge and the orthotropic steel deck specimen using bulb rib.

It is concluded that the developed PU-XFEM analysis code is useful for the quantitative evaluation on the path and rate of the fatigue propagation including the estimation of possibility of crack propagation through thickness of a plate in a three dimensional structure.

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Development of health monitoring indicators for steel bridges and its implementation to sensing systems

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ABSTRACT

Nowadays, there is a prevailing expectation for Structural Health Monitoring (SHM) based on real-time and online evaluation technologies to improve efficiency and reliability of maintenance of infrastructures. The target of this paper is to develop a practical SHM method, i.e. a sensing system which is defined by indicators based on the structural performance.

According to Maintenance Standards for Railway Structures in Japan, performance items include a) safety: load-carrying capacity, durability of fatigue resistance, running safety and stability, public safety, b) serviceability: train operating, running comfort, c) restorability. These items are related to the performance indicators such as, stress and displacement. Especially displacement is the key parameter for health monitoring because it is closely related to running safety. Moreover, in design, displacement is the indicator which is verified not to exceed the limit state.

To evaluate strucutural performance, indicator of displacement are useful. But, direct measurement of displacement generally requires fixed reference point, which makes it difficult and costly. To implement to sensing system, estimation method for determination of the maximum displacement from inertial measurements such as inclinometer, and accelerometer are needed. To this end, field test has been executed at a plate girder railway bridge which has eight spans with length of 8×22 m. The No.1 span and No.1 pier are chosen. The measurement includes displacement, inclination, acceleration, and temperature. Since bridges are flexible and displace under various loads, monitor both dynamic and static displacements are important. Therefore, our measurements include two ways which show below,

- Shot term continuous displacement measurement executed for 24 hours, using ring type displacement transducers and other sensors as shown heading 3.
- Long-term inertial measurement of piers has been executed using inclinometer, accelerometer. Recording of data are conducted 120 minutes by a trigger of the amplitude of accelerometer.

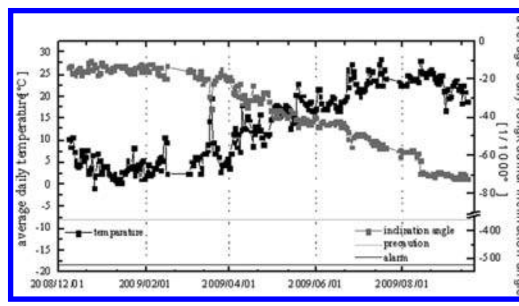

Figure 1. Representative long-term inclination angle of the steel bride pier with clinometers and temperature.

One of the results, figure 1 shows in a ten-month-long record of long term inclination of steel bridge pier by clinometers. Inclination of piers shows reaction on changing temperature, therefore static inclination has daily cycle. Focus on the long term trend, daily mean of inclination are described along average daily temperature. Since large inclinations are caused irregularity of gauges, it keeps up under regular limits. Threshold value which is calculated from the design limit value of angular rotation on the truck surface is also shown in figure 1. With consideration of seasonal periodicity or daily periodicity, it is possible to judge the soundness of piers form the magnitude relation of them.

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Retrofit and advanced investigation on fatigue cracks penetrating orthotropic steel deck plates

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ABSTRACT

The crack that penetrates through the deck plate occurs because the stress concentrates on the weld root. This concentration of stress is caused because the deck plate bends locally by the weight of the car. It is serious damage that causes the pavement damage and the road cave-in when this crack progresses. However, orthotropic steel deck with trough rib is difficult to detect fatigue cracks by visual inspections or Magnetic Particle Testing (MT) under the decks without pavement removal on steel deck plates. Then, it paid attention to PAUT to be able to expect a high detection performance as the inspection technique, and the detection performance was confirmed to the crack of a real bridge.

The result and the crack investigation imitative chart investigated by PAUT to the crack of penetration through the deck plate are shown in Figure 1. A left image shows the bridge axis right-angled direction section in the deck plate, and a direction of the deck plate thickness and a horizontal axis show the result of the survey between 35° in investigation angle and 80° in the spindle in a bridge axis right-angled direction. The part that enclosed it with a left image shows the crack, there is a crack signal up to the board thickness 12 mm that becomes upper of the deck plate from the lower side of the deck plate, and it can be presumed the penetrated crack. The penetration crack length of the direction of the bridge axis in the continuous confirmation in the direction of the crack signal in each section of the bridge axis like this, and the unpenetrated crack can be presumed to be length. Consequently, all crack length was estimated to be 356 mm. When the shape of the penetration crack that generated it in the steel plate deck was compared, it became a result that was about 7 mm in all crack length shorter. It was able to be confirmed PAUT is to be able to investigate the crack here in high accuracy.

Next, the guss asphalt was exchanged for the steel fiber reinforced concrete (SFRC) which increases the bending stiffness of the thin deck plate against the penetrated crack. Stress reduction effect using the SFRC was confirmed in the orthotropic steel deck bridge.

The stress measurement measured the strain at 5 mm position from each weld toe for the stress etc. of a deck plate and trough rib weld surrounding. The load was executed with the test car adjusted to 245 kN in total weight. One example of the result is shown in Figure 2. According to this, the stress range reduction to about 16% by the maximum value in the weld deck plate side toe of the deck plate and trough rib. The stress range reduction of about 50% was confirmed between cross ribs. As for the stress range in the deck plate weld toe at a cross rib reduced more than 50%. The improvement of the fatigue durability by the SFRC pavement can be expected for the penetrated crack through the deck plate.

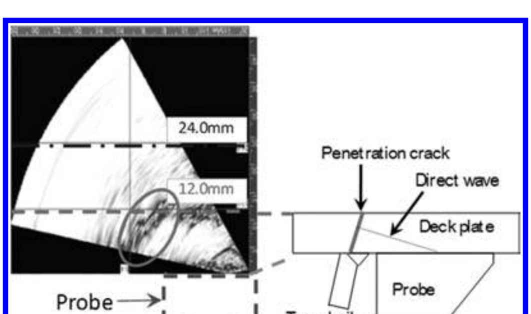

Figure 1. Example of sector scanning image of penetrated crack.

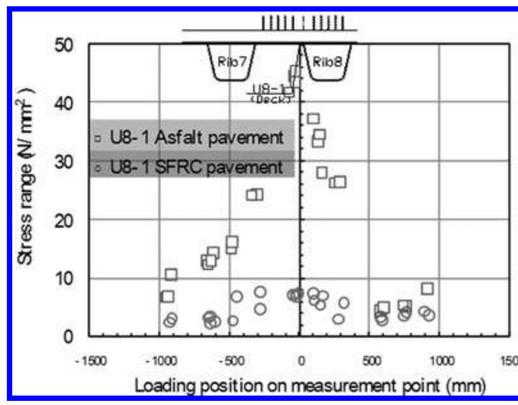

Figure 2. Comparison between SFRC pavement and asphalt (deck side toe between cross ribs).

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Study on improvement of fatigue strength of out-of-plane gusset welded joints by attaching GFRP

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ABSTRACT

In this study, in an attempt to examine the effects of GFRP on the improvement of the fatigue crack initiation life Ni and fatigue crack propagation life Np, fatigue tests were conducted for out-of-plane gusset welded joints to which GFRP were attached using the primer. The effectiveness of GFRP for reducing the stress at the toe of round weld was also investigated analytically. The effectiveness of GFRP for controlling the fatigue crack propagation rate was also considered from a viewpoint of fracture mechanics.

The dimensions of the specimen are shown in Figure 1. Figure 2 shows S-N curves indicating the regression curves of respective specimens. The regression lines of GN and GF were close to each other in the high stress range. The fatigue life of GF was much longer than that of GN in the lower stress range. In the high stress range, therefore, attaching GFRP may not be so effective as expected for strengthening. It has, however, been confirmed that attaching GFRP is effective in the lower stress range.

The conclusions obtained are described below.

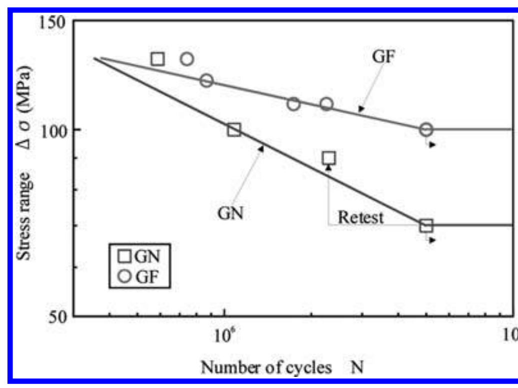

Figure 2. S-N curves (regression curves).

(1) It was confirmed that fatigue life could be improved by attaching GFRP to the round weld of the out-of-plane gusset welded joints.
(2) The improvement of fatigue life through the attachment of GFRP using the primer was attributed in this study to the reduction of the crack propagation rate and the improvement of crack propagation life by GFRP.

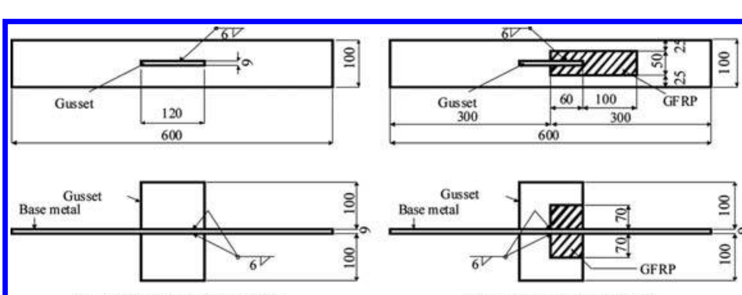

Figure 1. Specimens.

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Study on improvement of the fatigue durability by filling of mortar in U-shaped rib of orthotropic steel deck

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ABSTRACT

This paper reports to study on the method of retrofitting orthotropic steel deck without need for traffic restriction for eliminating fatigue cracks originating from the weld between the deck plate and U-shaped rib and preventing fatigue damage. We proposed the reinforcement method as filling of mortal in U-shaped rib and installation of the splice plates between the U-shaped rib as shown in Figure- 1. The static loading test and the wheel load running test were carried out in order to verify the fatigue durability and the stiffening effects by the retrofitting. The main conclusions and discussions are as follows,

1. As for the result of the static loading test, the stress reduction effective ratio due to reinforcement for local stress of neighborhood welding between U-shaped rib and deck plate were shown approximately 30% on the deck plate side of the center of the U-shaped rib, also U-shaped rib web was about 40% comparison with the original. Especially It was remarkable the stress reduction effective of the intersection of the U-shaped rib and crossbeam (Figures 2, 3).
2. As a result of the wheel load running test for the non-retrofitting model of the specimen, it was confirmed the crack at the intersection of the U-shaped rib and deck plate which occurred the non-welding point of the longitudinal welding of the U-shaped rib and deck plate at 600,000 cycles of the load running.
3. It was confirmed that the stress decrease effect of the measurement points due to the reinforcement are stabilized to the test end respectively, in addition, the crack on the deck plate after the test ended doesn't have much progress comparing with the original(Figure 4).

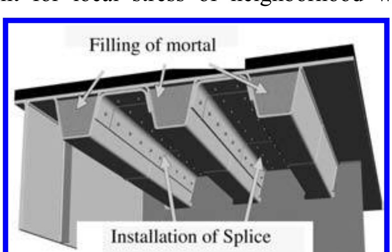

Figure 1. Retrofitting method.

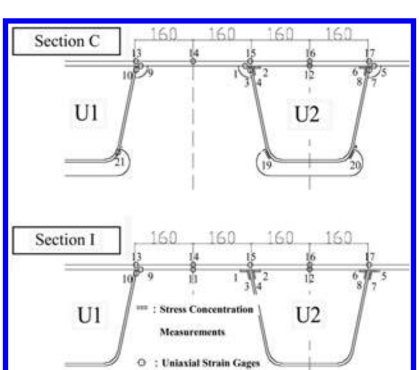

Figure 2. Location of stress measurement.

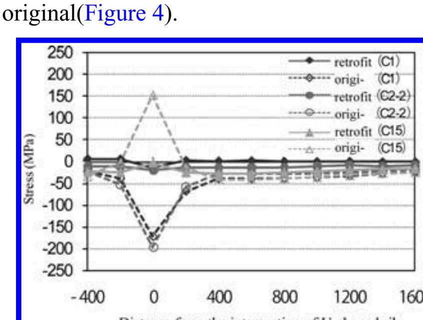

Figure 3. One example for stress influence line.

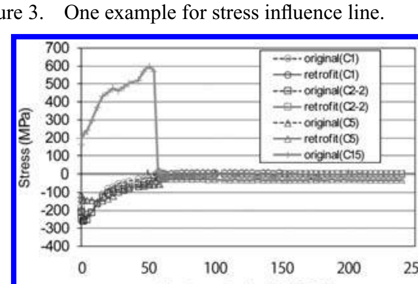

Figure 4. One example for change of stress.

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Study of fatigue assessment of orthotropic steel decks in consideration of variations in locus distribution of wheels

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ABSTRACT

Recent considerable increases in traffic intensity and wheel loads are causing fatigue cracks in orthotropic steel decks. Under traffic loading, in particular the effect of local wheel loads, longitudinal welds between deck plate and trough are subjected to local transverse bending moments and are susceptible to fatigue cracks. The stress in trough to deck plate welds is strongly influenced by actual-working load and run location.

Then, in consideration of the variation in locus distribution of a wheel, the method of carrying out fatigue assessment of orthotropic steel deck analytically are studied. In this study, the stress function is computed by the influence-line load of the vehicles load of a transverse direction. Locus distribution of the wheel is given with the frequency function of the normal distribution.The equivalent stress range in consideration of this locus distribution is calculated based on the linear damage rule, and fatigue assessment are performed.

1 MODELING OF RUN FREQUENCY DISTRIBUTION

Distribution of the run location of the transverse direction of vehicles can be expressed with a normal distribution. The frequency function is given by average and standard deviation by the following formula.

$$f(x) = \frac{1}{\sqrt{2\pi}\,\sigma} e^{-\frac{(x-\mu)^2}{2\sigma^2}} \tag{1}$$

Where,
σ: Standard deviation of frequency distribution of a run location.
μ: distance from left side lane space marks end to the maximum frequency location of centre of the wheel in slow lane.

The equivalent stress range $\Delta\sigma_e$ which is a stress which does equivalent fatigue damage with the same number of repetitions as a change amplitude stress is given by the linear damage side by the following formula.

$$\Delta\sigma_e = \sqrt[m]{\sum \Delta\sigma_i^{\,m} \cdot n_i \big/ \sum n_i} \tag{2}$$

Where,
$\Delta\sigma_i$: Stress range which acts on a weid joint (MPa)
n_i: Number of repetitions of stress range
m: $m = 3$ grade of a fatigue design curve

In Fig-1, lane space marks are made into the starting point($x = 0$), and the location of a run wheel is expressed with L_T. The stress range generated in target weld joint located in $x = L_T$ can be expressed with the function ($\Delta\sigma_{L_T}(x)$) of the load location of transverse direction. That is, $\Delta\sigma_{L_T}(x)$ is an influence line.

Next, the equivalent stress range in consideration of run frequency distribution of vehicles is examined [1]. Instead of stress range which acts on the joint in a formula (2) $\Delta\sigma_{L_T}(x)$, is used.

The frequency function of formula (3) is used instead of the number of repetitions of a stress range.

$$\sum n_i \cong \int_0^{3250} \frac{1}{\sqrt{2\pi}\sigma} e^{-\frac{(x-\mu)^2}{2\sigma^2}} dx = 1 \tag{3}$$

Therefore, the equivalent stress range in consideration of the distribution of the run location of the transverse direction of a wheel to the location of welding by which examination by reference is carried out is given by the formula of the 3rd square average of the integration of the frequency function of a normal distribution, and a stress function.

$$\Delta\sigma_{e_{L_T}} = \sqrt[3]{\int_0^{3250} \{\Delta\sigma_{L_T}(x)\}^3 \frac{1}{\sqrt{2\pi}\,\sigma} e^{-\frac{(x-\mu)^2}{2\sigma^2}} dx} \tag{4}$$

Where,
$\Delta\sigma_{e_{L_T}}$: The equivalent stress range in consideration of the variation in the run location of the transverse direction of a wheel.

With reference to the formula (4) mentioned above, calculation of the equivalent stress range uses numerical integration to a generating stress range. By the calculation program, the smoothing interpolation function which is the 3rd equation is determined from the data of a generating stress range, and numerical integration is repeated in the section length of 1 mm.

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Experimental study on the root fatigue life improvement for the reasonable design of steel pier seismic reinforcement

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ABSTRACT

Recently, many cracks have been found at the corners of beams to column connections of steel bridge piers. The majority of these cracks are fatigue cracks caused by incomplete penetration defects, as well as weld cracks related to those defects.

Because of the complexity of plate arrangements at beams to column connections of steel bridge piers, there exist zones where complete penetration is difficult, especially where welding lines intersect from three different directions (Miki et al. 2003). As a result, incomplete penetrations and weld cracks are often formed at the corners of beams to column connections of steel bridge piers.

For the current retrofitting to such damaged areas, the method of large coring has been developed and applied. This is a method of removing damaged zones, in which cracks and weld defeats converge, using a 100 mm diameter large coring, after reinforcing the beams by attaching bolted plates. This can improve the fatigue life of the detail efficiently in a short construction time.

However, this method leaves a difficulty: weld roots are exposed after removing the area with defects by coring. While weld roots, where present, are always closed in welded steel structures, in the method of large coring incomplete penetration defects are visible from outside. In this case, the stress concentration at the edge of the defect is larger compared to the case of the closed space. Also the corner of a beam to column connection is the point at which working stress from dead and live loads is very large, and coring increases this working stress. Since stress concentration is usually larger than before and working stress can still be large, there is a risk of the return of root cracks. And they can induce brittle fractures during earthquake.

But if the fatigue performance of the open root could be improved, it would be possible to simplify or even omit the reinforcement by bolted plates in the case of small working stress. This study, thus, aims at realizing reasonable seismic design by improving the fatigue life of the detail with open cracks after large coring. Since the likely crack initiation point is an open root, this study proposes several new treatments to the open root and experimentally examines their effects on the structural improvement of the fatigue life.

In this study, root edge drilling, pin shooting and cover welding are tried to improve root fatigue performance. And all these method are conducted together with ultrasonic impact treatment to control residual stress at surfaces and toes. Each of these methods showed fatigue improvement, but especially root drilling with cover welding was efficient.

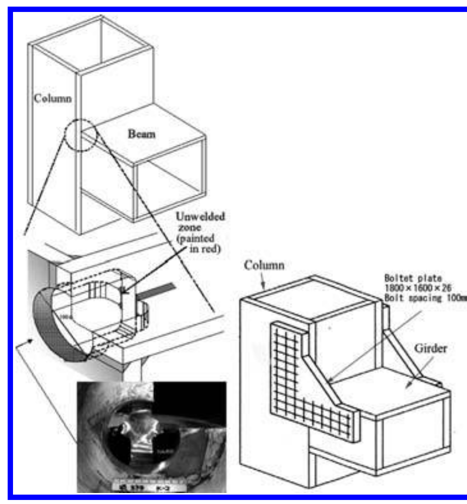

Figure 1. Description of a large coring method and bolted splice reinforcement.

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Effect of reinforcing method against fatigue cracking of orthotropic steel deck with bulb ribs

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ABSTRACT

Recently, thousands of fatigue cracks have been detected in orthotropic steel decks in Japan. Of these, fatigue cracking in welded joints between bulb ribs and lateral ribs is the most frequent type found in the Kansai area. Fatigue cracks are classified into 4 types, as shown in Fig. 1 (Tabata et al. 2007). At the intersection of the bulb rib and the lateral rib, the crack that propagates from the weldment of the lower part of the slit into the lateral rib web is d-type, and the other that propagates from the weldment of the upper part of slit into the deck plate is a-type. These cracks may propagate into the deck plate and have a bad influence on traffic.

It is important, therefore to grasp such fatigue cracking behavior. An effective method against the fatigue cracks is needed. In this study, we tried to grasp fatigue cracking behaviour in the welded joints between the bulb rib and the lateral rib by means of fatigue tests of the orthotropic deck plate specimen which is the same size and has the same structural detail as the actual bridge. Also, we verified the effect of the proposed reinforcing method using angle steels, as shown in Fig. 2.

Fatigue cracks were initiated at the weld toe of the upper part of the slit and propagated through the weldment into the deck plate, as shown in Photo 2. We confirmed that the fatigue crack detection life of the welded joint between the bulb rib and the lateral rib was improved more than eight times by applying angle steel reinforcement.

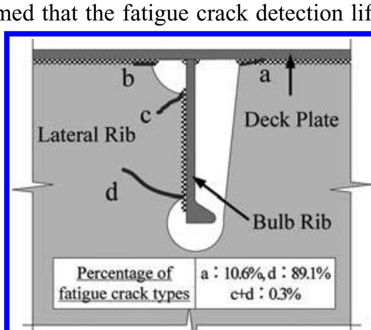

Figure 1. Fatigue cracks in welded joint between bulb rib and lateral rib in the Hanshin Expressway.

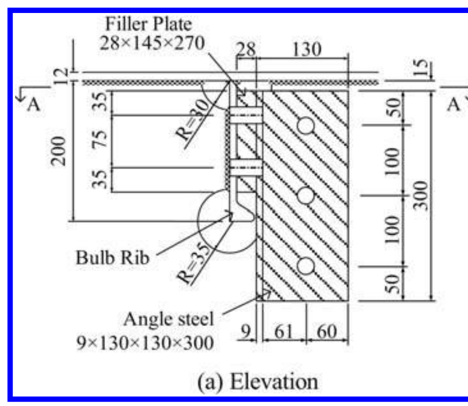

Figure 2. Intersection of bulb rib and lateral rib with angle steel reinforcements.

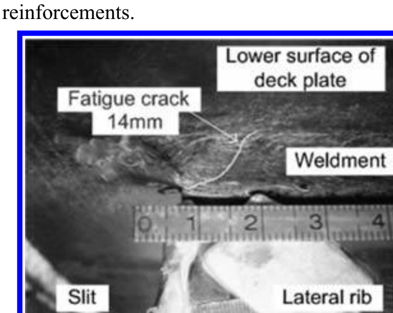

Photo 1. Fatigue crack at upper part of Rib 1 slit after 0.7 M cycles loading.

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SS25: New developments in bridge design codes

Organizer: A.S. Nowak

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Development and calibration of new reliability-based bridge design code in Korea

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Most internationally-renowned design codes for bridges have implemented limit state concepts including AASHTO and Eurocode. Since the first bridge design code was introduced in 1962 in Korea, regular revisions have been made in accordance with new social and technological needs. However, to meet with global standard, state-of-art technology and fast-changing local environments, new bridge design code is being developed based on reliability or limit states design concept.

This paper deals with the development of reliability-based bridge design code and calibration of load and resistance factors based on new live load model. To establish statistical characteristics of live loads, truck weights and traffic patterns information are collected using WIM system and video recording. Based on probabilistic analysis, new live load model and their statistics are determined. Statistical data on strength of concrete, reinforcing bars and steel members are collected and used in evaluation of member strength. Reliability analysis is performed on various types of bridges. Comparisons are made with current design code to prove the validity and effectiveness of proposed design code. It can be concluded that proposed live load model and proposed load and resistance factors give more consistent reliability indices for various load effects and span lengths.

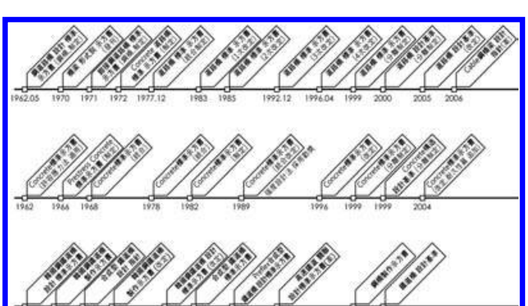

Figure 1. Evolution of design code for bridges.

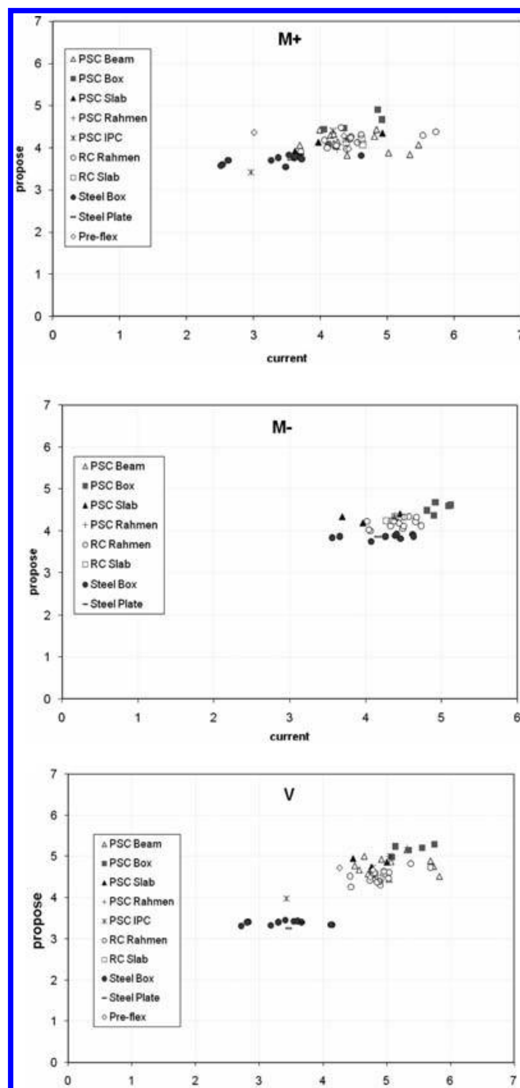

Figure 2. Comparison between proposed β and current β.

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A study on ductility of high strength steel bridge girder

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ABSTRACT

It is possible to apply the high-performance steels(HPS) to practical design not only light weight structures but also simple structures with simple weld details. However, from the results of previous researchers, it is found that the ductility of girder with HPS is reduced due to the increasing of yielding stress. Thus, the study on method to guarantee the proper ductility of girder with HPS is necessary. This paper investigates the flexural ductility of I-girders in negative moment region with HSB800(Fy = 680 MPa). The flexural strengths of bridge girder which assumed to be made by using 800 MPa class high performance steel are evaluated based on AASHTO LRFD design code.

Concrete deck cannot resist in negative moment regions due to the fact that the tensile stress is reached in the upper flange. Therefore, negative moment regions are designed by concept of noncompact section. Moment envelope of negative moment region is very sharp and higher than positive moment region. Therefore, plastic hinge negative moment region can be generated easier than that in positive moment region. Thus, it is necessary to provide the sufficient ductility for negative moment region.

This study prescribes an alternative bracing system to ensure sufficient flexural ductility. The alternative bracing system does not require any additional cross beam members. The alternative bracing system needs to move the cross beam members to near the bridge pier. From the result of A800E model, despite of the satisfaction of AASHTO LRFD(2007) compactness, the girder cannot attain the desired rotational capacity. However, By using alternative bracing system, it is found that the rotational capacity of the girder is increased compare with ordinary bracing system.

This study also prescribes Optimum α using yielding range when girder is reached on plastic moment. The girder have sufficient rotational capacity when cross beam is placed in optimum α.

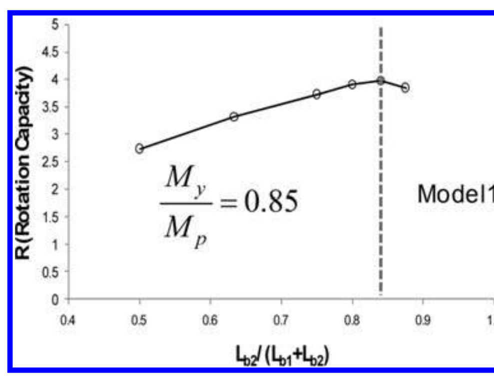

Figure 1. Results from FEM.

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Development of the AASHTO guide specifications for bridges vulnerable to coastal storms

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ABSTRACT

In response to the need to learn more about how to design and retrofit coastal bridges subjected to coastal storms such as hurricanes, the FHWA authorized a research project entitled Development of Guide Specifications and Handbook Retrofit Options for bridges vulnerable to coastal storms. This paper summarizes work that culminated in the 2008 adoption of AASHTO'S *Guide Specifications for Bridges Vulnerable to Coastal Storms*.

A literature study undertaken early in the project showed that while many investigators have studied the action of waves on structures, most of the work concentrated on the vertical components of the structures, or structures that differ greatly from a bridge superstructure. The existing research on superstructure components concentrated on offshore environments, where the design wavelength is much greater than the length of the structure.

Physical model tests were conducted in a 6 ft wide, 6 ft deep, 130 ft long wave tank at the University of Florida. Numerous plots were developed showing the comparison of wave forces predicted by various methods and the results of the wave tank experiments. The basis for a choice of a method to be proposed in the Guide Specification involved evaluation of the predicted forces compared to the physical model test data, the prediction of failures on the I-10 Escambia Bay Bridges, theoretical completeness, and practicality. Based on an evaluation of all these factors, it was decided to proceed with the Physics Base Model (PBM) developed by Ocean Engineering Associates, Inc. (OEA) method. The PBM, which can be applied to both submerged and subaerial structures, had great appeal due to its theoretical completeness.

The PBM was run for a range of span configurations and elevations (relative to the storm water level), water depths, and wave conditions and the maximum vertical force and associated horizontal force and moment data compiled. This data was then used to develop parametric equations for the various components of wave forces. The resulting equations are complex but amenable to solution by spreadsheet.

A three-level approach was adopted in order to analyze the wave forces for a given situation based on 100 year design conditions. Level 1 involves a conservative evaluation of the situation using "available data" for wind and storm water height. The appropriate sources for data for the Level 1 analysis would be either site-specific information on astronomical tide, wind and surge, or the use of the ASCE 7-05 wind maps adjusted for 100 year return period in combination with FEMA storm surge maps. A Level 3 analysis is much more thorough and involves the use of a number of sophisticated storm surge and wave computer models and must be performed by qualified coastal engineers. A Level 2 analysis falls somewhere between Level 1 and Level 3 analyses and considerable latitude is permitted in the choice of how far between Level 1 and Level 3 the engineer believes is appropriate for the particular situation.

Two limit states are recommended for the design of bridges vulnerable to coastal storms: strength and extreme event. Bridges which are designated as "critical/essential" by the owner should be designed at the Strength Limit State to achieve a performance level of "Service Immediate," or "Repairable Damage," depending on how quickly they must be passable after the storm.

"Service Immediate" can be taken to mean that the bridge may be assumed to be sufficiently undamaged, stable and aligned to be usable for rescue and recovery forces after a cursory inspection.

"Repairable Damage" can be taken to mean that some repairs could be needed to restore sufficient serviceability to put the bridge back in limited use within the owner's criteria for outage duration and after an inspection.

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Approach for developing calibrated service limit states for the AASHTO LRFD bridge design specifications: A progress report

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ABSTRACT

The notion of limit state, the boundary between acceptable and unacceptable performance, is fundamental in the *AASHTO LRFD Bridge Design Specifications* (AASHTO LRFD).

The current service limit states of AASHTO LRFD are intended to ensure a serviceable bridge for the specified 75-year design life. These limit states are based upon the traditional serviceability provisions of the AASHTO Standard Specifications for Highway Bridges. However, they are not calibrated using reliability theory. This paper describes ongoing work to develop calibrated service limit states under project SHRP2 R19B.

Among others, the current service limit states in AASHTO LRFD include limits on live load deflection of bridges, cracking of reinforced-concrete components, tensile stresses of prestressed-concrete components, compressive stresses of prestressed-concrete components, permanent deformations of compact steel components, slip of slip-critical friction bolted connections, and settlement of shallow and deep foundations. Some of these service limit states may relate to a specified design life; others do not. Many are presently very deterministic. One of the major objectives of the reported work is to develop the tools necessary to:

- design a new bridge to achieve a specified service life approaching or in excess of 100 years, this will require identification of service limit states and specification provisions to implement them for a minimum 100 year service life, and development of performance measures for calibration processes, as well as to monitor the performance of new bridges designed with the new service limit state provisions and to gage the performance of older bridges for the purpose of bridge management.
- determine how much service life remains in an existing, possibly deteriorated, bridge, and
- quantitatively determine the effect of retrofit, repair or maintenance on remaining life,

all with a specified certainty or probability.

To achieve the objective of developing the appropriate tools, candidate service limit states have to be evaluated against a set of criterion including:

- Is the limit state quantitatively and qualitatively meaningful?
- Can the limit state be calibrated?
- Does a limit state really relate to the service life rather than to some other characteristic?
- Does it provide a method to evaluate the significance of interventions in extending the service life of the structure component?

An acceptable probability of exceeding a SLS is much higher than for ULS. If the target reliability index for ULS is $\beta_T = 3.5$ to 4.0, then for SLS, $\beta_T = 0.0$ to 1.0 may be shown to be appropriate.

Given a target reliability index, the anticipated calibration procedure is as follows:

- Select representative location characteristics such as climate.
- Select representative traffic characteristics.
- Identify the design parameters.
- Provide for user prediction of performance and deterioration.
- Select the acceptability criteria, i.e., performance parameters that are acceptable, and those that are not.
- Calibrate the load and resistance factors to meet the acceptability criteria.
- Review the developed design parameters, and make necessary adjustments.

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Effect of tension stiffening in composite bridges in the light of Eurocodes

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ABSTRACT

One of the crucial problems concerning the design calculations of the composite bridges with continuous structural system is to take into account the cracks which can occur in the concrete deck slab in the regions of negative bending moments (hogging moments). The above problem can be considered ac cording to the procedure proposed in the Eurocode EC 4, EN 1994-2 (2005).

The Eurocode 4 allows to consider the stiffness of the composite girder with the cracked concrete slab deck using the diagram 'normal force – strain' for tensioned reinforced concrete structural members. Three fundamental regions of the above diagram can be differed, denoted by 'a', 'b' and 'c' (Fig. 1).

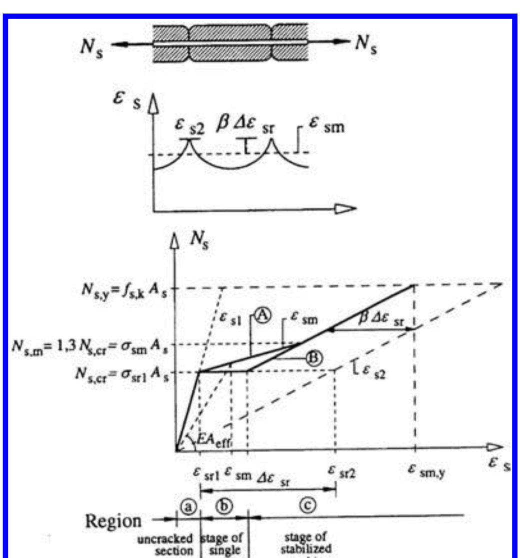

Figure 1. Normal force vs. mean strain for reinforced concrete tension member.

Region 'a' corresponds to the stage when the concrete bridge slab is un-cracked, region 'b' corresponds to the stage when the initial crack in the slab is formed, and region 'c' corresponds to the stage when the cracked pattern in the concrete slab is stabilized. For each of the above stages the formulae concerning the relation between the normal force in the concrete slab and the strains in the concrete and its steel reinforcement are given.

Next, the internal forces in the composite section are analyzed without and with the effect of shrinkage. The cracking moment is defined in particular in the mentioned above regions 'b' and 'c'. The additional normal force of the concrete section due to tension stiffening is also given in two fundamental cases – without and with prestressing the concrete slab by tendons.

Finally, the contribution of concrete in tension between the cracks in the stiffness of a composite cross-section is presented using the relevant formulae.

As a one of the conclusions, it is emphasized that the effects of tension stiffening in composite steel and concrete bridges proposed in EC 4-2 (2005) leads to the more economical solutions compared to the design procedures applied so far in many countries, including Poland. The relevant procedure can be applied both for designing process of the new composite bridges and for strengthening of the existing ones where taking into account tension stiffening of the cracked concrete slab allows to obtain more economical solution of the strengthening itself.

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SS26: WIM-based live loads for bridges
Organizer: A.S. Nowak

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Extreme truck load effect prediction for bridge structural reliability

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ABSTRACT

Structural design and evaluation are moving towards reliability based practice. The latest building and bridge design specifications in the US are examples of this trend. In developing these specifications, risks involved with major design factors were modeled as random variables, such as the live load for the structure, the resistance of structural members, the environmental loads due to the wind and flood, etc. The load factors and resistance factors were then derived by controlling the failure of concern. On the other hand, this first generation of reliability based specifications still has ample room for improvement. This paper focuses on one of such issues relevant to maximum load projection for the intended life span of the structure applicable to deriving the truck live load model and its load factor for highway bridge design and evaluation codes.

Using short term load data to extrapolate or project to remote future maximum load is often exercised for specification calibration and development. It was indeed done for the AASHTO LRFD bridge design specifications. However, this approach has not been adequately evaluated or validated due to lack of long term data, which is important for the structures' safety governed by these codes. This issue is focused here for the application to highway bridge design and evaluation, taking advantage of available long term truck weight data obtained using the weigh-in-motion technique. A new extrapolation method is proposed here for more reliable projection, based on the understanding of importance of the load probability distribution's high tail. This approach significantly reduces the mathematical length of extrapolation/projection and thus increases its reliability, also shown herein by application examples.

This method is also applied here to develop highway bridge evaluation requirements for practice in China. Over the past two to three decades, the economic development in China has nurtured the establishment of a highway network with a large number of bridges. However, there are still no nationwide specification provisions for assessing their safe load carrying capacity. The research work reported herein focuses on developing reliability based requirements for this purpose. In this study, weigh-in-motion data for more than 7.3 million trucks were gathered from highways in three provinces of China, continuously over one to 16 months in 2006 and 2007. The data were processed and projected to model the live load spectrum over 3-year and 100-year periods, respectively. The former is the required bridge inspection interval and the latter the bridge design life span, according to current Chinese maintenance and design specifications. The proposed projection method is shown to be more reliable compared with those reported. The resulting load spectra are used to assess the structural reliability of typical Chinese highway bridges at the component level. Based on the accordingly selected target reliability index, the live load factors for bridge evaluation are developed in this study, proposed to be included in the Chinese national specifications.

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US bridge formula (FBF-B) and implications of its possible application in Europe

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ABSTRACT

Bridges are designed using conventional load models, such as those defined in the AASHTO and Ontario bridge codes or the Eurocode 1991–2, to ensure that bridge members and elements are able to support the effects of service trucks and exceptionally loaded vehicles including those due to single truck crossings and overtaking events as well as the corresponding fatigue stress cycles To ensure the safety of existing bridge and highway networks, gross vehicle, single axle and multiple (tandem, tridem…) axle set limits are enacted in State or Federal laws (e.g. the 1956 Highway Act) or International directives (e.g. European directive 96/53EC). In particular, in response to the dramatic increase in heavy trucks that occurred in the late 50s and early 60s, and out of concern over the damage caused by heavy, single unit trucks, a bridge formula was developed in the US which accounted for the different levels of road damage caused by different vehicle configurations and weights including the number of axles and their spacing. In 1974 the Federal Bridge Formula-B (FBF-B) was passed into law to ensure that the truck fleet did not produce overstresses on the representative bridge network which was dominated by simply supported superstructure at the time. No change has been made to the US Federal Bridge Formula or federal gross and axle weight limits since 1974, while the truck fleet, design and volume have dramatically evolved.

To reduce GHG and CO2 emissions, road congestion, and to satisfy the increasing demand of freight transport and increase road transport efficiency, studies are carried out in the US and in Europe on the benefits and disadvantages of increasing the Heavy Vehicles weights and dimensions, and thus their capacity. There is a strong demand to extend the general gross weight limit up to 97,000 lbs in the US and to 44 t in Europe. Moreover, some European countries already allowed or experimented with European Modular System (EMS) vehicles with a total length of 25.25 m and a mass limit of 60 t.

To properly manage the changes in HGV weights and dimensions without increasing their damage to existing bridges, it is important to maintain some form of the FBF-B formula to limit the weight-to-length ratio, but the existing formula may require some up-dating after 35 years. It is also proposed to investigate the possibility of implementing the FBF-B to the European HGVs truck fleet and study whether it is applicable to limiting the damage on typical European bridge shapes and topologies.

An investigation is being carried out to study the damage caused by the most common US and European trucks to existing bridges for both maximum load effects and fatigue damage. Maximum bending moments at mid-spans and on piers, as well as shear forces of simply-supported spans, and continuous 2 and 3 span bridges, with span lengths from 10 to 100 m were considered. For each truck n with its maximum permitted weight Wn, and a weight limit imposed by the FBF-B of Wbfn, a load coefficient was calculated as: cn = Wn / Wbfn. For each truck (and weight Wn) and each load effect, a damage coefficient Kn with respect to a reference truck r was calculated, as the ratio of the maximum load effect induced by the truck n and the reference truck r. Then Kn was compared to the ratio Cn = cn / cr, to assess the homogeneity of the FBF-B weight limit. It is shown that the US bridge formula is adapted to simple supported spans of 20 m and two continuous spans of 10 m each. Fatigue damage under stress cycles is also considered and a similar approach is developed to assess the ability of the FBF-B to limit fatigue damage. Finally, some proposals are made to adapt the FBF-B for Europe.

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Live load models based on weigh-in-motion data

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ABSTRACT

The Weigh-In-Motion measurements provide the unbiased truck traffic data and it is a rational basis to develop the statistical model of live load for a code calibration. The objective of this paper is to present the development of statistical models for live load that is applicable to the AASHTO LRFD code calibration. The database presented in this study includes truck records from four states: Florida, Indiana, Mississippi, and New York. The total number of trucks exceeds 35 millions. For each location, the WIM data represents one year of truck traffic. For each measured vehicle, the recorded information includes the gross vehicle weight (GVW), number of axles, load per axle, axle spacing, lane of travel and speed. The raw data was filtered and preprocessed to ignore any errors in weight per axle, spacing between axles and speed. An additional filter was implemented to verify the class of the vehicle. For all truck, the maximum load effect was obtained using structural analysis procedures. The results were extrapolated up to 75 year maximum values. The cumulative distribution functions of GVW and live load moments were plotted on the normal probability paper. The plots show a large variation that indicates that the live load is a strongly site-specific. For comparison, the truck survey data used in calibration of the AASHTO LRFD Code is also presented.

WIM stations are usually hidden from the truck driver. Probable illegal overweight vehicles that can cause the maximum load effect are included in the records and can provide the unbiased load spectra. Although a WIM data measurement has improved, there is a need to filter the records. There is no widely acceptable guideline for data filtering and this procedure has a major impact on the live load distribution. Different projects resulted with different filtering criteria. It is understandable to remove the unrealistic trucks from the population but no heavy vehicles can be discarded. Based on the sensitivity analysis performed on various sites it was observed that removal of only 0.03% of all trucks from the top of the distribution can cut the maximum load effect by 32%. This can lead to the conclusion that all trucks in the filtered database have a great importance in live load prediction and correct filtering criteria are needed.

Multiple presence and degree of correlation analysis showed that the time of record of the passing truck has to have 0.01 second accuracy otherwise it is difficult to determine the accurate headway distance. A correlation analysis performed on the available new WIM data confirmed previous assumption that about every 500th truck is on the bridge simultaneously side-by-side with another fully correlated truck. However, the expected maximum weight of the fully correlated trucks is smaller than the maximum weight of trucks recorded at the same site. Therefore, a load combination with two fully correlated trucks in adjacent lanes does not govern. The governing combination is a simultaneous occurrence of the extreme truck and an average truck.

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Study of multiple presence probabilities for trucks using recent weigh-in-motion data

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ABSTRACT

An important parameter that controls the maximum load imposed on the structure is related to the number of simultaneous vehicles on the bridge, which is determined through data on truck headways under operating conditions. Accurate headway information should be collected through WIM systems. Simultaneous data on headways and weights is necessary to determine possible correlations between truck positions or the lanes they occupy and their weights or other characteristics such as truck type, size and numbers of axles. Fortunately, the data needed for multiple presence estimates is presently available and already contained in the raw data files captured by many WIM data loggers. Field measurements of truck arrival data to a 0.01 second resolution is necessary to establish multiple presence probabilities for a span.

Multiple presence is influenced by traffic volume and span length. When considering multiple trucks on a given span, a multiple presence event is said to have occurred if the gap between two trucks, that is the distance between the last axle of the leading truck and the first axle of the trailing truck, is less than the span length. Five WIM sites (ten directional sites) with free-flowing traffic in New York State were studied by the NCHRP 12–76 Research Team in order to determine the maximum multiple presence probabilities for various truck traffic volumes. Daily truck traffic volume was classified as light (less than 1000 trucks per day), average (more than 1000 trucks but less than 2500 trucks per day), heavy (more than 2500 trucks but less than 5000 trucks per day), and very heavy (more than 5000 trucks per day). Multiple presence probabilities were compiled for headway separations up to 300 feet, in 20-foot increments. Maximum multiple presence probabilities were obtained for each headway separation interval. These statistics can be used to simulate multiple-presence events for sites where accurate time stamps are not available. The paper will describe the findings of this study. Mr Sivakumar was the Principal Investigator for NCHRP 12–76.

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MENSUSMONITOR – Algorithm implementation for detecting live load events and assessment of structural effects on bridges

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ABSTRACT

Nowadays, Structural Health Monitoring (SHM) is a subject of major interest in the Civil Infrastructures domain. The monitoring systems used in this domain are a powerful tool to verify the structures performance conformity with the advantage to give a valuable database to study in more detail the real behavior of the monitored structures (Sousa et al., 2008a). The most recent monitoring systems allow, in an automatic as well programmable mode, the observation and register of a set of parameters. This is possible by predefining reading procedures that are interpreted by the acquisition systems resulting in a database with the respective sensors readings. The size of those databases can quickly reach considerable dimensions. Considering the flexibility of the current monitoring systems used in civil infrastructures, the readings sample rate can go from 0.00001 Hz to 200 Hz, in depending of the sensor type and/or of the respective acquisition system.

The long-term observation of concrete structures is mainly made by following the structure response to environmental actions and the delay effects of the concrete time dependent behavior, such as shrinkage and creep, with influence in the stress redistribution. For this case the sensors sampling rate is low, typical in the order of 1 sample per hour, and by consequence the volume of data is more easily manageable. However the long-term observation of metallic structures is different, since the dynamic response is crucial to evaluate the fatigue response of these flexible structures. For these structures, higher sampling rates are required, typically in the order of 200 Hz, and the data management in these cases is more demanding. For both cases, the monitoring systems provide a database with great potentialities to evaluate the structural safety. But to have the desired answers in useful time, those databases must be consulted and analyzed with specific tools to extract the desired knowledge.

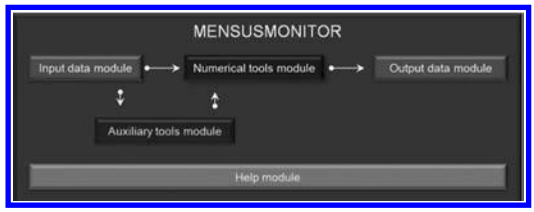

Figure 1. Functioning structure of MENSUSMONITOR.

Furthermore, although the monitoring systems deployed on bridges aim to help both the structural integrity assessment and the management of maintenance/repair operations of large groups of structures, they can also be utilized to characterize the crossing traffic, if properly designed for it. As the bridge is equipped with a kit of sensors, and assuming its behavior remains linear elastic under normal operation conditions, the whole set can act as a balance and therefore measure traffic characteristics (B-WIM).

This paper presents the implementation of an algorithm for detecting live load events and assessment of structural effects on monitored bridges. The events detection algorithm offer a set of results in form of histograms events that characterize several variables distributions like the number of events detected, the amplitudes of observed parameters, or even the level of load applied to the structure. The proposed algorithm is implemented in the MENSUSMONITOR – software specifically devoted to the structural health monitoring. Grounded in the know-how gathered by the research unit FEUP-LABEST in the recent years, MENSUSMONITOR proposes to give faster and efficient answers to the processing and interpretation of records collected by the structural monitoring systems deployed on bridges.

SS27: High performance concrete – lessons of past decades

Organizer: M.L. Ralls

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High performance concrete in Washington state

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ABSTRACT

The Washington State Department of Transportation (WSDOT) was very active in the development of high performance concrete (HPC). WSDOT, as a member of the AASHTO/SHRP Lead States Team, conducted a demonstration project in 1996 through 1998 using HPC to design and construct a three span bridge carrying State Route 18 over State Route 516.

The project presented an opportunity to compare the standard bridge designs with those using HPC. The design comparison proved the economic and long-term benefits of using HPC allowing the number of girder lines to be reduced from seven to five, and realizing a net cost savings of at least $50,000. As a result, WSDOT began using HPC in all its precast, prestressed concrete bridge girders and has used it for an average of 20 bridges per year since 1998. When cost savings are extrapolated to all HPC bridges, significant savings can result. Seven out of ten bridges designed in the last decade by Washington State have utilized precast, prestressed concrete superstructure elements.

HPC technology has also been used in the development of 83 inch and 95 inch deep precast, prestressed concrete "super-girders" for longer spans. This is particularly important with the increasing demand for "rapid construction" (get in, get out, and stay out) and satisfying environmental requirements to keep bridge piers out of wetlands and waterways. WSDOT achieves high economic value from the inherent cost efficiency of precast, prestressed concrete girders constructed with HPC when compared to other alternatives for long spans. WSDOT has developed 95 inch deep prestressed concrete girders capable of spanning 210 feet.

WSDOT has made several changes to its design methodology and specifications to optimize the use of HPC. In addition, HPC is also used in all bridge decks, cast-in-places piles, and deck overlays of latex modified concrete, micorsilica modified concrete, and fly ash modified concrete.

The use of HPC improves construction economy by permitting longer spans, reducing girder lines, and allowing shallow girders that improve vertical clearance. Additional benefits include faster construction, reduced inspection and maintenance times, longer service life and reduced life-cycle cost.

Keywords: High Performance Concrete, Super-girders, rapid construction.

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HPC lessons learned and future directions

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ABSTRACT

The first Strategic Highway Research Program (SHRP) brought needed research into High Performance Concrete (HPC) for Bridges. Once SHRP ended, the Federal Highway Administration (FHWA) was then asked to lead the effort in implementing the research results emanating from SHRP. One of the key efforts of FHWA was working to define HPC in terms of components that could be specified by states, and then working with individual states to build bridges and bridge components with HPC.

Initially only five states partnered with FHWA to try the new HPC technology—Texas, Nebraska, Virginia, New Hampshire, and Washington State. Then more states joined quickly as successful construction of HPC bridges emerged, with some lessons learned, in the first five states. Soon HPC bridge projects were also underway in Colorado, Georgia, Alabama, North Carolina, Ohio, Tennessee, and South Dakota. Additional states followed suit. More recently, in an FHWA survey conducted during 2006–2007, it was found that 16 State DOTs used HPC on up to 10% of their bridges, 19 State DOTs used HPC on 10% to 80% of their bridges, and 15 State DOTs used HPC on more than 80% of their bridges (Triandafilou 2009).

Now, some states are using self-consolidating concrete for certain bridge applications. This type of concrete can be very easy to place and improve the quality of the concrete in areas of dense reinforcing steel. Other states are innovating with Ultra High Performance Concrete in bridge members, taking advantage of very high compressive strengths.

This paper summarizes the initial steps and growth in use of HPC in US bridges, the lessons learned, and the future direction of HPC for bridges.

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HPC implementation in Virginia with lessons learned

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ABSTRACT

This paper discusses the implementation of the high performance concrete (HPC) program in Virginia as it addresses workability, durability, strength, and crack control. The lessons learned during this program are also presented. HPC that has high workability and is self-consolidating was introduced. HPC with low permeability for longevity throughout the structure and high strength in beams was used (Ozyildirim 2005). High strength in beams resulted in fewer beams and smaller beam cross sections. Lightweight HPC (LWHPC) with improved durability and strength was introduced to reduce superstructure and substructure loads for longer span structures, thereby reducing foundation requirements, and to extend the service life. For longevity of reinforced structures, crack control was sought through the selection of ingredients, mixture proportioning, moisture and thermal control, and proper placement and curing (Ozyildirim 2007). Mixtures were sought with less water, cement, and paste for reduced shrinkage and contraction. Wet curing of bridge decks was initiated. Control of thermal cracks in mass concrete was achieved by large amounts of pozzolanic materials and a reduction in cement content. Fiber-reinforced concretes were also investigated to control the severity and width of cracks. Specifications were revised and improved to obtain the desired product. The Virginia Department of Transportation is currently evaluating pilot projects using end-result specifications (ERS) to achieve the desired high-quality HPC.

Virginia's work with HPC has shown that more workable concretes can be obtained that facilitate placement and consolidation and provide a smooth surface finish. The lower permeability essential for durability can be achieved with pozzolans or slag cement alone or in combination at a moderate (not low) water-cementitious material ratio (Lane & Ozyildirim 2000). Pozzolans or slag cement also provide resistance to chemical attack and control heat of hydration. Attention to air-void parameters rather than the total air content is needed (Ozyildirim 2004). High strength concrete with normal or lightweight aggregates and with large 0.6-in strands can be used successfully to reduce the number of beams and allow longer spans. For longer spans, LWHPC should be considered to reduce the superstructure and substructure dead loads and foundation sizes. Cracks adversely affect durability, and can be minimized through proper selection of ingredients; mixture proportioning by minimizing the water, cement, and paste content; moisture and thermal control; adequate consolidation and curing; and addition of fibers to help to control length and width of cracks. More representative test procedures such as the use of temperature-matched curing are needed. Specifying performance parameters as in ERS is expected to lead to improved HPC.

HPC implementation is expected to provide longer service life, cost savings, reduced construction time, and lower maintenance. HPC has a high potential for improved service; however, care should be exercised and adequate QC/QA measures used in its production to ensure that these benefits can be achieved.

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Status of Texas DOT HPC implementation

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ABSTRACT

The Texas Department of Transportation (TxDOT) currently has over 33,000 bridges on the state system that it has responsibility to operate and maintain. Over 60 percent of the bridges were constructed at least 40 years ago. Thus, it can be seen that a long service life is required by the inventory of TxDOT bridges. If one percent of the bridges are replaced per year, the estimated required service life would be approximately 100 years. The concrete materials used 40 years ago typically will not provide this expected service life and thus significant maintenance may be required to keep the system operational. This paper discusses the implementation of obtaining improved concrete specifications to obtain high performing concrete structures.

The significant concrete distress mechanisms are presented and what is being done to specify concrete to counter them. The distress mechanisms include Alkali-Silica Reactivity (ASR), sulfate attack, freeze-thaw cycling, and reinforcing steel corrosion. Substituting supplementary cementitious materials (SCMs), including fly ash, ground blast furnace slag, and silica fume, for a portion of cement in the concrete mix is discussed in relation to each distress mechanism.

The occurrence of premature concrete deterioration attributed to ASR has been observed for over 15 years in TxDOT structures. Because the majority of the aggregates concrete producers use throughout the state can be classified as potentially reactive with respect to ASR, instead of prohibiting their use, concrete mixtures have been designed to mitigate the harmful effects associated with using these aggregates. The primary mitigation option that appears to be the most cost effective and fail safe is limiting the maximum quantity of cement and including a substantial amount of SCMs in the mix. Both sulfate attack and freeze-thaw cycling, though significant concrete distress mechanisms, appear to be adequately mitigated by current and former practices. These practices are presented along with the subsequent issues that have arisen when combined with specifying high performance concrete (HPC) is discussed. The most widespread distress mechanism TxDOT addresses by specifying HPC is concrete structure deterioration caused by reinforcing steel corrosion.

TxDOT first used HPC in the early 1990's on two projects where high strength concrete with an emphasis on low permeability for increased durability was specified (Ralls, M.L., Carrasquillo, R.L., & Burns, N.H. 1998). The evolution of methodology for specifying HPC is presented along with discussion on the suitability of performance testing for concrete permeability (Cox, W.R. & Pruski, K.R. 2003). It is conveyed that the use of SCMs are beneficial to produce HPC including results from research investigating concrete containing fly ash is presented. In addition to addressing improved concrete materials, measures including prefabrication and improved construction practices beneficial to constructing high performing concrete structures (Pruski, K.R. et al, 2002) are discussed.

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High performance concrete's evolution in NH

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ABSTRACT

The New Hampshire Department of Transportation (NHDOT) now uses a performance based, i.e., quality control/quality assurance (QC/QA), specification to consistently construct durable concrete bridge decks. This specification evolved from NHDOT's participation as a lead state in the Federal Highway Administration's (FHWA) High Performance Concrete (HPC) Bridge Showcase Program, undertaken from 1996 through 2000. NH's first HPC bridge, in Bristol, NH, was constructed in 1996. Development of the 8000 psi girder strength showed that too much air content can make strength gain difficult. However, such strengths make 12′–6″ girder spacings achievable. NH also learned that wide girder spacings have the undesired consequence of increasing deck falsework costs.

NH's second HPC bridge, also in Bristol, NH was constructed in 1999. With this project, the deck falsework costs were mitigated by the use of stay-in-place partial depth concrete panels spanning the girders.

NH's third HPC bridge, located in Rollinsford, NH, uses experimental carbon fiber reinforced polymer, instead of steel reinforcing steel, as the deck reinforcing. This is the first of NH's HPC projects to use a Quality Control/Quality Assurance (QC/QA) specification rather than a prescriptive concrete specification for the deck.

Today, NH's QC/QA specification includes pay factors that consider strength, air content, water/cement ratio, permeability, and concrete cover over the reinforcement. Contractor's can earn up to 7% of the concrete bid price for exceptional in place concrete. QC/QA concrete decks are now standard practice in NH.

Although at the time of this writing, NH's three HPC bridges are, at most, 15 years old, all decks and girders are still performing well.

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SS28: Construction, architecture & testing of soil-steel bridges

Organizers: Z. Manko & D. Beben

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Dynamic analysis of soil-steel arch road bridges

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ABSTRACT

The paper is presented the theoretical consideration of soil-steel objects (Gimån bridge and Wroclaw bridge – Fig. 1) during dynamic field load tests. The critical speed magnitudes, dynamic coefficients (DAF), velocity vibration, vibration frequency were determined in the paper.

Loads moving across the bridge at high speeds cause vibration of the bridge structure, impact as a result of rough road surface and unequal deflections of the vehicle springs, etc. Dynamic reaction stress exerted on elements of the bridge structure as well as the strains caused by mobile rolling stock become bigger than similar static loads; i.e. slow placement of immobile rolling stock of the same or similar weights. The exact calculation of the bridge dynamics considering all the above mentioned factors, the free vibration of the structure and also possibilities of resonance of the bridge require exceptionally complicated calculations which do not yet yield an absolute satisfactory solution.

Generally, the dynamic coefficient value is related to the so-called critical speed of the truck and the value of the largest vibration amplitude that occurs. This velocity can be calculated using many tests (movement of the same load at different velocities across the same bridge). The critical speed is defined as a speed during which the value of dynamic coefficient is maximum.

During the dynamic tests, a truck was used. The weighed front and rear axle loads together considerably exceeded the total weight of the truck (with its load included) of circa 300 kN. The speeds of the moving truck over the bridges (used earlier in the static tests were estimated as follows 5, 10, 20, 30, 40, 50, 60 and 70 km/h in both directions. Measurements of dynamic interactions were taken also when the truck was moving by threshold of 0.03×0.20 m fixed at half way of the bridge distance perpendicular to the longitudinal axis of the roadway and during its braking with various different velocities at different distant points of the bridge.

Figure 1. Wroclaw Bridge during passing the Kamaz vehicle on the object with speed of 30 km/h.

For the dynamic tests, two inertial inductive sensors PEVA 7225 type were fixed at the edge of roadway (or sidewalk) and reinforced concrete collar and strain gages for strains measurements in the transversal and longitudinal directions.

Based on strains at characteristic points and cross sections of the span, as well as deflections obtained at three different points of shell structure the dynamic coefficient values (DAF) φ were determined for all the dynamic load variances I–XXIV (Giman Bridge) and I–VI (Wroclaw Bridge). On their basis, among other things, the critical speed has been determined on the level $v_{cr.} = 60$ km/h. It was also noticed that the magnitude of the DAF were lower in comparison to values calculated in accordance with the Polish Bridge Load Standard of PN-85/S-10030 (1985) (12% & 23%), the Eurocode (2002) (15% & 16%) and the BV Bro (2004) regulation (19% & 25%) for the Giman Bridge and the Wroclaw Bridge respectively. It should be clearly mentioned that the calculated dynamic coefficients (DAF) were adopted just as the traditional road steel bridges (which in the experimental tests the values were obtained mainly much lower), whereas standard regulations in this range (in Poland as well as Sweden) do not yet relate to the such solutions in a design of the new structures.

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Dynamic testing of a soil-steel composite railway bridge

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ABSTRACT

The dynamic response of a long-span arch soil-steel composite railway bridge is studied. The bridge has a span of 11 m and a rise of 4.3 m (see Figure 1 and Figure 2). Strains, displacements and vertical ballast accelerations were measured during passages of a locomotive at different speeds. The results indicate that the speed of the locomotive has a large influence on the displacements, thrusts and moments (see Figure 3). The structure was found to be safe when measured values of moments and thrusts were compared with the live load calculations according to design codes. However, dynamic amplification factors as high as 1.45 were obtained for the moments at the quarter point and this is found to be much greater than the values specified in bridge design codes. Despite this, due to the high damping involved, bridges of this kind are believed to be less sensitive to resonance problems from passing trains.

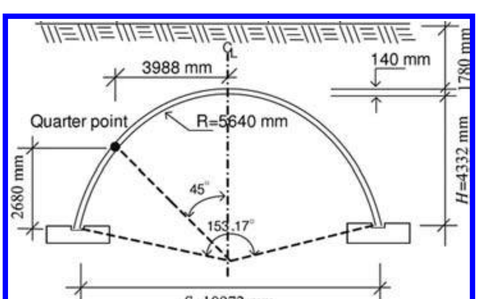

Figure 1. Sectional dimensions of the culvert.

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Figure 2. The construction of the steel culvert bridge. This new bridge was placed on top of an existing masonry arch bridge with a gap in between, which leaves the old bridge structurally functionless.

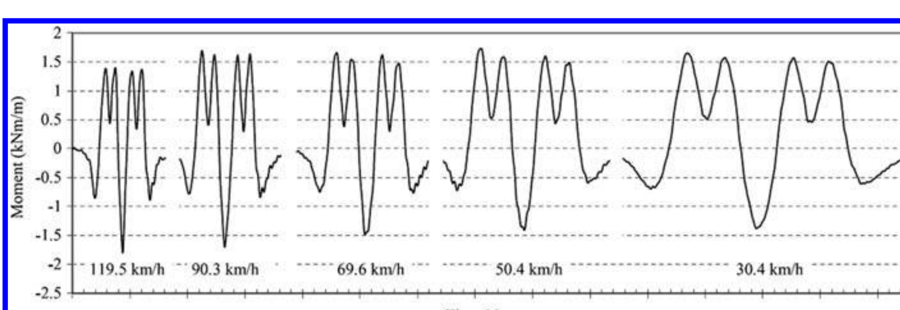

Figure 3. Measured moments at the crown centreline for different locomotive speeds.

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Rehabilitation of old arch bridges using corrugated shell structures

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ABSTRACT

There are many engineering structures located on all kinds of roads and railways in Europe which needs to be urgently upgraded or rebuilt or even replaced. A common result of degradation, caused by the destructive action of natural forces (e.g. floods), improper use or belated repairs, of bridges is a considerable decrease in their load-carrying capacity. This creates serious dangers to road and railway users. Therefore the bridges should be quickly and effectively repaired. Effective solutions entailing minimum costs, including social costs resulting from the traffic delays and road or railway closures, should be sought.

Small and medium-sized bridges and culverts can be modernized in many different ways. For example, the damaged bridges can be rebuilt or reinforced or even replaced on a new structure, however the cost of the last solution is very expensive. Often the flexible structures made from the corrugated steel plates (CSP), with different corrugations dimensions (e.g. for spirally pipes: 68×12, 100×20, 125×26 mm; and for structures made from plate sheets: 70×13, 100×22, 150×50, 152×51, 200×55, 380×140, 400×150 mm) are used as a good alternative for the traditional bridge solution.

Different shell shapes in the road's longitudinal direction – arch, box, circular (pipe and elliptical) – suited to the kind and size of the obstacles and the soil and hydrologic conditions are also used for this purpose. The advantages of this solution include: the short time (a few days only) needed to assemble the steel shell structure, the simple design, the relatively light deadweight (the structure can be founded on weak ground), the long life (often exceeding 80–100 years) and aesthetic values. Upgrading of engineering structures by means of corrugated-plate structures has been known and quite commonly employed on almost all continents (mainly in Canada and USA and in the Scandinavian countries) for many years. The shell structures made from corrugated steel plates were first used in USA at the beginning of the 20th Century.

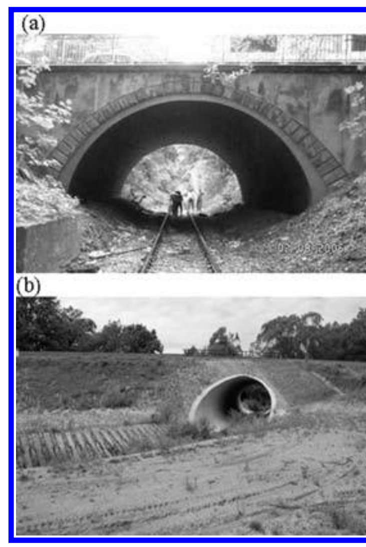

Figure 1. Side view on bridge after repair by using the CSP: (a) over railway in Augsburg (Germany), (b) under railway in Piensk (Poland).

Several successful projects of this kind in Poland have made such solutions quite popular among the Polish designers who use them to repair and reinforce small bridges and culverts. However, the actual design methods of such flexible structures in Europe are too simplified because the rational calculation methods are complicated and the advanced numerical model should be applied.

The construction works can be carried out very quickly and efficiently (about 5–10 weeks). The presented structural solutions are cheaper (on average 30–40%) than different methods of bridge repairs.

The examples of modernization of old road and railway viaducts (and culverts) in Poland and Germany by use of shell structure made from corrugated steel plates are showed in Figure 1.

SS29: Lessons learned from instrumented bridges

Organizer: M.Q. Feng

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Complementing long term health monitoring of bridges using numerical models

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ABSTRACT

The increasing trend for bridge owners to adopt long term health monitoring strategies for bridges under their management, using measured vibration responses, brings with it a need for associated numerical modeling to complement measured data sets to explain response trends observed.

The West Street On-Ramp Bridge, constructed in 2000, is a three span post-tensioned reinforced concrete, horizontally curved, over-bridge on the Santa Ana Freeway (I-5) in the city of Anaheim, California, Figure 1.

The acceleration response at eleven sensor locations, monitored since 2002, indicate a 5% decrease in the lower frequencies of vibration but the density of sensors is not sufficient to identify the cause using measured data alone. A three dimensional finite element model, (Fig. 2), was used to complement the monitoring programme.

Figure 1. West street on-ramp.

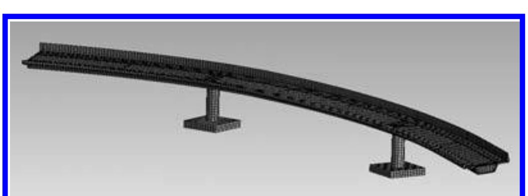

Figure 2. Shell/solid FE model.

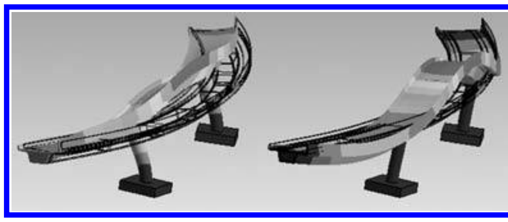

Figure 3. Modes shapes 1 & 2.

Table 1. Frequencies & model parameters.

Data Set Year	Measured 2002	Measured 2009	3D FE 2002	3D FE 2009
E_{deck} (MPa)	–	–	20100	20300
E_{bents} (MPa)	–	–	28300	22200
F_1 (Hz)	2.05	1.90	2.04	1.89
F_2 (Hz)	2.44	2.34	2.71	2.56
F_3 (Hz)	2.83	2.64	2.83	2.64

This 3D shell/solid model enabled assignment of boundary conditions consistent with construction details. Model input parameters, material properties and support stiffnesses, were varied to construct a model response space for the first three natural frequencies. Searches in the response space identified model parameters for optimum correlation with the measured frequencies. The first and second mode shapes are illustrated in Figure 3, while the model parameters found for correlation with 2002 and 2009 measurement data sets are listed in Table 1.

The Young's Moduli for the deck and bents for correlation with year 2000 frequencies were approximately 80% of values proposed by ACI based on their respective concrete strengths. The reduction in frequency over the period to 2009 is presently attributed to a reduction in the effective stiffness of the bents – this is the subject of ongoing study as is the effect of post-tensioning stresses on the various modes of vibration.

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Long-term bridge performance monitoring program in California

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ABSTRACT

The Division of Engineering Services at the California Department of Transportation (Caltrans) started to instrument bridges in 1999 for their long term performance monitoring and evaluation. As a structure deteriorates due to aging or suffers from damage caused by extreme loads such as earthquakes, stiffness of the damaged structural elements would decrease, and as a result, the global vibration characteristics of the structure would change. Therefore, by monitoring the structural vibration, one can identify the change in structural vibration characteristics and then further evaluate the change in the element stiffness.

Three new concrete bridges (Fig. 1) in the Orange County were instrumented with accelerometers, strain gauges, displacement sensors and pressure gauges at strategic locations. Traffic-induced vibration data are periodically collected locally or remotely through wireless Internet. They are processed with a variety of signal processing and system identification software tools to extract structural vibration characteristics and to further update their finite element analysis models. All the data were then stored in the developed database for statistic analysis.

A number of system identification methods were reviewed and developed in this study for identifying the structural element stiffness based on measurement of bridge vibrations caused by traffic and seismic excitations. A unique traffic excitation model was proposed for more reliable stiffness identification based on traffic-induced vibrations. The effectiveness of these methods in evaluating seismic damage on a bridge structure was demonstrated through seismic shaking table tests of a multi-bent multi-column concrete bridge model. Long-term monitoring data from the instrumented bridges were analyzed and developed into a structural stiffness database using a software platform developed in this study.

This project has demonstrated the potential of using the sensor technology for long term and real-time structural health monitoring and post-event damage detection.

The baseline updating methods based on vibration measurement and stiffness identification developed in this study can also be applied to existing bridges (that represent the majority of the Caltrans inventory) in the following two ways; one is for establishing the current baseline of the bridge for its future damage detection and deterioration assessment, and the other is for assessing the ongoing "health" if a database of similar types of bridges exist for the comparison purposes.

Figure 1. Instrumented bridges in California.

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Performance assessment of Jiangyin Bridge using long-term structural health monitoring data

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ABSTRACT

The Jiangyin Bridge is a suspension bridge with 1385 m main span, which crosses the Yangtze River in Jiangsu Province, China. It is the first bridge with a main span exceeding 1 km in Chinese mainland, and is also among few bridges in Chinese mainland with structural health monitoring (SHM) systems being installed in the end of last century. After operation for several years, the SHM system was found with malfunction in sensors and data acquisition units (DAUs), and with insufficient sensors to provide necessary information for structural health evaluation. The SHM system has been upgraded by a consortium of The Hong Kong Polytechnic University and Jiangsu Transportation Research Institute by repairing/replacing the disordered sensors and installing new sensors, renewing the data acquisition and transmission system, and developing a new structural health evaluation software system. The upgraded SHM system comprises over 170 sensors, including anemometers, GPS, displacement transducers, fiber optic temperature and strain sensors, and accelerometers. The system commenced operation in June 2005; since then structural health monitoring data have been continuously collected, including those obtained during the typhoon "Matsa" and one ship collision accident.

Making use of the long-term monitoring data, performance assessment is carried out for the bridge by means of multi-dimensional data analysis. This paper reports the performance assessment with emphasis on: (i) analysis of temperature profile and effects; (ii) recognition of wind characteristics and effects; and (iii) characterization of structural dynamic properties. First, the temperature profile and its effect on the movement of expansion joint and the variation of strain are analyzed. The deck effective temperature is evaluated from the temperatures in a deck cross-section to formulate the temperature profile. Upon this, the thermal movement of expansion joint is studied and the displacement-temperature correlation model is formulated for design verification and extreme value prediction. Another temperature effect considered is the temperature-induced strain variation. A linear relationship between strain and temperature is observed and the strain-temperature correlation model has been established for quantitative estimation of the thermal strain. Then, the wind characteristics specific to the Jiangyin Bridge site and the wind-induced lateral displacement of the deck are investigated. The wind data are analyzed to obtain: (i) mean wind speed and direction, (ii) wind speed profiles, (iii) turbulence intensity, (iv) gust factor, and (v) wind spectrum. Verifications of design assumptions and rules of thumb are made, and the wind-induced lateral displacement of the deck is explored. A linear relationship between transverse wind speed and deck lateral displacement is observed and the correlation model between them has been established for quantitative estimation of the wind-induced displacement. Finally, the modal properties of the Jiangyin Bridge are studied. Modal properties under normal conditions, including modal frequency and mode shape, have been identified from the acceleration data. Analysis of modal properties under extreme events, such as typhoon, ship collision, and subsequent damage identification is ongoing.

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SS30: Chinese bridges

Organizer: M.-C. Tang

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Major steel bridges for high speed railway in China

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ABSTRACT

Major steel bridges for high speed railway in China are recently developed. Some of these bridges, located Beijng-Shanghai and Beijing-Guangzhou High Speed Railways, across Yangtze River and Yellow River, are under construction, include: Tianxingzhou Bridge, Dashenguan Bridge (Fig. 1), Zhengzhou Yellow River Bridge, and Jinan Yellow River Bridge. All these major bridges are multy railway lines or rail-cum-road bridges, with heavy loading capacity and high railway operation speeds, therefore, the steel truss structure are adopted for all of them. Some new materials, new structures and new workmanships are applied accordingly.

Bridges are important structure to high speed railway and railway passenger special line, have main impact on stability of high speed train operation, as well as comfort and safety of passengers. Currently, Beijing-Shanghai and Beijing-Guagzhou High Speed Railways are under construction. They have total 4 major bridges across Yangtze River or Huanghe River, namely Wuhan Tianxingzhou Rail-Cum-Road Yangtze River Bridge, Nanjing Dashenguan Yangtze River Bridge, Zhengzhou Yellow River Rail-Cum-Road Bridge, and Jinan Yellow River Bridge of Beijing-Shanghai High Speed Railway. For all the above mentioned bridges, steel truss has been used with its good rigidity. Some new materials, new strutures and new workmanships have been applied.

Figure 1. Dashenguan Bridge.

Table 1. Objective Desig Speeds.

Project	Tianxing zhou	Dashen guan	Zhengzhou Yellow River	Jinan Yellow River
Objective Design Speeds (km/h)	200	300	350	350

Objective design speeds for both Beijing-Shanghai High Speed Railway and Beijing-Guangzhou Railway Passenger Special Line are 350km/h. According to the traffic conditions at the bridge location, and the structure features, the objective design speeds of 4 bridges above mentioned refer to Table 1:

These bridges have heavy self-weight load, and most of them are taking several railway lines or carriageways, so the live loads are considerable great also. Tianxingzhou Bridge undertake 4 railway linesand 6 carriageways, Dashenguan Bridge 4 railway lines plus 2 metro lines, both of them are the bridges with the largest design loads in the world. Please refer to Table 2 for these bridges span length and design live loads.

According to thelarge span, high Speed and heavy load features, some new techniques are adopted in the bridge design.

Table 2. Bridge Spans and Design Live Loads.

Bridge	Tianxingzhou Bridge	Dashenguan Bridge	Zhengzhou Yellow River Bridge	Jinan Yellow River Bridge
Span(m)	504	2×336	5×168	3×168
Railway lines	2 passenger special and 2 Class I railway	2 high speed railway and 2 Class I railway	2 passenger special	2 high speed railway and 2 passenger special
Highway lanes	6	/	6	/
Metro lines	/	2	/	/
Live loads (t/m)	35.1	34.7	19.1	25.6

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Major bridges in Shanghai

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ABSTRACT

Shanghai was the first city featuring major long span bridges in China. The design and construction techniques for long span bridge have undergone significant development and improvement over the past several decades.

Bridge construction in the city has kept apace with a series of impressive achievements. Among these, some major bridges have attracted particular attention. The construction of the Nanpu Bridge in 1988 is regarded as the beginning of modern long span cable-stayed bridge construction in China, doubling the largest span length in the country at that time. The 602 m span Yangpu Bridge was under construction right after and became the world's longest span cable-stayed bridge when it was open to traffic in 1994. Since then, design and construction technology for cable-stayed bridges have been continuously developed and improved in China. Shanghai's Yangtze River Bridge (Fig. 1) and Minpu Bridge are two long span cable-stayed bridges that incorporate particularly innovative techniques. Besides Lupu Bridge (Fig. 2) was built and came into operation in 2003 as the world's longest arch span at that time.

Shanghai's five cable-stayed bridges and Lupu arch bridge are described in this paper (Tab. 1), with special attention devoted to experience gained during construction, structural characteristics, technological developments, and innovative techniques.

Through the construction of these bridges, the design and construction techniques of major bridges have been developed. Compared with other types of structure, the modern cable-stayed bridge has a relatively short history, while building long span arch bridge is more difficult. Therefore, many problems related to design methods and construction technology require further study and improvement. Shanghai will continue to contribute in the development of major bridges.

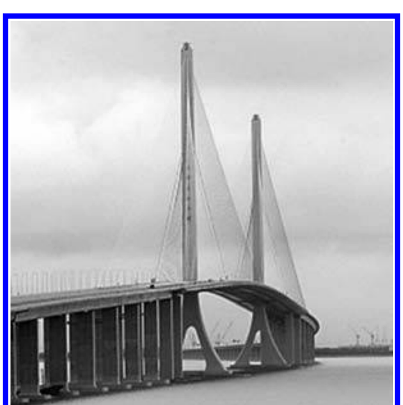

Figure 1. Shanghai Yangtze River Bridge.

Figure 2. Lupu Arch Bridge.

Table 1. Major bridges in Shanghai.

Bridge Name	Built Year	Span (m)	Bridge Type
Nanpu	1991	423	Cable-Stayed
Yangpu	1993	602	Cable-Stayed
Xupu	1997	590	Cable-Stayed
Shanghai Yangtze River	2009	730	Cable-Stayed
Minpu	2010	708	Cable-Stayed
Lupu	2003	550	Steel Box Arch

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Bridges in Chongqing – the bridge capital of China

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ABSTRACT

Being bisected by many large rivers, most notably the Yangtze River and the Jialing River, the city of Chongqing needs a lot of major bridges to accommodate its ever growing traffic volume. The city has been very progressive in accepting new ideas and innovative bridge concepts. The city now has over 50 major bridges, including the world's longest box girder bridge, and the world's largest arch span.

Major bridges built or under constructionm in the last twelve years were summarized in Table 1. All fourteen bridges in the table have span length larger than 250 m. It's an exhibit of modern bridge types, innovative construction technology and engineering creation. It wasn't surprising that the mountain city is now recognized as *the Bridge Capital of China*.

Table 1. Major bridges in Chongqing since 1997.

Bridge Name	Built Year	Span (m)	Bridge Type
Lijiatuo	1997	444	Cable-Stayed
Huanghuayuan	1999	250	Concrete Girder
Egonngyan	2000	600	Suspension
Wushan	2005	460	Steel Truss Arch
Shibanpo	2006	330	Hybrid Girder
Caiyuanba	2007	420	Steel Box Arch
Jiahua	2007	252	Concrete Girder
Wujiang	2008	340	Cable-Stayed
Yudong	2008	260	Concrete Girder
Chaotianmen	2009	552	Steel Truss Arch
Jiayue	Under Constr.	250	Cable-Stayed
Shuangbei	Under Constr.	330	Cable-Stayed
Qiansimen	Under Constr.	315	Cable-Stayed
Dongshuimen	Under Constr.	445	Cable-Stayed

Figure 1. Shibanpo Bridge – Hybrid with concrete and steel box.

Figure 2. Renderings of the Tied-Arch Caiyuanba Bridge.

Figure 3. Renderings of Chaotianmen Bridge.

Figure 4. Renderings of Dongshuimen Bridge.

In this paper, five major bridges recently built or under construction in the metropolitan area of city are introduced, including Shibanpo Brdige (Fig. 1), Caiyuanba Bridge (Fig. 2), Chaotianmen Bridge (Fig. 3), Qiansimen Bridge and Dongshuimen Bridge (Fig. 4).

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Cross-river bridges in Jiangsu

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ABSTRACT

Jiangsu Province is located in Yangtze River Delta, the mostly developed area in China. It is also the T-junction intersection of coastal areas and the Yangtze River adjacent to Shanghai, Zhejiang, Shandong and other provinces with busy land and water traffic. However, the separation caused by the Yangtze River obstructed the cross-coast exchange and development. Therefore, bridges constructed over the Yangtze River had been the dream of people living on both banks of the river. Since Jiangsu Province is located in the middle and lower reaches of Yangtze River, the huge water width and depth as well as the geological complexity make it extremely difficult to construct cross-river bridges over there. Until 1968, Nanjing Yangtze River Bridge, the first cross-river bridge along the water section of 450 kilometers in Jiangsu Province, was opened to traffic. In 1980s, with the development of social economy and the improvement of technology, Jiangsu started to conduct researches and planning for the construction of cross-river channels over Yangtze River. Therein, 12 positions were selected for the construction of bridges or tunnels (Fig. 1).

This article briefly introduces the construction of cross-river bridges in Jiangsu Province(Tab. 1).

In 2009, the State approved the general plan of Jiangsu coastal development. To help northern Jiangsu preferably blend into the Yangtze River Delta and join the Metropolitan Area of Shanghai, more cross-river bridges are needed. Recently, Jiangsu Province has further adjusted planning of cross-river passages,

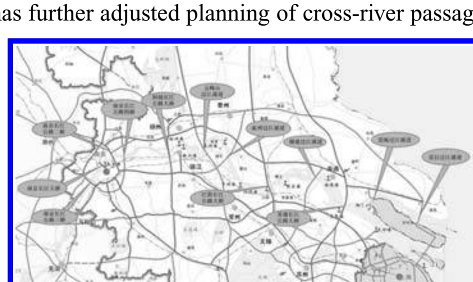

Figure 1. Cross-river Channels Plan for Express Ways in Jiangsu Province.

Table 1. Major Cross-river bridges in Jiangsu Province.

Bridge Name	Built Year	Span (m)	Bridge Type
Jiangyin	1999	1385	Suspension
Runyang	2005	1490	Suspension
Taizhou(Fig.2)	Under Constr	1080 × 2	Suspension
Nanjing 4th	Under Constr	1418	Suspension
Nanjing 2nd	2001	628	Cable-Stayed
Nanjing 3rd	2005	648	Cable-Stayed
Sutong(Fig.1)	2008	1088	Cable-Stayed
Nanjing Yangtze River	1968	160	Steel Truss Girder
Dashengguan	2010	336	Steel Truss Arch
Chongqi	Under Constr	185	Concrete Girder

Figure 2. Taizhou Bridge.

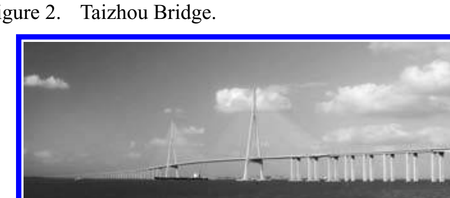

Figure 3. Sutong Bridge.

confirming that there are 24 cross-river passages in Jiangsu Province, 11 of which are integral parts of cross-river passages of Jiangsu Highway. Compared to first planned location, the geological, hydrological and navigation conditions of newly-planned bridge location are more complicated and the project construction is more difficult. We will make use of our own and domestic and overseas advanced experience, go on with the path of development and innovation, and make contribution to the development of bridges in our country.

General Session

Organizers: F. Biondini & S. Pakzad

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)
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Experimental study on repair welding under static and cyclic loads

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ABSTRACT

Plenty of bridges have been damaged such as fatigue crack, corrosion and deformation due to degradation of the bridges, unexpected increase of traffic volume and variation of traffic environment.

So, works such as replacement, improvement, repair and reinforcement are necessary. Considering the economical and social situation in Japan, the most suitable work has to be selected among them. Fixed structures such as bridge are required advanced technology that is suited for their situation.

As the repair/reinforcement work of steel bridges, the splice bolted joints are mainly used in a site. On the other hand, welding is not so general. So, the development of welding technique has been required one of the selective works.

At this moment, welding is performed by removing the loads through the setting up the bents or traffic control. But, welding under loads without traffic control is desirable.

The load conditions of bridges can be classified three types: no load, static load, fluctuated load.

Fitting the member and avoiding the defects are required in site welding.

In case of welding under no load and static load, it is possible to prevent of cold cracks and deformation, using suitable welding conditions. But, in case of welding under fluctuated load, it is impossible to prevent hot cracks. Hot crack may bring about the collapse of the structure.

About twenty years ago, a judging method of advisability of welding under fluctuated loads was proposed. And it applies for some bridges. But, it is not widely used by reason, it can be said that the suitable range of welding for avoiding the hot crack is extremely narrow.

In order to investigate the possibility of repair by CO_2 arc welding, welding under static and cyclic loads were performed.

As the results, cracks were not observed at the cruciform joints. However, cracks were observed at the

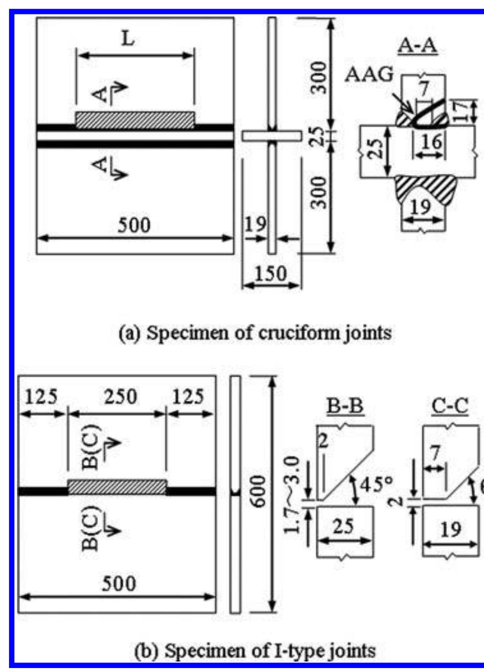

Figure 1. Schematic illustration of specimen.

I-type joints only in welding under cyclic loads. From the results of a fractography, the confirmed patterns of fracture were projections of cellular dendrite and striations. And it could be known that the magnitude of $\Delta\delta$ influenced hot crack.

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Long-term performance prediction of RC bridge slabs in a marine environment

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ABSTRACT

Unlike the case associated with hazards that are usually considered in structural design (e.g., seismic hazard), there has been a lack of research on marine environmental hazard assessment. Akiyama et al. (2010) established a methodology for the probabilistic hazard assessment associated with airborne chloride. Figure 1 shows an example of hazard curve associated with two different distances *d* from the coastline. In this paper, the procedure to integrate the hazard associated with airborne chlorides into time-dependent structural reliability assessment of RC bridge slabs is provided.

Failure probability of RC structures in an aggressive environment depends on many aleatoric and epistemic uncertainties involved in the evaluation of hazard assessment of airborne chlorides, corrosion process of steel bars, and deterioration of structural performance (Akiyama et al. 2009). For existing structures it is possible to reduce epistemic uncertainties using inspection results (Frangopol 2009). The relationships between observed physical quantities such as inspection results, and the PDFs of related random variables are used in Bayesian updating. When nonlinear relations or non-Gaussian variables are involved, an approximate solution can be found by using several approaches. Monte Carlo Simulation (MCS) is in general used because of its versatility. MCS-based methods for non-linear filtering technique have been developed since 1990s.

The term Sequential MCS (SMCS) is used in conjunction with time-dependent reliability assessment. This paper follows the procedure of SMCS applied to reliability analysis proposed by Yoshida (2009). Visual inspections of corrosion crack width and chloride concentration distributions by coring test are used as observational information. Using SMCS, multiple random variables related to observation information can be updated simultaneously.

The effects of marine environment, inspection results and the number of inspections on the updated estimates of RC bridge slab reliability are discussed.

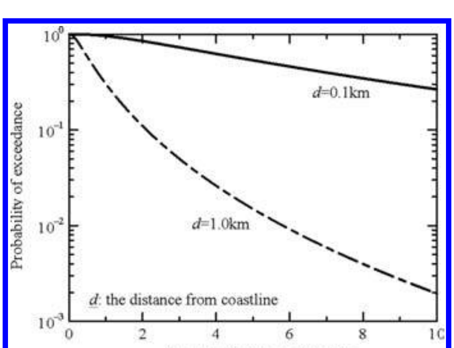

Figure 1. Probability of exceedance of various amounts of air-borne chlorides in Wakkanai City at distance of 0.1 km and 1.0 km from coastline.

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Arrigoni Bridge inspection and testing program

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ABSTRACT

The Arrigoni Bridge is a thirty (30) span structure including two prominent tied steel thru truss arches carrying Route 66 over Route 9, the Providence and Worcester Railroad and the Connecticut River between Middletown and Portland, Connecticut. In 2007, AI Engineers, Inc. (AI) of Middletown, Connecticut was tasked by the Connecticut Department of Transportation to perform a regular bi-annual inspection of the structure. However, in addition to this inspection, AI was also tasked with the ultrasonic testing (UT) of the 14 inch diameter steel pins in the bottom chords and electromagnetic inspection of the steel hanger cables in the main spans both of which had never been performed.

The paper outlines the plan that AI and the various non-destructive and equipment subconsultants used to perform the routine visual inspection and the new UT pin and electromagnetic cable inspection of this prominent structure. Topics discussed also include equipment manifests required to visually inspect the large trusses and an agreed upon procedure for the maintenance and protection of traffic for this 4 lane, heavily travelled structure.

Results from both the NBIS as well as the cable and pin inspections are also presented.

Figure 1. Arrigoni Bridge, Middletown, CT.

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On the applicability of random field theory to transportation network analysis

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ABSTRACT

In the last years, a significant scientific effort has been put in research regarding civil infrastructural networks and, in particular, transportation networks (e.g. Akgül & Frangopol 2003, Shinozuka et al. 2006, Dueñas-Osorio et al. 2007). In fact, lifelines have a role of paramount importance in post-emergency response and in the recovery activities when a natural (e.g. earthquake, hurricane, fire, flood, landslide) or man-made (e.g. explosion, terrorist attack, vehicular collision) extreme event strikes.

In sampling based performance analyses, the damage can be modeled as a random field. Random field theory (e.g. Bocchini & Deodatis 2008) can be proficiently used also to perform efficient parametric and sensitivity analyses that give useful information on the relative importance of the network elements. In these cases, the underlying assumption is that the network performance indicators are more sensitive to the spatial correlation of the damage than to the actual intensity of the demand at the locations of the various lifeline components. In fact, the redundancy of the network, its ability to redistribute traffic flows, the possibility of cascading failures and the presence of non-redundant links create a strong interdependence among the various components. Therefore, it is often reasonable to assume that the overall performance of a transportation network after an extreme event does not significantly depend on local effects of the hazard sources. The previously mentioned underlying assumption is addressed to estimate its applicability. In particular, a sensitivity analysis is conducted to evaluate the variation of the network performance indicator as a function of the spatial correlation of the damage. Figure 1 shows the strong dependence of the performance of a sample transportation network on the correlation length of the structural damage of its bridges.

In this paper an integrated analysis framework, including the structural analysis, the network performance analysis (based on Evans 1976) and the statistical result analysis of transportation networks, is presented. The same framework can be used for many further investigations, such as network maintenance optimization, development planning, loss estimation and network reliability assessment.

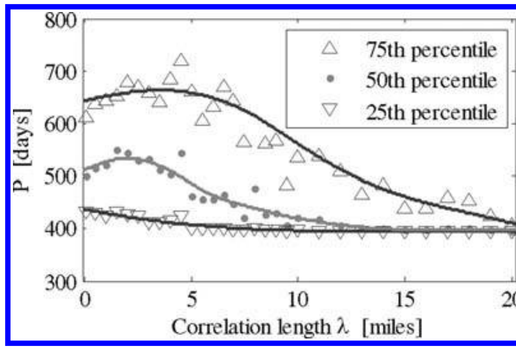

Figure 1. Percentiles of the performance indicator P as a function of the correlation length of the bridge structural damage.

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Design check of bridge over Tejo River in Carregado, Portugal

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ABSTRACT

This paper presents the independent design check performed for the bridge over Tejo river in Carregado, Portugal. The scope of the work involves geological and geotechnical analysis of foundations for the bridge and respective viaducts as well as the structural ans seismic analysis for each construction. This paper only deals with subjects that refer to the structural design check, mainly structural conception and design.

Three different structural solutions were used for bridge and respective viaducts. For the northern viaducts an "in-situ" concrete slab with beams was used with column-pile foundations. The total length of these three viaducts is 1473 m. The southern viaducts have a total length of 9230 m and are divided into 22 viaducts. The structural solution is prefabricated U beams with an "in-situ" slab. The final solution for each viaduct is monolithic with the column-pile foundations. Length of viaducts ranges from 250 to 530 m. The bridge has 970 m length with 7 columns. Internal spans have 130 m while the end spans have 95 m. The structural solution is a reinforced concrete box that support through steel struts the concrete flanges.

The design check consisted in an analysis of the several written and drawn elements of the Project and also a structural analysis of southern viaducts. Also some particular structural analysis, mainly related with the construction process, was carried out during the duration of the works.

The design review contributed decisively to the success of the work, in particular regarding: the development of faster, simple and economic constructive solutions; the improvement of solutions used for critical points where uncertainty is bigger; the optimization of the constructive process; the analysis of the behavior of the structure during the constructive process. The main conclusions of the design check mainly regarding its importance for the success of the overall process are highlighted in this paper, especially regarding the development of faster construction speed, simplicity and economy, but also regarding improvements of solutions used in some key points.

Figure 1. Aerial view of the bridge.

Figure 2. Bridge during construction.

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Using microsimulation to estimate highway bridge traffic load

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ABSTRACT

In bridge traffic loading there is an often-made assumption that free-flowing traffic incorporating dynamic effects governs for spans up to about 40 m and that thereafter congested traffic governs. This study uses traffic microsimulation as a basis for the comparison of regularly-used free and congested traffic flow models. The free-flow model developed by OBrien and Caprani (2005) is used together with a synthesized version of the congested flow models of the literature, illustrated in Figure 1.

Traffic microsimulation offers a comprehensive approach to the modelling of traffic as it models individual vehicle and driver behaviour giving a more realistic picture of traffic states and their consequent load effects on bridges. The Intelligent Driver Model of Treiber et al (2000) is used in this study.

In this study, 50 days of traffic is generated for the free-flowing models whilst 240 hours of continuous traffic is generated for the congested models, representing 10 weeks of data of 5 working days per week. For each of the main models traffic compositions of 0%, 50% and 90% cars are considered. In all studies, 5 sets of data were generated and processed to ascertain repeatability.

Three load effects for bridge lengths of 20 to 60 m were calculated both before and after the application of the traffic microsimulation model. In this way the impact of traffic microsimulation was examined with reference to traditional traffic models, whilst keep the same underlying traffic constant.

The Composite Distribution Statistics (CDS) model proposed by Caprani et al (2008) is used to extrapolate the simulated load effects to determine the characteristic load effects in each case.

The microsimulation model results suggests that standard congestion models are very conservative – a nominal gap of 15 m may be more appropriate than a nominal gap of 5 m. It is also found that lifetime load effect is sensitive to high percentages of cars and that the application of traffic microsimulation tends to reduce overall lifetime load effect values, in comparison to the standard free-flow and congestion models. Lastly, it was found that the governing form of traffic is sensitive to the values of DAF applied to free-flowing traffic. Indeed, even given current DAF values, the governing form of traffic can be less than expected for some load effects (in particular, Load Effect 2). Until further research is carried out into lifetime values of DAF, it will be difficult to adequately state governing forms of traffic for different bridge lengths. In addition, since the governing form of traffic is shown to depend on the load effect considered, it seems prudent to consider both traffic states in any bridge assessment. This being the case, traffic microsimulation is shown to be an ideal tool for this purpose.

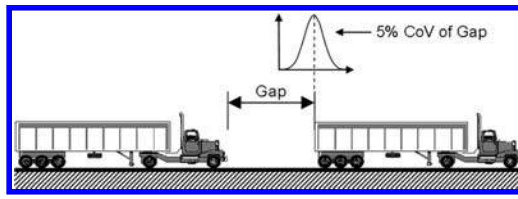

Figure 1. Illustration of the Standard Congestion Model (SCM) for this study.

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Pseudodynamic and cyclic tests on reduced-scale pier-deck sub-systems

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ABSTRACT

In this paper the experimental program on-going at the Laboratories of the University of Basilicata, within the Line 6 (Seismic Vulnerability of Bridges) of the SAGGI (Integrated Systems for the Global Management of Infrastructures) research project is described.

The DiSGG of the University of Basilicata was involved in the Line 6 of the SAGGI project, dealing with the seismic risk and vulnerability of the Italian highway bridges. An extensive program of experimental tests, is being carried out at the Laboratory of DiSGG, in order to assess the seismic behaviour of pier-deck sub-systems.

The test apparatus consists in a very stiff steel structure, simulating the bridge deck, driven by a double-acting servo-hydraulic dynamic actuator which applies the horizontal seismic forces/displacements of the deck. A couple of pseudo-dynamic actuators are used to reproduce the tributary weight of the deck.

Three different types of connection between deck and pier have been considered, i.e.: (i) rigid connection realised by stiff Z-shaped steel sections, (ii) semi-rigid connection realised by traditional neoprene pads and (iii) seismically isolated configurations realised by high damping rubber bearings (HDRB). As far as the bearing devices is concerned, two neoprene pads replaced from a real viaduct during scheduled maintenance services have been used. The isolation devices, instead, have been purposely designed according to specific performance objectives.

The testing models consist in 1 : 3-scale RC single shaft piers with hollow circular cross section with 1m diameter and 125 mm thickness. Eight pier models, four with 3.2 m effective height and four with 1.7 m effective height have been realized. The structural characteristics of the pier models have been derived from the examination of the piers of five multi-span simply-supported bridges of the A16 (Napoli-Canosa) Italian highway.

The longitudinal rebars of two slender and two squat pier models, have been artificially corroded through an accelerated oxidation process, calibrated to get a given reduction of bar diameter, corresponding to a corrosion period of approximately 10 years in a medium deterioration ambient.

The pier shaft has been realised using micro-concrete, due to the reduced dimensions of the models. Two concrete mixes were prepared with different strength in compression.

The mechanical behaviour of micro-concrete and the steel rebars have been evaluated by experimental tests on specimens taken during the construction of the pier models. The experimental program includes both cyclic tests at different displacement amplitudes and pseudodynamic tests at different seismic intensity levels.

The main parameters of the experimental tests (i.e. the displacement amplitudes of the cyclic tests and the Peak Ground Accelerations of the pseudodynamic tests) have been selected based on preliminary simulation analysis, with a numerical model implemented in Opensees.

The mechanical behaviour of the piers has been modeled with nonlinear force-based fiber elements with distributed plasticity, calibrating the constitutive laws of steel and concrete based on the experimental results of the characterization tests on the materials.

Nonlinear time history analyses have been performed to evaluate the seismic response of the pier-deck sub-systems and the cyclic behaviour of the piers to be expected in the pseudodynamic and cyclic experimental tests, respectively. Based on the outcomes of these preliminary analyses the program of the experimental tests on slender and squat piers have been defined.

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The case study and application of the substructure replacement technique for bridges having serious scouring of foundation

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ABSTRACT

To resolve the dangerous situation of naked bridge foundations, protecting or strengthening measures have to be taken or new bridges have to be rebuilt. Normally, protecting or strengthening measures can be classified into 3 groups:

(1) Riverbed stabilization structures, such as gabions, concrete blocks, weirs and submerged sills, can help to slow down scour tendency.
(2) Foundation protecting methods, such as gabions, concrete encasement and surrounding PC piles, can provide temporary protection with little strengthening effects for foundation.
(3) Foundation strengthening methods, such as enlarging foundations and increasing piers, can provide adequate strength for foundation but foundations are still vulnerable to flood due to their huge volume after retrofit.

Each protection or strengthening technique has its applicable situation.

Under the consideration of original bridge structure type, river hydraulic characteristics, nice appearance, traffic maintenance, demand of hydraulic engineering and superstructure integrity, an innovative Substructure Replacement Technique has been proposed to deal with the seriously exposed bridge foundations in Taiwan. By transferring the loading of superstructure from original bearings to temporary shoring structure, the Substructure Replacement Technique not only ensures the safety and integrity of superstructure and pier caps but also maintains the original traffic function. After replacing damaged foundations with new ones built below riverbeds and constructing new columns smoothly connected with the original pier caps, the safety of the bridge can be restored.

The innovative Substructure Replacement Technique was adopted by the first time in Taiwan to cope with naked foundations of the Shi-chou Bridge. Total 19 piers from P38 to P56 are strengthened using this technique.

The Substructure Replacement Technique has four outstanding characteristics:

(1) This technique is capable of replacing naked and dangerous foundations with new ones, which makes this method distinct.
(2) This technique utilizes jacks to transfer bridge loadings to temporary shoring structure. The temporary shoring structure not only sustains all the loadings from superstructure but also minimizes deviation displacement before and after construction, which is indeed a challenge.
(3) The temporary shoring structure is connected with the steel columns embedded in the new constructed RC piles, which is stable and safe. The temporary shoring structure for every pier is the same, which means the steel beams and angles can be recycled and the project will be cost effective.
(4) Only exposed foundations are demolished and consolidated, and the original superstructure, pier cap and part of the pier are retained. This enhances seismic capacity of the bridge and flood conveyance ability of the river at least expense.

The innovative Substructure Replacement Technique demolishes exposed foundations and constructs new piers and foundations which meet seismic and hydraulic requirements. Because the bridge loadings are transferred to the temporary shoring structure, the original flow of traffic can be maintained and the deviation displacement before and after construction is negligible. For naked bridge foundations, this cost effective technique is indeed a promising solution for promoting the performance of bridges at least expense.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Bridge maintenance prioritization through visual inspection results

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ABSTRACT

The current bridge visual inspection system in Taiwan was introduced in this paper. According to the collected visual inspected information, a further analysis was adopted to propose a criterion for prioritization of bridge maintenance. A bridge is composed of 20 components which are evaluated with DERU method (see Table 1). For each damaged component, corresponding maintaining methodology is recommended and its amount is estimated. All of the healthy conditions of bridge components are summarized as a 'structural index', combining the other three indices of 'serviceability index', 'vulnerable index' and 'distinctive index' into a comprehensive Maintenance Priority Index (MPI) result. Three different field cases were discussed in this paper. A list of emergent bridges was clearly observed as getting their MPI rating results. Consequently, the optimum bridge maintaining strategy was promising to be set by sorting the priority with MPI rating, and the limited resource can meet the largest benefit.

The prioritization method has been applied onto three different field cases. In Figure 1, all the cases' curves (Yilan county case, Taoyuan county case and entire Taiwan case) were presented in the same figure. Because the case III was based on worse initial conditions with the other two, it can be seen, as a result, most bridges come out with dramatically higher MPI values.

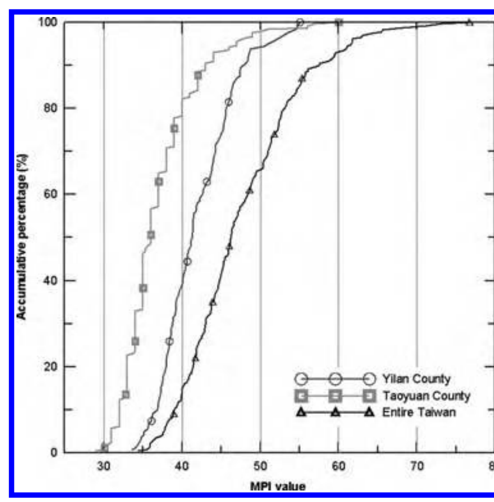

Figure 1. Comparative drawings of MPI distribution for Yilan county, Taoyuan county and entire Taiwan.

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Table 1. The criteria of DERU.

Grade	0	1	2	3	4
Degree (D)	None	Good	Failure Slightly	Failure	Failure Seriously
Extent (E)	Unable Inspection	$<10\%$	10%~30%	30%~60%	$>60\%$
Relevancy (R)	Unable Determine	Slight	Small	Middle	Large
Urgency (U)	Unable Determine	Routine Maintain	Within 3 years	Within 1 year	Emergency

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Time-dependent reliability analysis of systems with repairable or non-repairable components

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ABSTRACT

The paper presents an exposition of time-dependent system reliability analysis, and it illustrates several distinctions between the analysis of the system with repairable and non-repairable components.

In case of a system with non-repairable components, the computation of the cumulative distribution (CDF) of the time to failure is conceptually straight forward. However, in case of a system with repairable components, the concepts of failure rate and system unavailability are of relevance for life-cycle cost analysis. The computation of failure and repair rates as well as unavailability is complicated, because it involves convolutions of distributions of time to failure and time to repair. Nevertheless, given the advancements in computing, these quantities can now be easily computed.

The paper also formulates the evaluation of life cycle cost for both types of system. The analysis shows that non-repairable systems are costly over lifetime, because their failure will result in the loss of services provided by the structure for the rest of the time horizon. In this sense, repair is an effective way to maintain the functionality of the system, which is well known to engineers. For this reason, critical civil engineering systems, such as bridges, pipelines and transportation systems, are designed and operated with repairable components. This paper presents an analytical consistent approach to evaluate life-cycle cost. It is important to emphasize that in such cases an implicit use of non-repairable system reliability analysis will lead to incorrect results.

The method presented in this paper is generic, and it can be easily extended to other types systems subjected to more involved inspection and maintenance policies (Pandey et al. 2008).

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

An empirical life-cycle assessment of the relative cost-effectiveness of alternative materials for reinforcing bridge decks

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ABSTRACT

This paper compares the cost-effectiveness of alternative bridge deck reinforcement materials. First, the life-cycle activity profiles associated with each alternative are established on the basis of empirical data. The profiles highlight the differences in the frequency of life cycle interventions (deck replacements and deck rehabilitations) which translate into differences in the user cost of the alternative reinforcement materials – a profile with more interventions has more frequent workzones and subsequently, higher user cost. Thus the user cost of each alternative was calculated. Also, the agency cost of initial construction and subsequent rehabilitation were estimated. Finally, the cost-effectiveness of each material alternative was analyzed over the entire life cycle using the equivalent uniform annual cost (EUAC) which converts all cash flows to a yearly equivalent amount over a specified analysis period, for each alternative. The alternative with the lower EUAC is deemed the more economically efficient. To test the robustness of the evaluation outcome, the key input variables were incrementally adjusted and the impact of the relative cost-effectiveness was observed. The key variables included the durations of initial construction project, deck replacement, and deck rehabilitation; traffic volume; workzone speed; deck length and deck width, relative unit price of the reinforcement materials; discount rate; detour length and speeds where applicable; vehicle occupancy; and minimum hourly wage, average fuel economy; and fuel price.

The results suggest that solid stainless steel (SS) is the more economically efficient reinforcement option. However, for bridges with very low traffic volumes, the attractiveness of SS is far lower than that for high-volume bridges. Also, it was determined that as user costs increase, the relative attractiveness of solid stainless steel also increases by a certain rate. The sensitivity analysis for reinforcement price showed that solid stainless steel is generally more attractive unless when its unit price is approximately 7 times more than that of traditional carbon steel.

In conclusion, the research showed that using SS generally leads to significantly higher initial costs but drastically reduced costs over the bridge life cycle, particularly when user costs are considered in the analysis. In environments that are more vulnerable to the effects of corrosion, the relative superiority (in terms of cost-effectiveness) of stainless steel is expected to be even higher. In the current national environment that is characterized by uncertainty of sustained funding and high user expectations, technologies that yield longer lasting and superior performing infrastructure are useful for the operations of transportation agencies and to the wider society in general.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)
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A procedure to derive probabilistic fatigue strength data for riveted joints

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ABSTRACT

The availability of probabilistic fatigue strength data for riveted connections is mandatory to carry out reliability analysis of ancient riveted bridges. A procedure to derive probabilistic S-N fields (P-S-N fields) for riveted connections is proposed. Strain-life data as well as fatigue crack growth data from plain material are used to compute the total fatigue life of a simple riveted connection (Fig. 1), integrating both local and fracture mechanics approaches. The basic fatigue data is inputted in the probabilistic form as well as some parameters of the model with higher uncertainty (e.g. crack initiation size). The probabilistic strain-life model as proposed by Castillo and Fernández-Canteli (2009) was used to compute the crack initiation. In order to model the crack propagation, the Paris's law (Paris & Erdogan, 1963) was used with fixed exponent *m* and assuming a log-Normal distribution for the *C* coefficient. A three dimensional finite element model of the riveted joint is proposed in order to assess the local stresses/strains at the critical location. This model is able to account the clamping effects of the rivet on local stresses/strains. The clamping stresses on rivet and friction are assumed random variables, following triangular probability density distributions. Details about the experimental data (plain material and riveted connection) and numerical modeling can be found in Silva (2009).

The probabilistic inputs are accounted in the fatigue modeling procedure using the Monte Carlo sampling technique. Figure 2 illustrates the P-S-N field computed for the riveted connection and compares it with the experimental data illustrating the very satisfactory performance of the model. All experimental data falls within a 98% probability of failure band and is symmetrically distributed around the 50% percentile. The P-S-N field was derived from 10000 fatigue life calculations, for each stress range applied to the connection, using randomly selected inputs, respecting the respective statistical distributions.

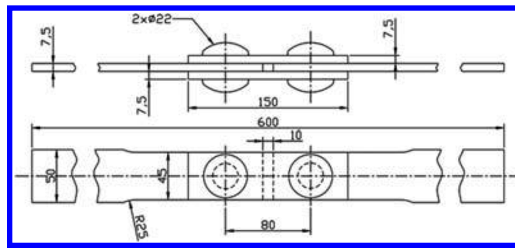

Figure 1. Riveted joint (dimensions in mm).

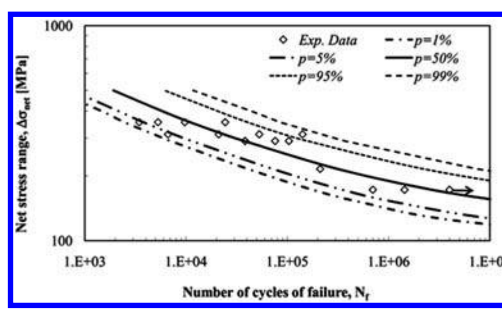

Figure 2. Predicted P-S-N field for the riveted connection.

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Uncertainty evaluation of reinforced concrete structures behavior

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ABSTRACT

When evaluating the behavior of reinforced concrete structures, as several bridges, we should take into consideration that the majority of parameters, like used materials and geometry, vary along the structure. This article presents a methodology, that takes this fact into consideration, and which purpose is to characterize, in a consistent way, if any reinforced concrete bridge presents a behavior within the expected one, or not. The methodology is divided on following steps: 1) Develop a calibrated deterministic numerical model; 2) Determine random distribution function for each input parameter; 3) Develop a non linear probabilistic analysis; 4) Calculate a liability index which relates, in a consistent way, the proximity of numerical and experimental data.

Two sets of laboratory tested beams, with different support conditions, simply supported in one batch (Figure 1) and mixed supported on the other (Figure 2), are firstly analyzed by this methodology (Matos *et al.*, 2008). Those beams were executed at same time, presenting different reinforcement typologies and concrete covering. Main conclusions of this first analysis are the applicability of purposed methodology (Figure 3 and 4) and the importance of gathering any data from analyzed reinforced concrete structures. The behavior of reinforced concrete bridges can be so evaluated through this methodology, presenting it, a great utility for their safety evaluation by detecting any abnormal behavior.

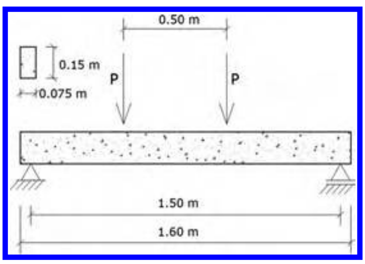

Figure 1. Simply supported beam.

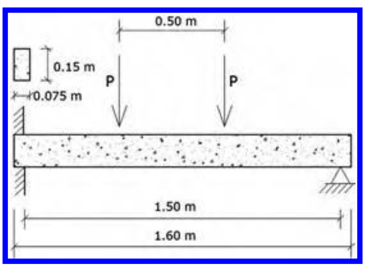

Figure 2. Mixed supported beam.

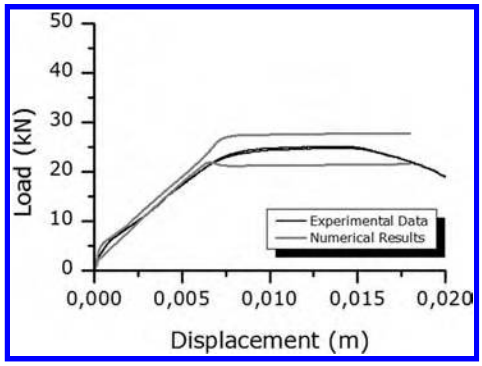

Figure 3. Simply supported beam results.

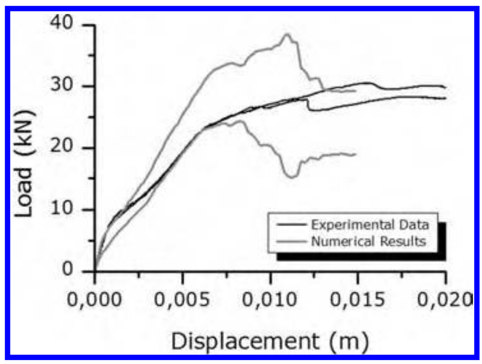

Figure 4. Mixed supported beam results.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Toward a more rational serviceability considerations for high performance steel bridges

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ABSTRACT

There have been significant advances in development of high performance materials over the past two decades. High Performance Steel (HPS) is an example that provides up to 18% cost savings and up to 28% weight savings when compared with traditional steel bridge design materials. HPS offers higher yield strength, enhanced weldability, and improved toughness which results in smaller cross sections, lighter and much more economical designs. On the other hand, smaller cross section leads to more flexible bridges that do not satisfy the existing serviceability deflection criteria.

AASHTO Standard Specifications limit live load service deflection to L/800 for general bridges and to L/1000 for bridges that are used by pedestrians. These limits were employed to avoid "undesirable structural and psychological effects due to their deformations." However, results of prior studies indicate that deflection and L/D limits do not necessarily address these objectives. Existing limits do not prevent damages in structures because they check global deflection, while the damages are a consequence of local deformations such as connection rotations and twisting of floor-beams relative to support members. Furthermore, human susceptibility is more influenced by derivatives of deflection rather than the deflection itself. Thus, there is a need for development of a more rational serviceability criterion which is the objective of this study.

Control of undesirable psychological effects on passengers and pedestrians is apparently one of the primary reasons for AASHTO deflection limits. However, prior research work indicates that it is not just live load defection but vertical acceleration and bridge dynamic characteristics (e.g., frequency) that control vibration and human perception.

There are a few alternate design methods which have been developed to better address serviceability and durability issues. However, these methods have not been adopted in part due to practical limitations in their application but mostly due to lack of consensuses. Prior studies have suggested that speed parameter is the most important parameter in determining bridge dynamic response under moving truck load. In this study, bridge dynamic responses (deflection & acceleration) versus speed parameter are investigated for several cases and a good approximation of bridge response in terms of speed parameter is presented.

This paper includes initial results of an extensive analytical study to develop more rational serviceability and durability criteria. The analytical study employs 2-D and 3-D Finite Element (FE) models to evaluate dynamic response of bridges under moving truck load. Results are compared to special cases where exact solutions exist. The results are in agreement with the exact solutions. Currently, parameter study is being performed. Among parameters considered are truck speed, number of spans, damping ratio, number of trucks, and spatial effects. The paper will also present vibration criteria of other available countries specifications such as Canada and Europe.

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Refined load rating analysis of in-service bridges in North Carolina, USA

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ABSTRACT

In order to conduct an assessment of the actual load carrying capacity of the state's in-service bridges, North Carolina Department of Transportation's Bridge Management Unit has identified several bridge structures to perform refined load rating using a combination of 3-D Finite Element Analysis and Diagnostic Load Testing. The short-listed bridges carry primary routes and are mostly constructed prior to the 1950s. The majority of these bridges require posting for North Carolina legal loads. Bridge types included in this project are single or multi-span, skewed or square, reinforced concrete deck girders, concrete slabs, steel girders, and three sided reinforced concrete culverts. The refined load rating analyses have been completed on seven bridges, each having its unique geometric, material and structural characteristics.

The general procedure for evaluating the current load ratings by determining the actual live load carrying capacity of the bridge initiates with the development of a 3-D finite element (baseline) model of the bridge superstructure, followed by field load testing of the bridge, then utilizing the field measured strain and displacement data to validate and refine the baseline model and finally, using this refined model to compute the live load ratings of the bridge. The diagnostic load testing of the bridge enables to capture the real-time live load response characteristics and its spatial distribution.

Live load ratings for the North Carolina bridges computed according to this refined methodology provide for a realistic estimate of the reserve load carrying capacity of the structures. In most cases, the results of the refined rating procedure indicate that rating factors may increase by up to 75% when compared with results of the conventional analysis procedures thereby, recommending either a substantial increase or complete removal of posting limits on specific bridges. However, there are some instances where the actual bridge condition supported by the field load test data justifies lowering the rating factors thereby, recommending either a decrease of existing posting limits or even posting of certain bridges that are currently not posted.

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Designing the rehabilitation of the Henley Street Bridge in Knoxville, Tennessee, USA

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ABSTRACT

The existing Henley Street Bridge is a six-span, 423 m long open spandrel reinforced concrete arch bridge flanked by 50 m long, three-span approach girder structures at each end. The arch span lengths range from 56 m to 97 m with an average rise to span ratio of 0.30.

The existing bridge deck contains 26 expansion joints that are believed to be the primary sources for water intrusion causing concrete deterioration in the elements of the arch structure during its 79-year life. In the present condition of the bridge, the arch ribs, and several spandrel columns do not rate for the design vehicle HS-20. Further, the arch ribs at the spring lines and spandrel columns located near the span quarter points and arch crowns exhibit seismic demands in excess of their respective structural capacities. Based on a detailed condition assessment of the existing structure, and the results of analysis, a decision was made to salvage the arch ribs, the arch piers, the pier foundations, the approach curtain walls and portions of the abutments. The existing superstructure deck and the spandrel columns would be replaced as a part of the rehabilitation work.

Special emphasis was given to arrive at a design solution that would address some of the pertinent elements of a sustainable design process. The key areas that were considered include enhancing mobility, durability and safety, utilizing high performance materials, applying context sensitive solutions, and complying with environmental and preservation laws and regulations.

Built in 1930–32, the bridge has been determined eligible for listing in the National Register of Historic Places. Therefore, the bridge improvements have been designed in accordance with the National Historic Preservation Act and the Department of Transportation Act of 1966.

Further, in order to arrive at a durable bridge structure that would be capable of handling one additional lane of traffic, a 26 m wide (out-to-out), 523 m long continuous superstructure deck is designed with expansion joints located only at the abutments. However, the continuity of the superstructure resulted in increased force and deformation demands due to interaction of multi-span arches subjected to combined load effects at various sections of the bridge structure. A combination of innovative design techniques are used to mitigate these adverse load effects.

The bridge construction work is scheduled to commence in 2010. The estimated cost of the rehabilitation work is $30 million (US currency). When completed, the structure will be the longest jointless multi-span open spandrel deck arch bridge in the world.

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Stress intensity factors evaluation for riveted beams applying FEA with VCCT

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ABSTRACT

Structural integrity assessments of old steel riveted bridges are more and more frequent. Most of these structures were built at the end of the 19th century or the beginning of the 20th century with angles and plates joined by rivets and made of puddle iron or wrought steel. Fatigue is one major concern for these structures since they show a long operational period with increasing traffic intensity, many times without the required rehabilitation procedures. Despite the S-N approach is widely used to assess the fatigue damage for riveted steel constructions, Fracture Mechanics appears as an alternative to perform residual life calculations. However, the use of the Fracture Mechanics is very often limited to the application of simplified formulae for stress intensity factors evaluation, available in standard handbooks that may lead to inconsistent fatigue life predictions.

A methodology for the evaluation of stress intensity factors of cracked riveted members, based on detailed 3D finite element models, is proposed. Despite being generally enough to be applied to other riveted geometries of interest, the proposed methodology is demonstrated for a riveted T beam, also investigated by Moreno & Valiente (2004) using an analytical approach. Figure 1 illustrates the geometry of the beam. The beam was subjected to both axial loads (N) and bending moment (M). A 3D finite element model of the riveted beam was proposed to compute the stress intensity factors for a crack propagating at the plate web, using the Virtual Crack Closure Technique (VCCT) (Krueger 2004). The clamping stresses of rivets are accounted in the model through a temperature variation. Figure 2 illustrates the stress intensity factor variation for the riveted beam under a bending moment, M. The stress intensity factor was normalized using the maximum bending direct stress ($\sigma = Mb/2I$). After an initial increase in the stress intensity factor, a reduction is observed until zero. This reduction is accompanied with an increase in tensile stresses in the angles near to the crack.

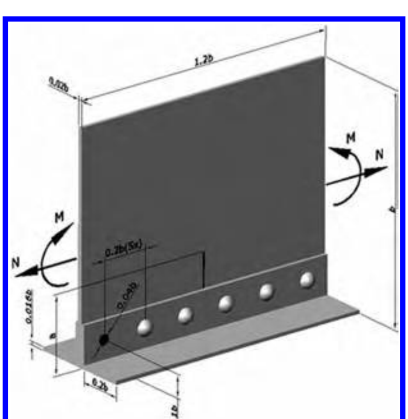

Figure 1. Geometry of the cracked riveted T beam (b = 0.5 m).

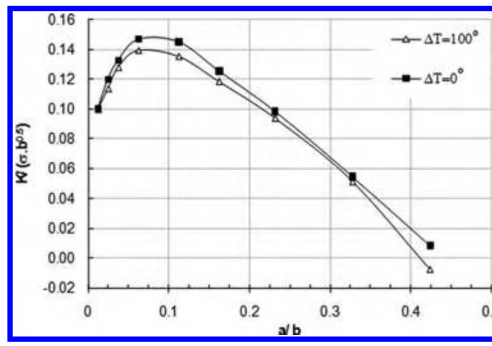

Figure 2. Stress intensity evolution for the cracked beam under the action of a bending moment.

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Deterioration and maintenance of RC bridge decks under uncertainty: Condition and reliability indicators

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ABSTRACT

In this paper, a condition-based approach describing lifetime deterioration of RC bridge decks is proposed. Time-dependent performance indicators such as condition and reliability indices are evaluated over time in order to develop a rational bridge maintenance strategy. A probabilistic analysis considering the condition index a non-linear function of time is introduced. The condition index represents the percentage of deteriorated bridge deck area (concrete spalling due to environmental attack). The reliability index can be obtained by considering the flexural failure mode described by its limit state function.

The proposed approach is condition-based. This means that condition index governs the decision process in terms of maintenance assessment. Moreover, through the application of this approach, visual inspections are able to predict the residual lifetime of the deck depending on the inspected deck location (top or bottom of the deck, right or left lane of traffic). Figure 1 shows the mean of the condition index over time under maintenance. Initially, the condition index is perfect (i.e., zero) until cracks propagate to the concrete surface; when service life is reached, the condition index starts to deteriorate due to crack propagation.

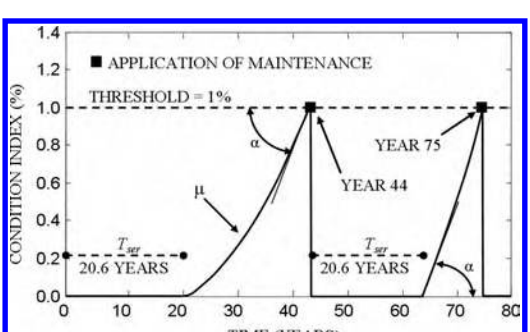

Figure 1. Profile of mean of the condition index over 80 years with maintenance.

In this paper, condition and reliability indices are directly related by using a corrosion model that affects both. In fact, spalling of concrete surface is a direct consequence of active corrosion of the steel reinforcement.

The applicability of this approach has been proved by studying an existing RC bridge deck located near Wausau, Wisconsin, USA. The results obtained have highlighted that maintenance planning can be developed based on visual inspections data and reliability can be predicted.

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Multi directional hysteretic damper with adaptive post-elastic stiffness for seismic protection of bridges in near fault zones

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ABSTRACT

This paper presents a new hysteretic damper suitable for seismic protection of highway bridges in near-fault (NF) zones. The damper is a Multi-directional Torsional Hysteretic Damper (MTHD). The MTHD is made of eight steel cylinders named "yielding cores", symmetrically arranged, which are designed to yield in torsion. The torsion is created by arms attached to the cores. The torsional arms are guided by a rail system at top of the device which is designed to convert planar multidirectional displacements of the deck (to which the rail system is attached) into rotation in the arms and twisting in the cores. All eight yielding cores are supported against bending by a single stiff central column. Easily adjustable arm length allows for easily adjustable reaction force and maximum allowable displacement of MTHD. This unique working mechanism of the device makes it capable of large displacement capacities. More importantly, the damper possesses a hyperbolically varying post-elastic stiffness as a result of its special working mechanism which creates a geometric hardening effect. The variable stiffness is found to be effective in limiting the lateral displacement of the seismic-isolated bridge decks in near-fault zones and is the focus of this paper.

Brief illustrated description of the device is given and the associated analytical formulations of force-displacement response of MTHD are briefly introduced. Since stable function of metallic dampers rests on the cyclic performance of energy dissipaters at their repeated plastic-strain excursions, finite element analysis of yielding cores (energy dissipater unit of MTHD) was performed to study the strain distribution over the energy dissipater. The analysis show that slight flexibility of supporting structure will cause small bending in the cores which are supposed to be subjected to pure torsion, and thus cause some non-uniformity in strain distribution in case of elasto-plastic material model with zero plastic hardening. However, small plastic hardening in steel will ensure uniform distribution of plastic strains over the yielding part of the yielding cores. The cores are thus judged to be capable of stable cyclic performance provided that suitable grade of steel is used.

To study the effect of post-elastic hardening feature of MTHD on the performance of a typical bridge under NF ground motion, nonlinear time history (NLTH) analyses are conducted. NLTH analyses of seismic isolated bridges with MTHD and with hysteretic dampers having elasto-plastic behavior, using NF ground motions revealed the adaptive behavior of the device which is a result of this gradual hardening feature, such that at lower displacements (DBE), force levels are close to a regular system while at highest levels of displacements (MCE) when displacement control becomes critical in preventing the deck from falling off of the supports, the device hardens to make the substructure yield and limit deck's displacement at any direction. The results of sensitivity analyses, performed to study the effect of the MTHD on seismic performance of NF bridges in relation to the ground motion magnitude and characteristic strength of the energy dissipation device, are presented.

The paper is devoted to analytical issues only. Tests are under way in the Engineering Sciences Department of Middle East Technical University to verify MTHD's working mechanism, results of analytical formulations and cyclic performance of the device which will be presented in subsequent publications.

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Distribution of live load effects in integral bridge abutments and piles

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ABSTRACT

Structural analysis of highway bridges using complicated 3-D finite element models (FEMs) to determine live load effects in bridge components is possible due to the readily available computational tools in design offices. However, throughout the design process, using such complicated methods is tedious, time consuming and expensive. Therefore, most design engineers prefer using simplified 2-D structural models of bridges and live load distribution equations available in current bridge design codes such as AASHTO (American Association of State Highway Transportation Officials) LRFD (Load and Resistance Factor Design) Bridge Design Specifications (2007) to determine live load effects in bridge components. In AASHTO LRFD Bridge Design Specifications, live load distribution equations are available only for the girders of jointed bridges. AASHTO does not have any provisions for the calculation of live load effects in integral bridge components including the girders, abutments and piles. Consequently, these live load distribution equations are also used for designing the girders of integral bridges. In addition, most design engineers generally calculate the live load effects in the abutments and piles of integral bridges by using the AASHTO live load distribution equations developed for the girders of jointed bridges. This approach is based on the assumption that the same rotations about a transverse axis perpendicular to the longitudinal direction of the bridge occur both in the abutments and the girders under live load due to the monolithic construction of the superstructure-abutment joint in integral bridges. However, it is anticipated that the concentrated rigidity of a particular girder combined with those of the adjacent girders connected to the abutment having a smeared rigidity, may produce a live load distribution within the abutment and piles different than that calculated using the live load distribution equations developed for the girders of jointed bridges. Therefore, using AASHTO live load distribution equations may results in either conservative or unconservative estimates of the live load effects in the piles and abutments of integral bridges. Thus, in this study, live load distribution equations for integral bridge substructures are developed. For this purpose, numerous 3-D and corresponding 2-D structural models of typical integral bridges are built and analyzed under AASHTO live load. In the analyses, the effect of various superstructure and substructure properties such as span length, girder spacing, girder stiffness, abutment height, pile size, pile spacing and foundation soil stiffness are considered. The results from the 2-D and 3-D analyses are then used to calculate the live load distribution factors for the abutments and piles of integral bridges as a function of the above mentioned properties. live load distribution equations are then developed to estimate the live load moments and shear in the abutments and piles of integral bridges using these live load distribution factors and nonlinear regression analysis methods to address the above mentioned uncertainties and to provide useful tools to the bridge engineering community at large for the design of integral bridge abutments and piles under live load effects. It is observed that the developed live load distribution equations yield a reasonably good estimate of live load moment and shear in the abutments and piles of integral bridges.

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Static and dynamic model validation and damage detection using wireless sensor network

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ABSTRACT

Structural health monitoring (SHM) is brought to attentions due to aging of civil infrastructures and their undeniable effect in the public safety. Traditionally, SHM has been developed based on the wired network of sensors and centrally depository of data. While, the high costs and the installation difficulties associated with the wired sensor network have limited the large-scale application of SHM, technological advances in wireless sensor network (WSN) have made the SHM more affordable and potentially scalable. In some recent SHM projects WSN is selected as a promising data acquisition system which implies the progressive trend of using WSN in SHM.

This paper presents a set of static and dynamic tests and damage detection on a 3-dimensional steel truss utilizing WSN. The Imote2 platform, developed by Intel, together with SHM-A sensor board, is used for acceleration measurements. An analytical model of the truss structure is developed based on the available specification of the truss and the properties derived from static tests. Extracting the modal properties of the structure from dynamic tests, the analytical model is updated. Subspace state space algorithm is used to identify the dynamic properties of the truss. Finally, a local damage is simulated on the structure and a set of data from new measurements is passed through the statistical damage detection algorithms to indicate the changes in the dynamic characteristics of the system.

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Modern permanent and portable restraint systems for bridges

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ABSTRACT

What does really happen when a 38 ton truck crashes into a bridge restraint system? Due to some hazardous accidents on bridges in the past when heavy goods vehicles broke through bridge safety barriers causing structural damage on bridge structures, the necessity for a more detailed understanding of the impact procedure was obvious. Therefore some research projects under great efforts of accredited test houses, national road authorities, bridge designers and industry were carried out to show what really happens when a 38 ton truck crashes into a bridge barrier system. Now the industry offers highly sophisticated barrier systems to provide very high containment levels causing less stress for the bridge structure – for bridge designers it is possible to optimize their static load cases and calculations.

Figure 1. H2/TB51 test with 13 ton bus and H4b/TB81 test with 38 ton truck.

Research projects and modern developments of industry brought up that it is possible to provide high containment levels for bridges at acceptable loads on the bridge structure. Nowadays bridge designers are able to choose from various kinds of "premium restraint systems" complying with national requirements for bridge design and the European standards for restraint systems.

Designing bridge structures by using the failure loads of barrier anchors is not state of the art. For having a load case that comes close to reality also the impact loads mentioned in some national standards are not sufficient for calculating impacts on safety barriers. Now it is possible to operate with forces measured in crash tests. Numerical simulations on this basis brought up that premium restraint systems decrease the impact load on bridges to such an amount that an upgrading of the containment level on existing bridges is also possible.

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Effect of soil bridge interaction on the distribution of live load effects among integral bridge components

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ABSTRACT

An integral abutment bridge (IAB) is one in which the continuous superstructure, the abutments and the single row of steel H piles supporting the abutments are built monolithically to form a rigid frame structure.

In bridge design, most bridge engineers prefer using simplified two dimensional (2-D) structural models and live load distribution factors available in current design codes to determine live load effects in bridge components. Although the monolithic construction of IABs forces the substructures to interact with the backfill and foundation soil under thermal and gravitational load effects (Dicleli & Albhaisi 2003), the current state of design practice in North America and Europe normally neglects soil-bridge interaction effects in live load analyses of IABs. That is, the backfill behind the abutments is not considered in the 2-D structural models of IABs for live load analysis and the piles are usually modeled as simple equivalent cantilevers fixed at some distance below the ground surface. Although many research studies have been conducted on the effect of backfill and foundation soil on the performance of IBs under thermal effects (Khoadir and Hassiotis 2005), research studies on the performance of IBs under live loads are scarce.

In this study, the effect of soil-bridge interaction on the magnitude of internal forces in IAB components (deck, abutment and piles) due to live loads is studied. For this purpose, structural models of typical IABs are built by including and excluding the effect of backfill and foundation soil. The analyses of the models are then conducted under AASHTO live load. In the analyses, the effects of the backfill and foundation soil on the magnitude of the internal forces in IAB components are studied for various structural, geometric and geotechnical parameters such as bridge size, abutment height and thickness, pile size and orientation, number of spans and foundation soil stiffness.

The analyses results revealed that soil-bridge interaction has a significant effect on the magnitudes of the live load moments in the components of IABs. Including the effect of backfill behind the abutments in the structural model is generally found to result in larger superstructure support and abutment moments and smaller superstructure span and pile moments. The difference between the live load moments for the cases with and without soil-bridge interaction effects is found to be a function of foundation soil stiffness. However, soil-bridge interaction is found to have only a negligible effect on live load shear in the superstructure. Furthermore, it is found that the analyses performed using the equivalent pile length concept inconsistently yield either conservative or unconservative estimates of the internal forces in the components of IABs except for the superstructure shear where the results of the equivalent pile length model coincide with those of the models including soil-bridge interaction effects. Based on the findings of this research study, it may be recommended to include the abutment-backfill and soil-pile interaction behavior in the structural model of short to medium length IABs for the purpose of live load analyses. The linear soil-bridge interaction modeling techniques presented in this paper may be used for this purpose.

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Live load distribution in integral bridge girders

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ABSTRACT

Integral abutment bridges (IABs) possess many economical and functional advantages. Consequently, they have become very popular in many parts of USA, Canada and Europe. However, standard design methods for IABs have not been fully established yet. Thus, many practicing engineers use the provisions for regular jointed bridges in current bridge design specifications such as AASHTO (American Association State Highway Transportation Officials, 2007) to design IABs. This also includes using such provisions for the design of IAB girders under live load effects. Most bridge engineers use simplified two-dimensional structural models and live load distribution factors (LLDFs) readily available in bridge design specifications such as AASHTO (2007) to determine live load effects in bridge girders. The live load distribution equations (LLDEs) were basically developed for jointed bridges where the superstructure is separated from the abutments via expansion joints. However, in the case of IABs, the monolithic construction of the superstructure-abutment joint forces the superstructure and the abutments to act together under live load effects. The continuity of the superstructure-abutment joint in IABs is found to improve the distribution of live load moment among the girders especially for short spans (Dicleli & Erhan, 2009). Accordingly, using the LLDFs in AASHTO LRFD Bridge Design Specifications (2007) for the design of IAB girders may result in incorrect estimates of live load effects. As a result, in this study, LLDEs for the commonly used prestressed concrete girders of single-span IABs are developed to offer a practical tool to design engineers for the calculation of live load effects in IAB girders. For this purpose, two and three dimensional finite element models (FEMs) of several IABs are built and analyzed. In the analyses, the effects of various superstructure properties such as span length, number of design lanes, prestressed concrete girder size and spacing as well as slab thickness are considered. The results from the analyses of two and three dimensional FEMs are then used to calculate the live load distribution factors (LLDFs) for the girders of IABs as a function of the above mentioned parameters. LLDFs for the girders are also calculated using the AASHTO formulae. Comparison of the analyses results revealed that LLDFs for girder moments and exterior girder shear of IABs are generally smaller than those calculated using AASHTO formulae especially for short spans. However, AASHTO LLDFs for interior girder shear are found to be in good agreement with those obtained for IABs. Consequently, direct live load distribution formulae and correction factors to the current AASHTO live load distribution equations are developed to estimate the girder live load moments and exterior girder live load shear for IABs with prestressed concrete girders. Comparison of the LLDFs obtained from the analyses of FEMs and those calculated using the developed equations revealed that the developed formulae yield a reasonably good estimate of the live the load moment in short to medium span prestressed concrete IAB girders.

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Comparison of fatigue behavior under concentrated loads of orthotropic decks and crane runways

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ABSTRACT

The introduction of repeatedly moving single loads (wheel loads) in a plated steel structure is a frequent design situation. As a rule the wheel loads are transferred via distributing means into the load carrying structure such as an asphalt surfacing in case of the steel deck of a road bridge or a rail in case of a crane runway girder (Fig. 1).

In case of modern plated steel structures welds are usually located in the load application region leading to a significant increase of the actual stresses compared with the computed nominal stresses according to engineer's theory.

The increase in stress has got several reasons: (i) The shapes of the weld toes and roots form geometric notches amplifying the nominal stresses. (ii) The welding leads to material homogeneities; moreover, residual stresses are induced by the welding procedure. (iii) The concentrated load causes local stress peaks (Fig. 2). Altogether, the wheel loaded region undergoes a complex multiaxial state of stress with several stress rising features.

Considering the application of a stationary static single load the stress-rising effects might be negligible because the stress peaks can be eliminated by harmless local plastifying. In contrast, the wheel loads reciprocate normally, and the cyclic occurrence of the stress peaks might lead to an accelerated material failure that means fatigue.

The state of stress in the wheel load application region of an orthotropic steel deck and a crane runway shows similarities. The occurrence of local stress components in addition to the global nominal stresses is characteristic of both. Since the local stress components interact with the flexural stresses due to global bending a design situation with combined stresses has to be assessed. As the different stress components act cyclically and out of phase the wheel load application turns into a complex multiaxial fatigue problem with non-proportional loading.

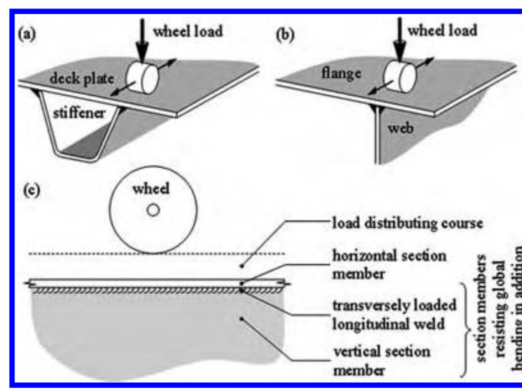

Figure 1. Load application into a steel deck (a) and a crane runway girder (b), schematic load application section (c).

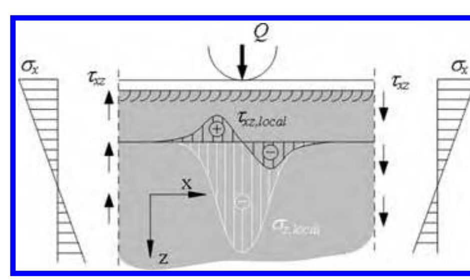

Figure 2. State of stress in the load application region.

In this paper a summary of known fatigue phenomena caused by wheel loads is presented. Finally, the current design approaches of the fatigue assessment for orthotropic decks and crane runways are compared focusing on following issues:

- local stresses and their distribution
- influence of wearing
- fatigue loading
- fatigue categories
- theory of failure

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The strain development in concrete under cyclic loading

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ABSTRACT

Based on the experiments of J.O. Holmen, parametric description of the development of secant modulus of elasticity of concrete under cyclic loading is proposed.

Constants *a* and *b* are introduced, *a* for the decrease of the secant modulus of elasticity in the first phase of its development under cyclic loading, *b* for the remaining proportion of the original secant modulus of elasticity at the beginning of the third phase of its development.

A mathematical function for describing the strain development in a concrete specimen under cyclic loading is sought for. This function should be able to give a value of modulus of elasticity in every particular moment of cyclic loading, thus respecting the three phases in strain development under cyclic loading. The reduced value of modulus of elasticity can be then used for calculating deflections increased by damage accumulation caused by cyclic loading.

The instant value of modulus of elasticity after n_i load cycles is:

$$E_{n_i} = \omega_{F_i} \cdot E_{n_0} \tag{1}$$

where E_{ni} modulus of elasticity of concrete after n_i load cycles; ω_{Fi} = fatigue damage function after n_i load cycles; and E_{n0} = modulus of elasticity of concrete at the start of the cyclic loading.

The fatigue damage function decreases the original modulus of elasticity of concrete at the start of cyclic loading to quantify the deteriorative effect of n_i load cycles of cyclic loading. It comprises of a power and an exponential part. The exponential part is dependent on S_{max} (the maximum stress level during the cyclic loading) and represents rapid decrease of modulus of elasticity in the first phase of cyclic loading, together with the stable progressive decrease of modulus of elasticity in the second phase of cyclic loading. The exponential part is independent of S_{max} and represents

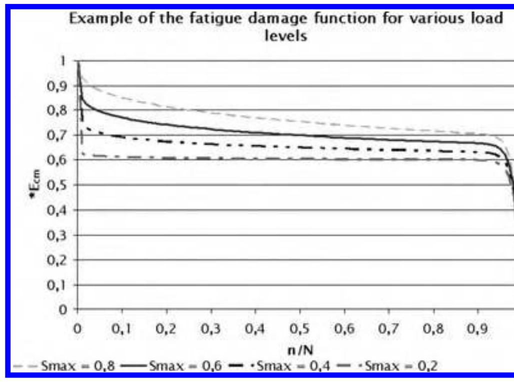

Figure 1. Example of the fatigue damage function for various load levels.

the rapid decrease of modulus of elasticity in the third phase of cyclic loading:

$$\omega_{F_i} = 1 - \left\{ \left[a^{c_4 \cdot \frac{1}{S_{max}^{c_3}}} \cdot \frac{n_i}{c_1 \cdot N} \right]^{\frac{1}{c_4 \cdot \frac{1}{S_{max}^{c_3}}}} + b \cdot \exp\left[\left(\frac{n_i}{N} - 1 \right) \cdot c_2 \right] \right\} \tag{2}$$

where all the constants and variables are explained in the full text of the paper.

The fatigue damage function should be used for assessing deflections of structures subjected to cyclic loading either in "in-hand" calculations or inserted into FEM software producing a complete useful life analysis of a particular structure.

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Cracking of reinforced concrete deck on skewed bridges

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ABSTRACT

A large number of highway bridges are skewed, due to the requirement of crossing other waterway or roadway with various angles. For example, in Michigan they represent 35% of all bridges in the state. It has been noticed that many of recently constructed reinforced concrete decks on skewed bridges have shown significant cracking, especially in the corner areas. They are perceived to be relevant to the skew geometry. This research effort was to investigate the cause of the observed cracking and to develop measures to mitigate or eliminate it.

Accordingly, this study had a focus on corner cracking in reinforced concrete decks of skew highway bridges. A survey of state transportation agencies in the U.S. was conducted on this subject. It found that deck corner cracking in skew bridges is commonly observed in the entire country. Deck inspection for bridges in Michigan was also performed in this study. Cracking intensity in these decks was viewed as an effect of several possible causal factors, which was collected from 40 bridge decks, including 20 straight and 20 skewed structures. Analysis of the inspection results indicates no clearly agreeable causal relations. Two skew decks were instrumented using temperature and strain sensors for the concrete and the ambient environment. Concrete deck's temperature and strain response was collected to thermal-, shrinkage-, and truck-wheel-loads. Test results and thereby calibrated finite element analysis results show that the main cause of skew deck corner cracking is cement concrete's thermal and shrinkage load. Based on current Michigan practice of skew deck design and construction, additional reinforcement in the corner areas is therefore recommended to reduce concrete stresses. Further research is also recommended to develop solutions using optimal combinations of ingredients in concrete and to minimize the constraint between the deck and the supporting superstructure.

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The "Filetto" Bridge on the Santerno River near Bologna (Italy): Seismic retrofit and reinforcement design

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ABSTRACT

The Filetto Bridge was built about in 1930, to over-pass the Santerno river and to connect the village of Fontanelice to the village of Casalfiumanese, two little towns near Bologna, a city located in the northern part of Italy. The bridge has a total length of 110 m, subdivided into 4 spans, each one 27 m long. The restraints are composed by two abutments at the two ends of the 4-span bridge and by 3 intermediate piers laying in the riverbed; the deck is 4,5 meters wide (only one roadway 3,0 m wide). The section of the piers is composed by 4 square columns 70 cm × 70 cm each, located at the angles of a total section of 240 cm × 240 cm in plan: the columns are connected each other by several arched beams. The abutments are a kind of concrete caissons, laying on the slopes at the two sides.

The bridge was constructed with the post-stressed concrete technique (Figure 1).

The aim of the research work here presented is to investigate the seismic behaviour of this structure according to the rules of the new Italian Seismic Design Code, in order to focus the basic problems and to study some possible solutions that can improve the seismic behaviour of the bridge.

Preliminary analysis shows that the existing structure is not able to support the new seismic excitation mainly because the benefit of the post-stress is partially lost due to the traction stresses in the deck, and the present supporting devices cannot support the drifts between the pile and the deck.

The results obtained from several static and dynamic analysis show the necessity of a reinforcement design; the fundamental works which provide the structure with the best performances are:

– complete demolition of the superior r.c. slab of the whole deck and realization of a new slab with lightweight aggregated concrete;
– substitution of the actual restraints with elastomeric/rubber bearings (isolators), see Figure 2;
– realization of structural joints on the abutments to avoid the effects of thermal expansion on the slab;

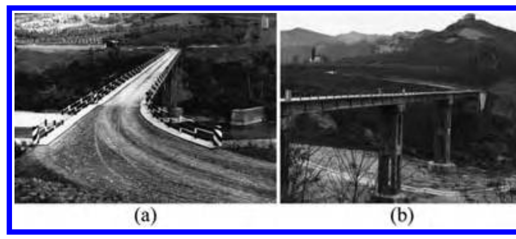

Figure 1. Historical photographs: (a) the gravel roadway leading to the bridge path; (b) overall image.

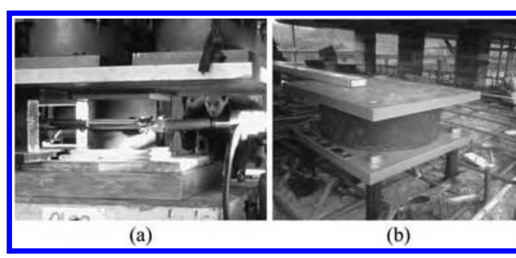

Figure 2. (a), (b) elastomeric bearings.

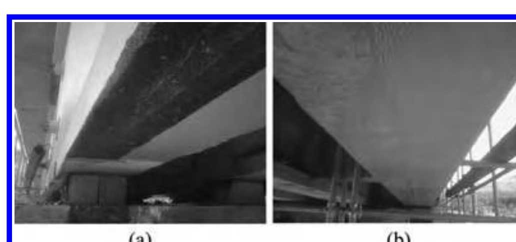

Figure 3. (a), highly resistant carbon fiber strips; (b) painted fiber.

– reinforcement of the r.c. beams of the decks with the insertion of highly resistant carbon fiber strips, in order to restore the loss of post-tension effect (Figure 3).

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Masonry arch bridges – towards a hierarchical assessment framework

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ABSTRACT

Masronry arch bridges are estimated to account for more than 40% of European bridges. Often more than 100 years old these bridges have performed well in service and are arguably the most durable and sustainable bridge type. However, the gradual deterioration of materials with time, coupled with the increase in loading from modern road and rail vehicles, make re-assessment, maintenance, repair and strengthening inevitable in order to ensure that safety, performance and serviceability are sustained at an acceptable level. While numbers of masonry arch bridges in the US are lower these issues are equally relevant.

Bridge managers and owners use a variety of assessment algorithms to identify safe load capacities. The underlying theory behind different assessment approaches varies. The required bridge data also varies from method to method as do the dimensional models used. This diversity of assessment methods explains, in part, the resulting scatter of assessment ratings achieved for a single bridge. What is often more difficult to rationalize is the anomaly of an apparently higher level, or more rigorous, assessment algorithm resulting in a lower assessment rating.

UCD is working with the National Roads Authority in Ireland to rationalize a hierarchical framework of assessment algorithms whereby increasing assessment effort is rewarded by demonstrable convergence toward the ultimate capacity of the bridge. The aim is to develop an algorithm hierarchy for masonry arch bridges. As part of this work 12 masonry arch bridges, characteristic of the range of bridges on the Irish road network, have been assessed using three assessment methods. This paper presents a discussion of the assessment approaches, the assessment ratings achieved and the trends in assessment ratings of the different assessment approaches vis-à-vis each other and also measured responses, to passing weighed vehicles, for two of the bridges

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Target proof load factors for highway bridge assessment in Central and Eastern European Countries (CEEC)

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ABSTRACT

The paper proposes a simple method to obtain the target proof load that guarantees a required safety level on the bridge against the actual traffic. The proof load factors are obtained by means of a reliability-based calibration with 3 different safety levels (2.3, 3.6, 5.0). Two traffic scenarios have been considered, one representative of Western Europe (The Netherlands) and other representative of New Member States (NMS) and Central and Eastern Europe (Czech Republic, Poland, Slovakia and Slovenia). The final result is a simple procedure based on charts and tables that a bridge evaluator without specific knowledge of reliability theory can easily apply.

As an example, in table 1 is presented a summary of the results obtained for a safety level ($\beta = 2.3$), as function of actual resistance and span-length, for Holland, for the case that bridge information is available.

For the rest of the countries, similar values are obtained as a function of safety level and span-length (ARCHES-D16 2009). Therefore, the values in table 2 are proposed for proof load factors to be used in NMS and CEEC countries in the absence of more accurate values obtained for the specific bridge for a Reliability index $\beta = 2.3$.

The values presented in tables 3 and 4 were obtained for the considered countries. In the cases that documentation is not available,

Table 1. Proof load factors for Dutch traffic and $\beta = 2.3$.

	Span-length (m)					
R/Rn	10	15	20	25	30	35
1.0				0,18	0,38	0,49
0,9	0,61	0,70	0,84	1,07	1,16	1,22
0,8	0,86	0,98	1,14	1,26	1,31	1,35
0,7	1,04	1,12	1,26	1,33	1,37	1,41
0,6	1,11	1,18	1,28	1,37	1,41	1,44
0,5	1,14	1,20	1,30	1,39	1,44	1,46

Table 2. Proof load factors for NMS and $\beta = 2.3$.

	Span-length (m)					
R/Rn	10	15	20	25	30	35
1.0						0.31
0.9	0.15	0.28	0.45	0.55	0.59	0.61
0.8	0.51	0.58	0.69	0.78	0.82	0.84
0.7	0.63	0.69	0.82	0.94	0.96	0.98
0.6	0.72	0.78	0.92	1.00	1.04	1.05
0.5	0.78	0.84	0.96	1.04	1.07	1.09

Table 3. Proposed proof load factors for non-documented bridges and Dutch traffic conditions.

Span length (m)	β		
	2.3	3.6	5.0
10	1.18	1.61	2.25
15	1.23	1.67	2.33
20	1.33	1.80	2.51
25	1.41	1.91	2.64
30	1.45	1.95	2.70
35	1.49	1.99	2.75

Table 4. Proposed proof load factors for non-documented bridges in NMS and CEEC countries.

Span length (m)	β		
	2.3	3.6	5.0
10	0.83	1.13	1.57
15	0.89	1.20	1.65
20	1.01	1.36	1.85
25	1.08	1.44	1.97
30	1.11	1.46	2.00
35	1.12	1.48	2.01

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Synthetic fiber ropes to replace steel wire in pedestrian suspension bridges

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ABSTRACT

Rope making has seen a transition from the traditional hemp ropes used during the Inca Empire to the steel wire ropes in the 1830s and now to synthetic fiber ropes. Synthetic fiber has displayed immense potential in terms of its tensile strength, which has made its application in various industries possible. Research and development has led to the manufacture of high modulus and high tenacity products, which have better strength-to-weight ratios than steel ropes. Steel wire ropes which are used for pedestrian suspension bridges prove to be highly redundant, considering the loads they face during their lifetime. They are also more prone to weathering. The cost of repair or replacement of corroded wire is higher as the frequency of replacement goes up based on various climatic conditions. Synthetic ropes provide a feasible option but its application to this field is yet to be explored. Based on the strength requirements for a pedestrian rope bridge, appropriate fiber has to be selected for the bridge. Companies which cater to the rope industry conduct numerous tests to constantly upgrade their products and to overcome any drawbacks the material might have. This study makes an attempt to substitute steel wire in rope bridges to synthetic fiber ropes made from high modulus plastic. By studying the test results available from the companies for every product, a decision about the material with apt properties is made. This will not only make the bridge light but also make the construction process less time consuming and tedious. A suitable structural form has been developed which takes advantage of the suspension and stress ribbon structural types for pedestrian bridges. These bridges will be built on a community basis along remote trails in secluded mountainous regions across the world. Local manpower and material will be used as far as possible in order to provide economic advantage to the people.

Similar programs conducted by developmental organizations have been studied. The Swiss Development Corporation (SDC), Helvetas and Government of Nepal are jointly involved in building rope bridges under the Trail Bridge Program in Nepal. These short span bridges can be easily built from synthetic ropes. The SDC also developed standards for building such bridges in the mountainous terrain of Nepal. These standards will be used as guideline for determining the applicability of the synthetic ropes. The capacity of the ropes is well illustrated by the fact that they are used in heavier industries like the off shore mooring; where adverse environments too are bound to affect the life of the rope.

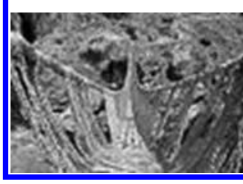

Figure 1. Last surviving hemp rope Inca Bridge.

Figure 2. Synthetic Rope used for mooring.

Figure 3. Trail Bridge in Nepal.

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Seismic evaluation of Sogutlucesme Viaduct

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ABSTRACT

Marmaray is one of the most challenging infrastructure projects in Turkey. The purpose of the project is to provide safe, comfortable and cost-effective train transportation for passengers and cargo between the Asian and European sides of Istanbul in Turkey. The project consists of three contracts namely; Bosporus Crossing Contract comprising an immersed tunnel, the Commuter Rail Contract which includes constructing a third interstate railroad and upgrading the existing two lanes commuter rail system for 100 years of service life. The third contract is related with the Rolling Stock.

The major structure within the scope of second contract is Sogutlucesme Station Viaduct. This viaduct has a particular importance for being the first aboveground Railroad Passenger Station constructed on a viaduct in Turkey, and also has special worth in both aesthetic aspects and socio-cultural life of the precinct. It comprises 6 parallel bridges with 19 axes on horizontal curve as distinct structural systems; 4 railway platform having lengths of about 500 meters and 2 passenger platforms with lengths of 400 meters. Platforms consist of prestressed precast reinforced concrete box girders. Piers are constructed as single columns of two octagonal sections varying in shape along the height. Footings are formed as spread or piled footings depending on the subsoil conditions.

The presented paper summarizes detailed investigation and design works including condition survey, structural capacity and seismic evaluation studies for the viaduct for its conformity with the state of the art design codes and new seismic design criteria revised after 1999 major earthquakes. The seismic capacities of all structural components have been evaluated by determining their Capacity/Demand ratios for two different seismic excitation scenarios having different hazard levels for return periods of 70 and 475 years, and comparing those with the acceptance criteria.

Photograph 1. Aerial view of the Viaduct.

Necessities of the rehabilitation works are also briefly summarized.

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Sustainable design for steel-concrete bridges

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ABSTRACT

As bridges are of vital importance in the worldwide infrastructure network and significant in economy, the request for sustainable structures is urgent. Sustainability of bridges means lifecycle design, but lifecycle design over a span of more than 100 years. So in contrast to sustainability issues for buildings the durability and the flexibility to adapt to changed conditions and increased traffic volume play a crucial role for the assessment of the overall lifecycle performance (LCP) of bridges. This contribution tries to draw a general concept assessing environmental, economic and social or functional quality as the 3 faces of a sustainability qualification, however in view of the special needs of bridges. First example calculations for medium span bridges in the frame of an expertise ordered by the Hessian Road and Traffic Administration to assess the cost-effectiveness of steel-concrete composite bridges, particularly with regard to the construction method, see Kuhlmann et al. (2006), have demonstrated the advantages of steel composite bridges (Fig. 1).

Three different evaluation methods for cost-efficiency are presented and the conclusion is drawn that for bridges it is not sufficient to compare only construction costs without considering construction time, neither construction method nor the type of construction.

As the construction of a bridge causes traffic hindrances it is intended to minimize these in order to keep socio-economical costs as low as possible. The effect of these costs can be taken into account following EWS, one of the evaluation methods. Similarly the aspect of reduced construction time can be considered by the approach of compensation costs according to *ARS* wherein additional costs are accepted for a shorter construction time.

Regarding bridge costs over an entire lifetime construction costs represent only a small part. Costs for maintenance, restoration, modernization as well as demolition of the bridge have to be considered as well. As these costs develop over the lifetime of the bridge, a fully integrated lifecycle model has to be derived. In the frame of a European RFCS research project SBRI (Kuhlmann, U. et al. 2009) with 9 partners among others a tool for evaluating new steel/composite bridges from sustainable perspective will be developed. This project also aims at minimizing environmental and cost impact of modern steel / composite bridges using a holistic approach which combines Lifecycle Assessment (LCA), Lifecycle Cost (LCC) and improvement of social and functional quality by a Lifecycle Performance (LCP) analysis.

Looking into the future a certain responsibility regarding sustainability has to taken into account. The criterion of sustainability has become a key issue. There is a need for sustainable long-living structures, also taking into consideration future demands such as increasing traffic volume.

The calculations and comparisons conducted in the expertise want to boost the willingness to consider various aspects of a structure within its lifespan and to take new paths assessing and evaluating bridge structures concerning sustainability and lifecycle performance.

Figure 1. Composite bridge “Dambachtal”.

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Estimating natural frequencies, damping ratios and FEM models of suspension bridges from wind response measurement

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ABSTRACT

In this paper, a novel approach to identify dynamic characteristics of structures under stochastic excitation is proposed. Following a general framework of the Stochastic Subspace Identification (SSI) technique (Van Overschee & De Moor 1996), an enhanced algorithm, which is named "Enhanced Canonical Correlation Analysis" (ECCA), to properly determine the order of a system in state variable estimate has been developed.

The SSI technique is closely related to multivariate statistical analysis. The implementation process of the SSI can be represented in two steps: the first step is to estimate the state vectors and the second step is to extract the dynamic properties from the estimated state vectors. The first step includes the determination of the system's order and can be carried out through various algorithms. Currently, three different algorithms are considered for state vector estimates, which are associated with classical multivariate techniques: Partial Least Square (PLS), Canonical Correlation Analysis (CCA), and Multiple Linear Regression (MLR) techniques (Arun & Kung 1986).

The efficiency of the proposed approach has been proven by comparing it with the existing approaches in applications to numerical data as well as field measurements of a suspension bridge, New Carquinez Bridge (NCB) in California. In such comparison, the focus is on the investigation of capabilities of the four techniques (the PLS, CCA, MLR, and ECCA) to discriminate structural modes from noise modes in terms of singular values of their corresponding matrices. Figure 1 presents the normalized Singular Value Distributions (SVD) considered in the four techniques. In stochastic realization, the dimension of the state vectors, the order of a system, has been determined only based on such SVD.

As a result, it has been demonstrated that the ECCA is quite effective in discriminating structural modes from noise ones considering the phase of the state vector estimates. In fact, in the implementation of the ECCA, the structural modes of the NCB including weakly excited modes have been identified without using a so-called stabilization diagram. Therefore, using the ECCA technique in stochastic realization could significantly reduce the need for a procedure for the selection of structural modes.

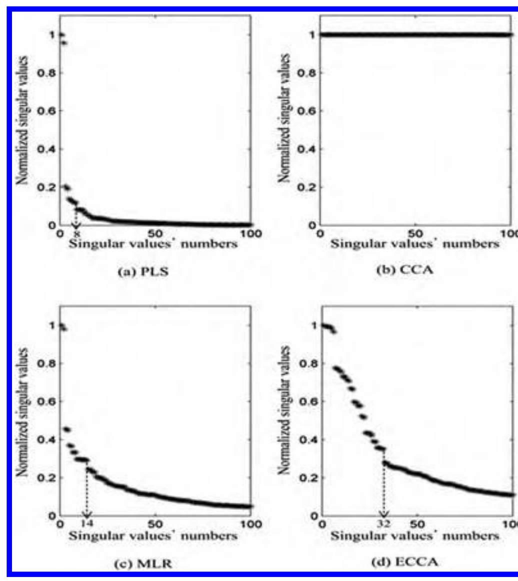

Figure 1. Field data: the normalized singular value distributions considered for the PLS, CCA, MLR, and ECCA techniques.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Using orthogonal pairs of rollers on concave beds (OPRCB) in seismic design and retrofit of highway bridges

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ABSTRACT

In this paper the use of Orthogonal Pairs of Rollers on Concave Beds (OPRCB), as a simply manufactured and installed and low cost isolating system has been studied for seismic isolation of bridges. Rollers installed in two orthogonal directions make possible the movement of the superstructure in all horizontal directions, and concave beds give the system restoring and re-centering capabilities (Figure 1).

The high vulnerability of the bridge structure, shown in Figure 2, has been discussed in a previous study (Hosseini et al. 2008).

For investigating the efficiency of OPRCB isolators the sample bridge have been considered once with its conventional support systems, and once installed on OPRCB isolators, and its seismic responses have been calculated using several accelerograms. The responses related to Imperial Valley earthquake, as a sample, are shown in Figure 3.

Based on the numerical results it can be concluded that using OPRCB isolators in seismic design and retrofit of highway bridges is very effective in reduction of seismic demands.

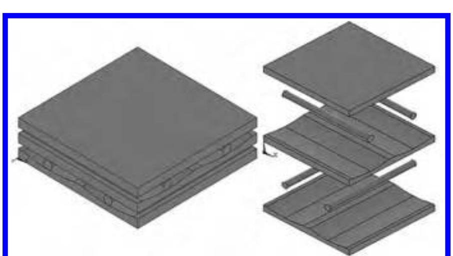

Figure 1. The main parts of the OPRCB isolators (Hosseini and Soroor 2009).

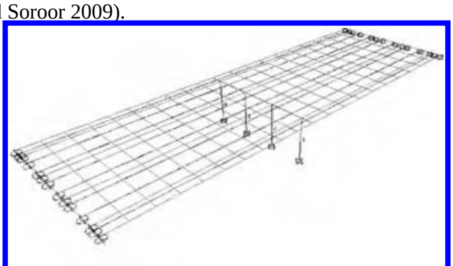

Figure 2. Computer model of the bridge for seismic evaluation.

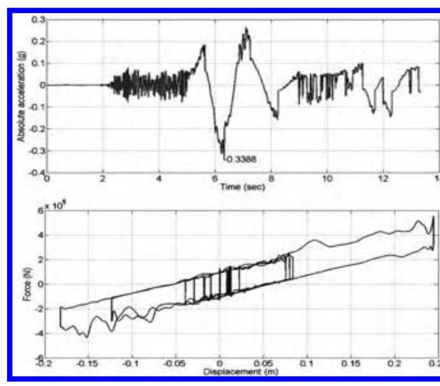

Figure 3. Acceleration time history of the isolated bridge deck and the hysteretic response of the bridge isolators to Imperial Valley earthquake scaled to 0.7 g.

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Two cable-stayed bridges designed for easy access and low maintenance

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1 INRODUCTION

Learning from access difficulties to aging bridges for inspection and repair, bridge owners are paying more attention to access and details in design of new bridges. Cable-stayed bridge is a new bridge type, which became popular during 1980s. The American cable-stayed bridges built in this period of time are showing aging and require major rehabilitations. When the design of the William H. Natcher Bridge started in 1991, the Parsons Brinckerhoff (PB) design team has paid great attention to the access and maintenance that resulted in an easy access and low maintenance bridge. Subsequent to the Natcher Bridge, PB improved the Kanchanaphisek Bridge design and made it easier for access and lower in maintenance cost than the Natcher Bridge.

Designing a long lasting and owner-friendly bridge was the goal set by the PB design team at the beginning of both projects. This means providing easy access for inspection and maintenance and minimizing maintenance items. These goal and means are achieved during conceptual design stage of both bridges.

2 BRIDGE DESCRIPTION

2.1 *William H. Natcher Bridge*

William H. Natcher Bridge has a 1200-foot main span. It is flanked by two 500-foot back spans. The bridge carries two lanes of traffic in each direction between Kentucky and Indiana. The bridge provides a 70-foot under clearance and a 1000-foot wide navigation channel for barge traffic. (Figure 1) The bridge has been opened to traffic since 2002. It has been the longest span crossing the Ohio River.

2.2 *Kanchanaphisek Bridge*

On November 15, 2007, the Kanchanaphisek Bridge crossing the Chao Phraya River in south Bangkok opened to traffic. (Figure 2) The 500 m or 1,640-foot long main span is the longest span in Thailand. It is also longer than any cable-stayed bridge spans in the United States.

3 SPECIAL DESIGN FEATURES

William Natcher Bridge crossing the Ohio River between Kentucky and Indiana and the Kanchanaphisek Bridge crossing the Chao Phraya River in Bangkok, Thailand are designed to minimizing the future maintenance cost. This new concept was developed in 1991. Low maintenance achieved by eliminating components, which require inspection, repair and replacement, or by designing components which could be removed easily for replacement. Low maintenance cost is achieved also by providing easy and ready access to all major components, such as cable anchors.

Both Natcher Bridge and Kanchanaphisek Bridge have no tie-downs, which is common to cable-stayed bridges before year 2000. Kanchanaphisek Bridge has no bearings as well. Natcher Bridge uses elastomeric pads for bearings and bumpers because they require low maintenance. All bearings and bumpers could be accessed directly and replaced easily.

Both Natcher Bridge and Kanchanaphisek Bridge have a large chamber on top of their towers to house the cable anchors. Steel cable anchorage frames become working platform not only for future maintenance work but also for original construction. The most special design is an opening in the chamber bottom slab and a ring on the roof ceiling. This allows lifting heavy replacement cable anchors directly from a truck parked on the deck up to their final positions. It also allows delivery of heavy repair equipment.

These two bridges are low maintenance examples to future cable-stayed bridge design.

Development of a cooperative management framework for bridge maintenance using IFC data model

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ABSTRACT

The bridge maintenance management benefits from various software applications which assist managers in solving a multitude of engineering problems. However, these software applications lead to large volumes of unstructured or loosely structured data with poor interoperability and reusability. For instance, Bridge Management System (BMS), which is a wide-accepted system for both project-level and network-level maintenance management of existing bridges, may be not interoperable with other tools, such as asset management tools and health monitoring systems. In order to ensure data consistency and interoperability between software applications, bridge information modeling technique is recently emerged along with the rapid development of building information modeling (BIM) and Industry Foundation Classes (IFC) data model (Lee and Jeong, 2006).

An Industry Foundation Classes (IFC) based data model, i.e. an extension to IFC, is proposed for representing the maintenance information of bridge structures. The objective of the research is to recommend a common data structure and a cooperative computer environment which enhance the information exchangeability and consistency between various computer applications in bridge maintenance management.

A generic process model is firstly investigated for representing the activities existing in the bridge maintenance process, such as performance requirement identification, condition assessment, maintenance planning, and maintenance operation management; the process model is used to identify the information items for the development of data model and is schematically described. Then a data model extension is developed for IFC, which is neutral and open data model developed within the International Alliance for Interoperability (IAI). Then, a conceptual framework model is developed to integrate bridge information model and BMS (refer to Fig. 1). The IFC data model extension is developed following the methodology similar to that used by BuildingSMART. The integration framework model brings new functions such as 3D visualization, real-time structural analysis, automatic quantification and costing to bridge maintenance management. Additionally, interoperability and data consistency between computer applications are guaranteed.

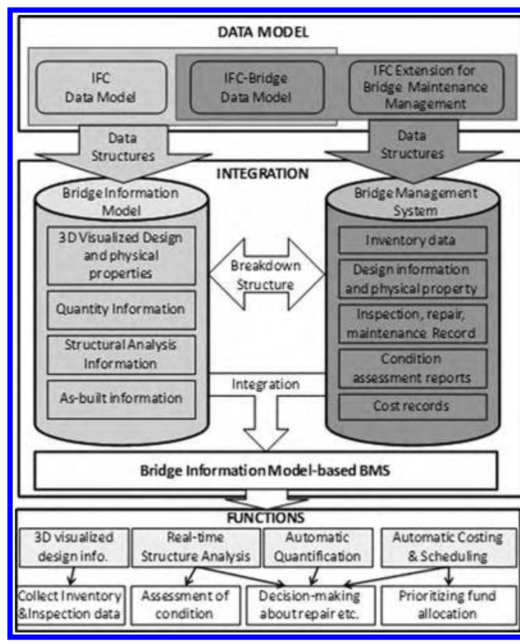

Figure 1. Coceptual framework model for BIM and BMS integration.

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Development of live load model using weigh-in-motion data

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The live load model used in the bridge design represents the effects of traveling vehicle on the bridge. The current live load model in Korea Bridge Design Code needs to be updated to consider the fast growing truck traffic and weight. Also the reliability-based code requires statistical data of the load model. This paper deals with the development of live load model using data from BWIM (Bridge Weigh-In-Motion) system and portable WIM (Weigh-In-Motion) system.

Truck weight data are collected on various sites in the area. They include highways, national roads and provincial roads. Procedures for determining the maximum load effects are determined. From the collected data, maximum weights are estimated for each truck type and sites using the extreme analysis. Only upper 10% of data are used in the estimation and assumed as having extreme type distribution. Various factors such as truck types, total weights, headway distances, and correlation of truck types and weights are considered for each lane. The maximum load effects are evaluated for different span lengths of the bridge. Multiple presence of trucks in one lane (series of trucks) and two or more lanes(side-by-side trucks) are considered. The probability of multiple presences of trucks is determined from the video recording and other studies. The results are compared with live load models from various design codes.

Two different design live load models are proposed. The first model is composed of two or three axle loads with uniformly distributed lane load. The second model is composed of five axle loads with uniformly distributed lane load. Both models are under review by designers and practitioners before the final model is proposed for the new LRFD Korea Bridge Deign Code. Multi-lane loading factors are also proposed based on the probability of side-by-side truck probability. More reliable and long-term data for truck weights and probability of multiple presences should be collected for better estimation of actual effects of truck live loading. Also live load model for very long span bridges needs additional investigations.

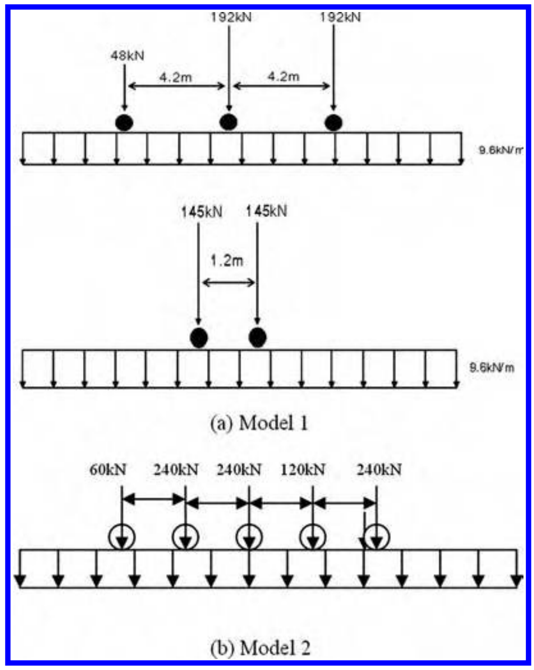

Figure 1. Proposed live load model.

Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

A review of metallic bridge failure statistics

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ABSTRACT

Studying the failures of the past can be useful in mitigating the incidence and potential of future failures. A first step towards the understanding and quantification of the risk of failure of bridges can be provided by acquiring knowledge on the failure mechanisms of existing structures and the root of causes of collapse. Studies aimed independently for different types of bridges, in terms of their material and form, may lead to the identification of predominant failure causes and modes for each bridge type. Clearly, trends picked up through statistical analysis can aid in identifying and understanding the potential of the most significant hazards affecting bridge structures and help in planning against their consequences.

In this paper, damage/failure/collapse cases of metallic bridges worldwide from the early 19th century up to the present are synthesised and classified. A database of 164 failures of metallic bridges extracted from the literature, the web and news reports was compiled. Of the 164 reported cases, 87 (51%) were classified as 'collapse', 73 (47%) as 'no collapse' and 4 (3%) as unknown. The evaluation of the database is carried out in terms of identifying the factors contributing to the failure or collapse, the modes of failure or collapse, structural form of the bridges, construction date and age of the failed bridges and the number of casualties. The paper concludes with a discussion of the significance of consequence analysis in terms of risk assessment giving some general guidelines on how consequences of bridge failure can be quantified.

The distribution of failure causes for the collapse database is shown in Figure 1. The most important factors (almost equally) contributing to collapse are design errors, limited knowledge, and natural hazards (21%). Clearly, there is no single dominant cause of collapse for metallic bridges, though the top three amount to almost two thirds of the whole. Further analysis of the database (Table 1) shows that limited knowledge played a major role in the recorded collapses up to the mid-twentieth century but decreasing considerably in recent years. On the other hand, Table 1 shows an overall increase in the bridge collapses that

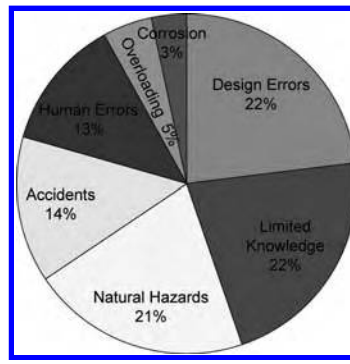

Figure 1. Failure causes leading to collapse of metallic bridges.

Table 1. Cause of bridge collapses by date of failure.

Collapse cause	Pre-1900	1900–1940	1941–1990	1991–
Limited knowledge	10 (53%)	5 (36%)	4 (13%)	0 (0%)
Design errors	4 (21%)	4 (29%)	7 (23%)	5 (25%)
Natural hazards	1 (5%)	3 (21%)	8 (26%)	6 (30%)
Accidents	1 (5%)	1 (7%)	6 (19%)	4 (20%)
Human errors	2 (11%)	1 (7%)	4 (13%)	2 (10%)

occurred due to natural hazards and accidents over the years. Remarkably, design and human errors remain constant in percentage terms during the investigated 150-year period, given the emphasis on quality assurance and other human error mitigating procedures introduced in the past thirty years.

Scour and buckling failures were found to be the most common collapse modes for metallic bridges whereas fatigue was found to be the most common failure mode in the case of non-collapsed (damaged) metallic bridges. The statistics have also shown that truss bridges are more susceptible to total collapse failures and girder bridges to failures that do not necessarily result in total collapse.

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Shear transfer mechanism in slab-on-girder bridges

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ABSTRACT

Headed shear connectors are commonly used to transfer longitudinal shear forces across the concrete–steel interface in bridges. While the connectors are mainly meant to create composite action, they restrain the free shrinkage of concrete decks and as a result cause transverse cracking. A good understanding of the composite action and interaction between stud and its surrounding concrete is very beneficial for developing a remedy for the problem of deck cracking, which is the most significant part of bridge maintenance cost in the US. The aim of this work, therefore, is to develop more insights into the shear transfer mechanism in welded shear studs to quantify its effect on bridge deck cracking. Preliminary results indicate that under gravity load shear studs uniformly contribute to transfer of shear stresses, although the end studs (farther from center) have slightly more contribution. However, under shrinkage load the end studs transfer majority of the load (Fig. 1). Furthermore, it is determined that a wider shear stud spacing can be used without significant increase in deflections.

Furthermore an innovative concept to possibly eliminate shrinkage deck cracking is briefly discussed. It may be possible to achieve this goal if a mechanism is developed such that it delays the composite action to the time that a significant portion of the free shrinkage strain has occurred. This can be achieved by using threaded studs (Fig. 2), which are not fastened for the period concrete undergoes a significant part of it shrinkage which can be limited to the time that the bridge is still not open to traffic. In this period the studs are freely slipping with the concrete and no tensile stresses are developed in the deck. Upon completing the bridge construction, all the studs are tightened so that full composite action can be achieved. Practicality of the purposed method is a matter which requires further investigation.

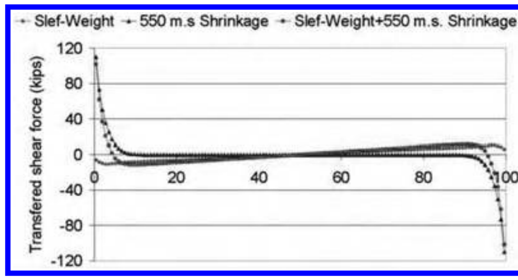

Figure 1. Distribution of shear force at deck-girder interface.

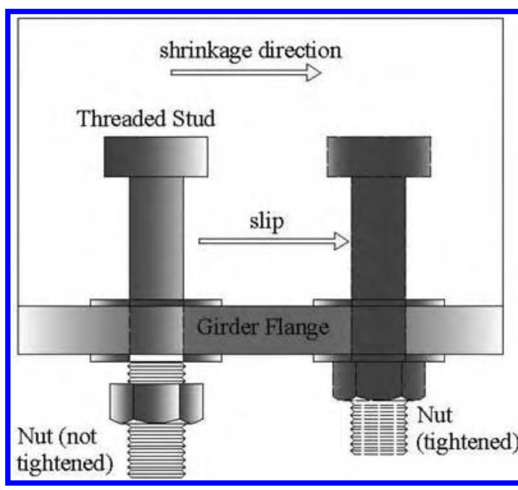

Figure 2. Distribution of shear force at deck-girder interface.

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Estimation of optimum isolator parameters for effective mitigation of seismic risk for bridges

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ABSTRACT

Seismic isolation of bridges is a design methodology that is based on limiting the magnitude of the seismic forces transferred to the substructures. The performance of seismic isolated bridges (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 will produce a more economical bridge design under seismic effects.

Seismic isolators used in bridge applications may be classified into two groups as rubber-based and sliding-based (Naeim & Kelly 1999). The force-displacement hysteresis of these isolators is generally idealized as bilinear for design purposes. The characteristic strength, Q_d and the post-elastic stiffness, k_d, are the main isolator parameters that affect the behavior of a SIB for a given ground motion with specific frequency characteristics and intensity (Dicleli & Buddaram 2006). Thus, the optimal selection of these isolator parameters based on minimizing the MID and MIF will result in an economical design of the SIB. Accordingly, the objective of this research study is to formulate closed form equations as functions of the isolator, bridge and ground motion properties to calculate the optimum Q_d and k_d of the isolator to minimize the MID and MIF. Using the developed equations it will be possible to select the proper isolator properties that will result in an economical SIB design.

To achieve the above stated objective, first, sensitivity analyses are conducted to investigate the effects of several bridge, isolator and ground motion parameters, on the optimum values of Q_d and k_d. From these sensitivity analyses, the parameters that affect the optimum values of Q_d and k_d are identified. Next, for each one of the identified parameters, nonlinear time history (NLTH) analyses of typical SIBs are conducted to determine the optimum values of Q_d and k_d for an assumed range of values of the parameter under consideration. Next, the available data is plotted as a function of some dimensionless parameters proposed by Makris& Black (2004a, 2004b). Nonlinear regression analyses of the plotted data are then conducted to obtain closed form equations for the optimum values of Q_d and k_d, to minimize the MID and MIF. The obtained closed form equations are then verified using a suite of ground motion data different than that used for the development of the same equations. It is observed that the optimal Q_d is proportional to peak ground acceleration of the ground motion and bridge mass and inversely proportional to the peak ground acceleration to peak ground velocity (A_p/V_p) ratio of the ground motion. It is also found that the optimal k_d is proportional to peak ground acceleration of the ground motion and bridge mass, inversely proportional to Q_d and is a polynomial function of the A_p/V_p ratio of the ground motion.

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Development of a new cycle counting method for cyclic thermal strains in integral bridge piles

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ABSTRACT

The daily and seasonal temperature changes result in imposition of cyclic horizontal displacements on the continuous bridge deck of integral bridges and thus on the abutments, backfill soil, steel H-piles, and cycle control joints at the ends of the approach slabs. Due to these seasonal temperature changes the abutments are pushed against the approach fill and then pulled away, causing lateral deflections at the tops of the piles that support the bridge as observed from Fig. 1 (French et al. 2004). 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. As the length of the integral bridges gets longer, the temperature-induced cyclic displacements and forces in steel H-piles components may become larger as well. This may result in the reduction of their service life due to low-cycle fatigue effects (Dicleli & Albhaisi 2003, Arsoy et al. 2004).

In this study, the field measurements obtained for integral bridges are used to determine the amplitude and the number of temperature induced cycles on steel H-piles in integral bridges. Using the obtained measurements, the number of large strain cycles per year due to seasonal temperature changes and the number and relative amplitude (relative to the amplitude of large displacement/strain cycles, i.e. $\beta =$ small strain cycle amplitude / large strain cycles amplitude) of small strain cycles per year due to daily or weekly temperature changes are determined. Additionally, the number of small cycles (secondary cycles) between the maximum and minimum cycle above and/or under the large strain is counted. Using the available data on the number and amplitude of temperature induced displacement-strain cycles, a new cycle counting method is developed to determine the number and amplitude of large and small displacement/strain cycles (small strain cycles are composed of primary and secondary strain cycles). Then, a new equation is obtained to determine a displacement/strain cycle

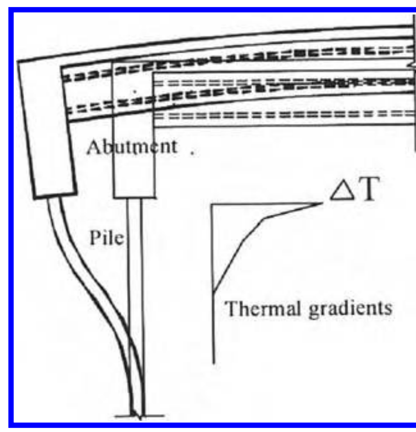

Figure 1. Pile displacement due to thermal gradient.

amplitude representative of a number of small amplitude cycles (primary and secondary) existing in a typical temperature induced displacement/strain history in steel H-piles of integral bridges.

It is found that, the secondary strain cycles have a negligible effect on low cycle fatigue life of steel H piles in integral bridges.

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Properties of self-consolidating light-weight concrete in massive structures

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ABSTRACT

Structural light-weight high performance concrete has been used in bridges for several years as the self-consolidating concrete. The latest trend is the utilization of Self–Consolidating Lightweight High-Performance Concrete in bridge engineering. This concrete combines the favorable properties of lightweight high-performance concrete (LWC) with those of self-consolidating concrete (SCC) and offers an excellent solution for such applications.

This paper presents test results of hydration heat, workability properties and mechanical properties (compressive strength, tensile strength and modulus of elasticity) of light-weight self-consolidating concrete, hardening under adiabatic conditions (to simulate the true conditions of concrete hardening inside of a massive structure) and isothermal conditions. The influence of curing conditions on mechanical properties of LWSCC were considered.

The tests were performed for concrete mixtures made using Portland cement CEM I 42,5R (European Standard EN 197-1:2000), fly ash, silica fume, Sika Viscocrete 3 superplasticizer and two type of lightweight aggregate: Pollytag size of 4–8 mm and 0–4 mm, Liapor size of 0–2 mm and 4–8 mm, natural sand 0–2 mm and natural coarse aggregate size 2–8 mm. A constant amount of paste was assumed with a variable proportion of light-weight aggregate to natural aggregate. The natural aggregate was replaced by the same volume of the light-weight aggregate. Compressive strength was tested on cube specimens 150 × 150 × 150 mm. Splitting tensile test was carried out on cube specimens 100 × 100 × 100 mm. Modulus of elasticity was tested on standard cylinders 150 × 300 mm.

The relationship between the compressive strength and density of concrete after 28 days of hardening, obtained from tests is shown in Figure 1.

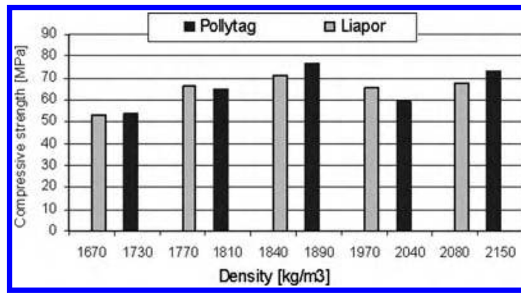

Figure 1. Compressive strength and density of concrete after 28 days of hardening.

The relationship between the compressive strength of concretes hardened under isothermal conditions (laboratory) and concretes hardening under conditions simulating the curing in massive structures, for concrete with Pollytag aggregates and Liapor aggregates is presented in Figure 2. The relationship is approximated using a logarithmic function.

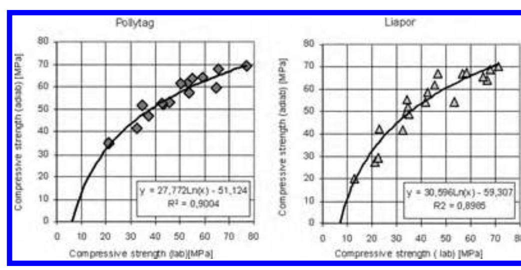

Figure 2. Relationship between the compressive strength of concretes hardened under isothermal and adiabatic conditions.

The developed concrete mixes have good self-consolidating properties, density lower than 2000 kg/m^3. The required properties of concrete strongly depend on the fabrication procedure, including initial preparation of aggregates, dosage method and mixing of components. The light-weight self-consolidating concrete requires a careful selection of the aggregates.

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Seismic vulnerability assessment and comparing various retrofit designs for an existing highway bridge

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ABSTRACT

In this paper, having the high vulnerability of the bridge proven in a previous study of the authors (Hosseini et al. 2008), three retrofit designs, including: 1) increasing the strength and stiffness of the bridge by adding the fixity of deck connections to abutments in lateral direction and using diagonal bracing elements at one end of the deck beside the bridge abutments in longitudinal direction, 2) adding some diagonal braces between columns in transverse direction and at one end of the deck beside abutments in longitudinal direction, and 3) using dampers in diagonal members between piers in lateral direction and also at one end of the deck in longitudinal direction were investigated for selecting the most appropriate retrofit technique (Figure 1).

To find out if these retrofit techniques are adequate, in addition to another set of POA (Hosseini et al. 2008), the results of which can not been presented here because of lack of space, a set of Nonlinear Time History Analyses (NLTHA) was performed by using some appropriate accelerograms. To get a better insight into the differences between the three retrofit design and also the effect of damping value of the dampers used in the third retrofit design the maximum displacement responses of the retrofitted structures, in both longitudinal and lateral directions, and also those related to different values of damping coefficient of the used dampers in the third retrofit design were obtained. Figure 2 shows a sample of these results.

Based on the numerical results the third design leads to lower response peak values. However, the final decision should be made based on technical and financial considerations.

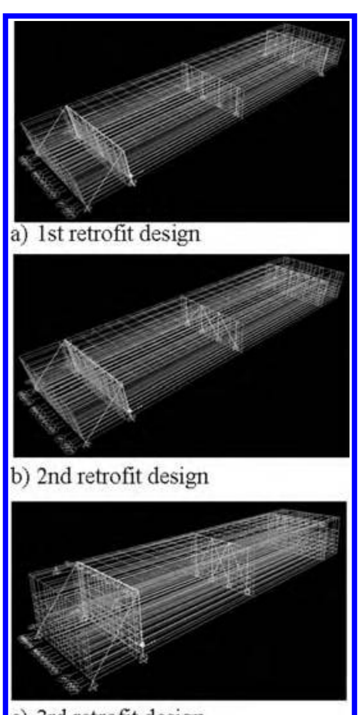

Figure 1. The three proposed retrofit designs.

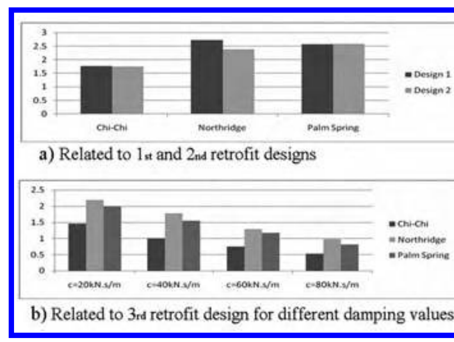

Figure 2. The maximum values of displacement response histories (cm) of the retrofitted bridge with the three designs.

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Experimental evaluation of spliced PSC-steel-PSC hybrid girder connected with advanced joints consisting of perfobond ribs

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ABSTRACT

A spliced I-girder bridge is to connect prefabricated segments which are made in the factory and transported to a site. This girder usually uses PSC segments since it can be easily connected by tendons. However, this study focuses on the steel-PSC hybrid system. Thus, it is necessary to study the joint which can be applied to both spliced concept and hybrid system.

This study proposes the joint consisting of multiple perfobond ribs in connecting spliced steel and PSC segments. The test beam consists of a steel component in the center, two PSC components at both ends, and two joints connecting the steel and PSC components. However, joints are embedded into the PSC components by casting concrete to the PSC and joint at the same time. Thus, two spliced PSC segment and one steel segment is fabricated in a factory. Then, these segments are transported to a test site and assembled into one girder. The spliced steel segment can be connected to the joint part of PSC segment by bolting, since the joint also has a connecting steel section.

For the spliced PSC-steel-PSC hybrid I-girder with a length of 40m, a full scale test is conducted under three-point loading as shown in figure 1.

The first crack is visually detected at the interface between the end of lower steel plate in joints and PSC components. As the load increases, typical flexural cracks propagate to the top surface of the girder. In the final state, tensile failure at the bottom occur in the PSC segment, whereas the joint part do not failed. The load-displacement curve at the center of the steel-PSC hybrid test beam is plotted in figure 2. The displacement responses are linear until the first crack forms. After the first crack occurs, nonlinear behavior is initiated due to the crack propagation. The ultimate load is 1,812 kN, corresponding to displacements of 601 mm.

Experimental results show that the ultimate strength of the spliced hybrid girder is determined by the strength of the PSC part. In addition, the spliced hybrid girder shows sufficient ductility ratio, even though perfobond ribs are considered as stiff shear connectors. Therefore, the spliced hybrid PSC-steel-PSC girder

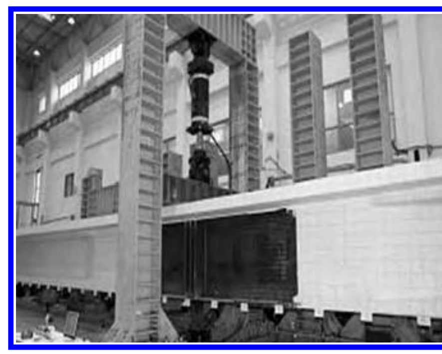

Figure 1. Photograph of test set-up.

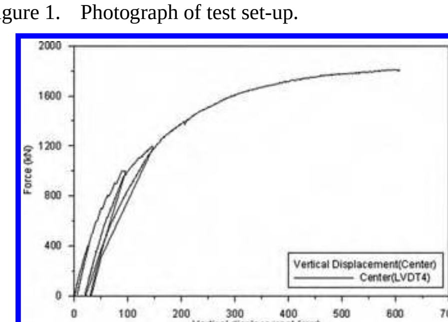

Figure 2. Load-displacement curve at the center.

with joints consisting of perfobond ribs is found to be an efficient system in order to guarantee sound performance up to the ultimate capacities of both the PSC and the steel girder.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Construction of the hybrid cable-stayed bridge

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ABSTRACT

'Cheong Poong' Bridge is hybrid cable-stayed bridge, which has main span of 327meters crossing over the lake 'Chung Ju', Korea.

The bridge has spans of 57.5 m + 327 m + 57.5 m, which makes the uplift reaction by the unbalance of the span composition. Therefore, Hybrid system that is the combination of composite section (reinforced concrete deck and I-shaped steel girder) for the center span and concrete section for side spans has been adopted, accepting the decreasing effect of weight balance. And the bridge is located at inland unusually in Korea for cable-stayed bridge, floating crane and large barge which are needed for erection steel segment can not access inland. So Construction method of girder by girder and panel by panel erection is applied and weight of erection unit can be controlled in less than 200 KN, consequently applying the light weight of derrick crane. Concrete pylon height with typical H-shape is 103 m and constructed by auto climbing form. The main span of 327 m across the lake is constructed using the free cantilever method and the side span of 57.5 m is constructed by the full staging method.

The Construction of the foundations and pylons started in March 2006 and superstructure construction began in April 2007. It is now under construction and expected to open to traffic at year of 2010. The 'Cheong Poong' cable-stayed bridge would play a role of symbolic landmark for the tourism development in Chungcheong province, Korea as well as of transportation by replacing the existing concrete girder bridge.

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Figure 1. A view of Cheong Poong Bridge.

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Low cycle fatigue strength of cruciform welded joints considering plate thickness

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ABSTRACT

Welded joints of steel bridge piers may have thick plate thickness which could reduce fatigue strength of welded joints. Hence, it is necessary to grasp the influence of the plate thickness on fatigue strength of welded joints. In this study, the influences of the plate thickness ranging from 25 mm to 40 mm on low cycle fatigue strength were investigated by bending fatigue tests under large plastic strain on cruciform welded joints.

The test set up is shown in Figure 1. Span was 450 mm, and three-point-bending tests under displacement controlled conditions were done. To conduct the alternating loading condition, we upset the specimen for every cycle. Applied plastic strain range at weld toe was over 10%, which was based on previous studies (for example, Sakano et al. (1997)). However, measuring the plastic strain range at weld toe was difficult because of the limit of strain measurement. So, we carried out finite element method (FEM) analyses before fatigue tests in order to obtain the relationship between local plastic strain range at weld toe and plastic strain range at the location of measuring strain gauge, which was located in 5mm apart from the weld toe.

As a result, we can apply the local plastic strain at the weld toe over 10% when the plastic strain at strain 5mm apart from the weld toe is 5%. Moreover, it can be expected that fatigue strength of specimens with 40 mm thickness is lower than that of specimens with 25mm thickness.

From fatigue test results, the cracks on the specimen with both 25mm and 40 mm thickness initiated from the weld toe, and these cracks propagated widely. This result indicates that the fatigue failure was governed by crack initiating from the weld toe. The influences on low cycle fatigue strength of plate thickness were examined using the relationship between plastic strain range and the number of cycle as shown in Figure 2. Figure 2 shows the low cycle fatigue strength at the number of cycles when 5 mm crack was detected on specimens. Figure 2 shows that scatter of both results is small. Furthermore, by increase of the plate thickness, the fatigue strength decreased approximately by 40%. Consequently, the increase in the plate thickness may affect the low cycle fatigue strength of the cruciform welded joints. This result is reasonable because the local strain increases with increase in the plate thickness as mentioned in the results of FEM analyses.

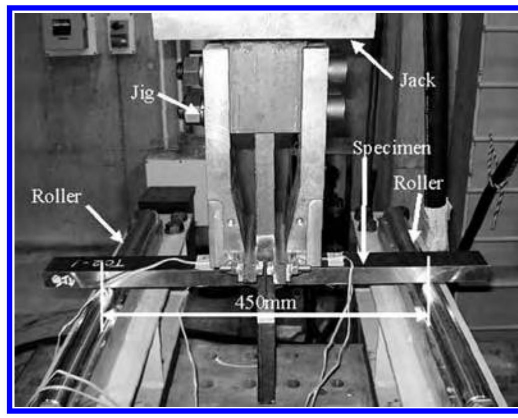

Figure 1. Test set up.

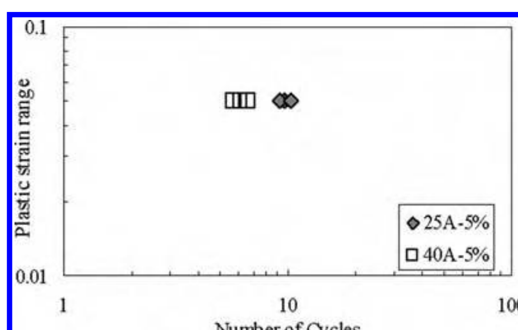

Figure 2. Low cycle fatigue test results.

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An analysis on the torsion of hybrid bridges with corrugated steel webs considering tensile strength of concrete

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ABSTRACT

Even though composite box-girders with corrugated steel webs are widely used in civil engineering practice, the torsional behavior of composite box-girders with corrugated steel webs is not fully understood yet. Mo et al. (2000) studied the torsional behaviors of the composite box-girder with corrugated steel webs. They developed the analytical model for torsional behavior of the cross section of the composite box-girder with corrugated steel webs. Then, Mo et al. (2006) proposed the torsional design procedure for the composite box-girder with corrugated steel webs based on the analytical model proposed by Mo et al. However, the tension behavior of concrete is not considered in their analytical model so that the initial stiffness is generally underestimated. In addition, their analytical models only applicable to the rectangular section.

In this study, analytical model proposed by Mo et al. is modified for more general case.

At first, the relationship between stresses of reinforcement and concrete was established by the space truss analogy to express the effective thickness of the slab and crack angle in terms of each stress. The principal tensile stress was not neglected in this relationship to fully consider the property of concrete. Then all strains of reinforcement and concrete were expressed as equations of the effective thickness of the slab and crack angle using compatibility conditions.

For the constitutive law, this model is considered the softening effect for the compressive behavior and the tension stiffening effect for the tensile behavior in concrete slab.

Through these equilibrium equations, compatibility conditions and the constitutive law, the relationship between external torque, internal stresses, internal strains and twist angle can be established.

In this study, to analyze the torsional behavior of the composite box-girder with corrugated steel webs by proposed method, the analysis program having an iterative algorithm is developed. For discretized points of maximum surface strain, assumed values of effective thickness and crack angle are updated iteratively by all equations established in each step. Final result can be acquired pictorially by plotting all points for each maximum surface strain discretized.

The developed program is successfully verified by comparing with the experimental result, and it is found that the analytical result shows good accuracy for the torque-rate of twist relationship by considering the effect of tension behavior of the concrete.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Application of a new metal spraying system for steel bridges. Part 5. Analysis of the corrosion prevention mechanism of the system

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ABSTRACT

An outline of a new metal spraying system that has been applied to steel surface at room temperature, the corrosion inhibition effect of the system, and the corrosion prevention mechanisms of the zinc-aluminum pseudo alloy-sprayed steel structures had been introduced in IABMAS'04 and IABMAS'06.

Furthermore, the reference service life prediction based on the actual condition of steel bridges applied the metal spraying system had been reported in IABMAS'08.

Japan often experiences high humidity and hot temperatures, especially during the summer. Therefore, metallic corrosion prevention techniques are important requirements for the steel construction projects. With this in mind, a new metal spraying system was developed.

Conventional galvanized steel has been used as a corrosion prevention method for many years. However, such technique mars the beauty of the scenery, and it does not have sufficient durability in a serious corrosive environment. If a paint coating is applied to a galvanized surface, it will not have sufficiently hardened film adhesion.

Further, conventional metal spraying systems require stringent measures to prepare a clean surface, and they must operate at a high temperature more than the melting point of the sprayed metal. Therefore, conventional metal spraying systems require a high- grade blasting process for steel surface preparation, and this technique cannot be used on covered surfaces such as concrete or mortar, which have poor heat resistance.

The new metal spraying system, a useful metallic corrosion prevention method suitable for some steel bridges, has been experimentally evaluated. The sprayed complex zinc and aluminum film was pronounced to offer corrosion resistance in excess of steel galvanized with 550 g/m^2 of zinc. Adhesion of the sprayed metal is very stable with a surface preparation method that coats the primer, which is a substratum surface preparation material.

Some kind of metal-sprayed specimen had been exposed the harsh seashore for 8 years and a half. In this paper, the evaluation results that include surface visual observations, electrochemical potential measurements of the exposed metal surface, scanning electron microscope observations of the specimen section, and electron probe microanalysis of the specimen surface and section are reported. On the basis of the above results, the corrosion prevention mechanism with the metal spraying system is discussed in comparison with other sorts of sprayed metal. Main fruits of this research are the following.

(1) It is judged that zinc-aluminum pseudo alloy-sprayed film that consisted of a mixture of both metals maintains least noble potential on the basis of the measurement of the electrochemical potential.
(2) It is considered that zinc-aluminum- sprayed film has sufficient effect of anodic protection working according to corrosion occurring in the X-cut part.
(3) It is recognized that aluminum or zinc-aluminum-sprayed film, even if nonpaint coating, has corrosion surrounding interception effect that restrains chlorine penetration. Meanwhile, zinc-sprayed film without paint coating is not expected to maintain the corrosion surrounding interception effect. Addition of the paint coating to the zinc film is expected to maintain the above effect. However, it is important that inhibition of the chlorine penetration with combination of sealing materials and paint is discussed.
(4) It is considered that aluminum in the sprayed film is not easy to be oxidized based on oxygen analysis in the sprayed film. Zinc in the zinc-aluminum pseudo sprayed film has less oxygen than zinc single sprayed film. Therefore, it is presumed that zinc oxidation is prevented by mixed aluminum. And, it is judged that aluminum contributes to restrain oxidation consumption of the sprayed film that maintains corrosion inhibition effect.

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Assessment of existing steel structures – Recommendations for estimation of the remaining fatigue life

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ABSTRACT

In many countries and regions, the traffic infrastructure projects suffer from low funding. There budget is tight for new infrastructure building and, thus, the importance of inspection, maintenance and assessment of the existing traffic infrastructure increases. A new fatigue assessment guideline for the estimation of the remaining fatigue life of steel bridges has been written by technical committee 6 from ECCS [Kühn et al. 2008]. It will be a useful tool for the complementation of bridge management systems, used commonly for condition assessment.

Design specifications and rules are harmonised throughout Europe. They are under constant development, but there is still a lack of forwarding and concentrating experiences as well as developing rules for the fatigue assessment on existing steel structures. This paper presents a guideline with a proposed fatigue assessment procedure for existing steel structures embedded in information about old materials and non-destructive testing methods for the evaluation of details. Particular attention is paid on remedial measures which are proposed for weak details and damages caused by fatigue. The developed fatigue assessment procedure can be applied to existing steel structures under cyclic loading in general, but the guideline concentrates on the existing traffic infrastructure made from old steel, because of the public importance. The proposed procedure summarizes, regroups and arranges the knowledge in the field of assessment on existing steel to be applied by practicing engineers. The procedure is a milestone in knowledge transfer from a state of scientific knowledge to state-of-the-art.

According to the agreement between the Joint Research Centre (JRC) of the European Commission in Ispra and the European Convention for Construc-tional Steelwork (ECCS) a series of reports are publisher as Joint-JRC-ECCS-Scientific Technical Reports, that may be downloaded from the e-bookshop of the Commission, see http://eurocodes.jrc.ec.europa.eu.

These Technical Reports are being prepared by experts from CEN/TC 250/SC3, the CEN-Technical Committee responsible for the preparation and further development of Eurocode 3, and from ECCS Technical Committees related to the subject in question. They aim at:

– Giving the scientific background and further explanations to the Eurocode 3 rules as published,
– Presenting the state of the art and preparing the field for the maintenance, further harmonisation and further development of the Eurocode 3 rules.

A high priority project of CEN/TC 250 is to include in the Eurocodes technical rules for the assessment and retrofitting of existing structures, which becomes more and more important in the context of sustainable development in the construction field. To this end the presented JRC-ECCS-Joint Report has been published.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Computational methodologies for the blast vulnerability assessment of steel bridge girders

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ABSTRACT

A challenge for bridge security is to efficiently develop a thorough risk assessment accounting for numerous bridge assets within an integrated transportation network, and prioritize those assets for the allocation of blast retrofit/mitigation funding. Typically, a risk assessment includes quantification of the vulnerability of tens to hundreds of individual bridge spans and numerous associated explosive threats. For these assessments, it is desirable to have a robust, fast-running, computational tool that can give an accurate scoping value for the blast vulnerability of the primary structural members. These scoping calculations populate risk ranking strategies like those recommended in AASHTO (2002) and FWHA/AASHTO (2003). Once scoping assessments have been made and risk prioritization has been set, only the highest risk spans are reevaluated with higher-fidelity models.

This paper presents study of a bridge girder response using both a fast-running SDOF (Single Degree of Freedom) dynamic modeling technique as defined in dynamics textbooks (Biggs 1964) and recommended in military handbooks (UFC 3-340-01 2008 & UFC 3-340-02 2002) and a non-linear, dynamic, 3-dimensional fully-coupled, finite element model in ALE3D (Nichols 2007). The blast response of a representative steel bridge girder shown in Figure 1 is computed and discussed for the two methodologies.

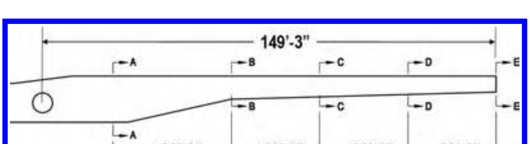

Figure 1. Sample bascule bridge girder used in the study.

This comparison identifies the relationship between the scaled range, Z, and the girder response for both the fast-running SDOF model and the high-fidelity finite element model. The results indicate that the flexural response of a long-span, stiff, girder exposed to a man-portable threat may not govern the solution at scaled ranges once thought to be compatible with such an assumption. Future analysis will examine the trade-offs between the time it takes to perform a high-fidelity model and the advantage to the additional refinement as well as the ideal scaled range for getting the most accurate SDOF results.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Design live load for long span bridges

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ABSTRACT

The available live load models were developed for short and medium span bridges. The present paper deals with the development of live load for long span structures. The maximum loading for a short span bridge is caused by the heaviest possible trucks that can travel over it. In contrast to short and medium spans, long span live load must be considered as traffic mix, where one heavily overloaded truck does not have significant influence. The roadway is not necessarily entirely covered by the heaviest trucks. As the load length increases, the load per unit length will decrease and its distribution will be closer to uniform, because cars and other light vehicles are injected into the traffic stream. For long span bridges critical scenario is traffic jam situation with vehicles moving at crawling speed and very small headway distance.

The number of trucks and participation of heavy vehicles in the traffic patter have a trend of rapid growth. Higher percentage of trucks runs overweight, particularly because it is to their economic advantage. The design live load based on actual weigh-in-motion data reflects current traffic patterns, quantities of trucks and their weights. The available data base includes weigh-in-motion (WIM) truck surveys and videos of the traffic jam situations taken on selected bridges. The weigh-in-motion database was obtained for a variety of sites from the project NCHRP 12-76.

The available WIM data served as a basis for simulation of a traffic jam situation. Values of uniformly distributed load were derived for a wide spectrum of span lengths between 180 m and 1500 m (600 ft and 5000 ft). Trucks were kept in actual order, as recorded in the WIM surveys. Results of the simulations were plotted as a cumulative distribution function (CDF) of uniformly distributed load for considered span lengths using all results, daily maximum and weekly maximum combinations. The mean value of uniformly distributed load oscillates between 7.30 and 10.95 kN/m (0.50 and 0.75 k/ft). The bias factors (ratio of mean to nominal) were calculated for the heaviest 75-year combination of vehicles. The 75-year uniformly distributed loads were derived from extrapolated distributions. It was noticed that the bias factor values for some sites do not exceed 1.25, which is similar as for short and medium spans, as shown in the NCHRP Report 368 (1999). It is recommended to use HL-93 also for those long spans. Brides in localizations with high ADDT and high percentage of overloaded trucks require development of site specific models. For some sites with very heavy traffic, the bias factor reaches value 2.0. Therefore, for those bridges the uniformly distributed load should be higher. The coefficient of variation is calculated from the slope of transformed CDF from weekly maximum values.

The developed live load model is recommended to be taken into consideration in the AASHTO LRFD Bridge Design Specifications.

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Management of interstate 95 in Pennsylvania

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ABSTRACT

The Interstate highway system is the primary transportation network in the United States. On the eastern coast, Interstate 95 links Maine to Florida. The segment through Pennsylvania is 82 km in length and consists of numerous structural transportation assets including 209 bridges, 13 dynamic message sign structures and 28 traffic incident cameras. Effective, proactive administration of bridge assets on this corridor is essential to continued safe traffic operations on the Interstate.

The key aspects of Pennsylvania's I-95 corridor management approach include bridge administration, inventory of the bridge assets, maintenance and preservation of the bridges, and the planning for improvements of the corridor which have become the fundamental ingredients for the successful support of the vital Northeast region infrastructure component.

Bridge administration is a broad umbrella effort covering many functional activities, including bridge evaluation by inspection and load rating analysis, performing on-demand structural repairs, preservation, and routine maintenance to realize the maximum useful life of a bridge possible before its eventual replacement.

The bridge asset inventory is obtained and updated as changes are identified through the bridge safety inspection program at the element level. The capture of element level data provides detailed information on the deficiencies and modes of distress contributing to the overall condition ratings, appropriate repair strategies, quantities or extent of distress and associated costs. This allows for improved asset management decisions.

Maintenance and preservation of the bridges are key to uninterrupted operations. The objective is continually performing routine maintenance activities, e.g., water blasting of beams, structural plating or patching, to extend the service life of the structures until corridor reconstruction occurs. The need to perform emergency repairs occasionally occurs, however, and must be dealt with while maintaining the confidence of the traveling public.

Planned improvement projects in the corridor consist of projects ready for construction, projects that are mid-term and long range projects. For the I-95 corridor, in the near term, approximately $350 million in construction will be bid for a variety of structural and interchange improvement projects. Over the next ten years approximately $2 billion in construction projects will occur. The long range plan of projects occurring in the next 20 years and beyond contain forecasts for an additional $4 billion in construction.

The over-arching goals of PennDOT's asset management strategies with an emphasis on a bridge program including Interstate 95 are:

- Ensuring that bridges are safe for the efficient movement of people and goods.
- Reducing the backlog of bridge deficiencies
- Using good practice in design, construction, and maintenance to sustain the continual improvement of our bridges in a cost effective manner and achieve a bridge life of 100 years.

The approach and strategies employed in Pennsylvania to manage the I-95 corridor will ensure the goals of safety, mobility, quality of life for the Philadelphia metropolitan region.

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Monitoring of the Manhattan Bridge and interferometric radar systems

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ABSTRACT

Presented herein is a monitoring and measurement study of the vertical and torsional displacements of the midspan of the Manhattan Bridge using Interferometric Radar and Global Positioning Systems (GPS). The Manhattan Bridge is a particularly interesting case study due to its immense size, unique loadings, high traffic volume, age, and recent multimillion dollar rehabilitation and stiffening program. The main span of the Manhattan Bridge is 1470 ft with an overall anchorage-to-anchorage length of 2920 ft. The current configuration includes 7 vehicular traffic lanes and four subway tracks.

The Interferometric radar system is an non-contacting, innovative microwave radar sensor (IBIS-S) used to simultaneously measure the displacement and vibration responses of multiple locations of a structure from distances up to 0.5 kilometer. The IBIS-S system has a maximum accuracy of 0.01 mm (0.0004 inch) and a maximum sampling frequency of 200 Hz (Nyquist frequency of 100 Hz).

GPS systems use triangulation from satellite signals to accurately locate the absolute position of a receiver and are routinely used in a variety of applications. The systems were employed to measure the midspan deflections of the bridge under normal automobile and train traffic loadings. The GPS data characterizes the maximum deflections as well as deflection time histories simultaneous at 80 points along the midspan.

Figure 1. Manhattan (foreground) and Williamsburg (background) Bridges.

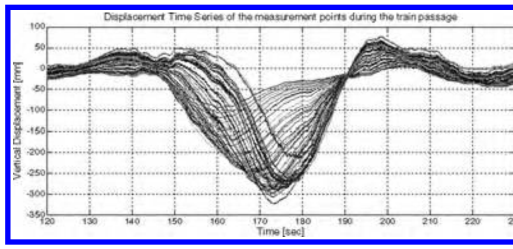

Figure 2. Vertical displacement time series data of the transverse beams measured by the IBIS-S system during train passage.

The total amplitude of displacement, peak to peak was on the order of 300 mm when loaded by a single train. Both measurement systems (GPS and Interferometric Radar) compared well with one another and are promising technologies in the evolving area of bridge deflection and vibration measurements. The GPS system lends itself to long-term monitoring while the IBIS-S system can be rapidly deployed for short-term displacement and vibration monitoring. Both systems have advantages over traditional bridge instrumentation (e.g. strain gauges and accelerometers) in terms of the ease of deployment and parameters measured (displacement).

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A rational partial composite bridge beam transition

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ABSTRACT

The New Zealand Transport Agency Bridge Manual requires composite steel girder-concrete slab bridge beams to be assessed to the NZS 3404 Steel Design requirements. To account for shear connector fracture this standard limits the shear connection capacity for partially composite beams to 50% of that required for a fully composite beam. This criterion is unduly severe often requiring load restrictions to be applied and this typically does not correlate with bridge performance. Simplified plastic analysis methods for determining the flexural capacity of a steel girder-concrete slab bridge beam require the shear connector strength determined for the typical static failure modes, being concrete bearing and shear connector shear strength, either in the gross section or fixing to the steel girder flange. Shear connector fracture is also a critical limit state through the partial composite beam transition that needs to be appropriately assessed.

The paper summarises “desk top research” reviewing trends in the flexural capacity versus shear connection capacity interaction curves for composite beams with the neutral axis in either the concrete slab or the steel beam. This review has shown the following:

- Reliability of the shear connector capacity can be used to model the shear connector fracture limit state through the transition
- A discrete smoothed empirical transition model is rational, repeatable and gives consistent reliability to fracture failure, refer to Figure 1
- Correlation to the shear connection limits specified in international standards is met
- For NZS 3404 Steel Design an approximate 20% increase in load carrying capacity can be achieved with this typically negating the need to load restrict bridge beams

The method has been specifically developed for application to an extreme positioned overload vehicle load case where fatigue effects can be ignored.

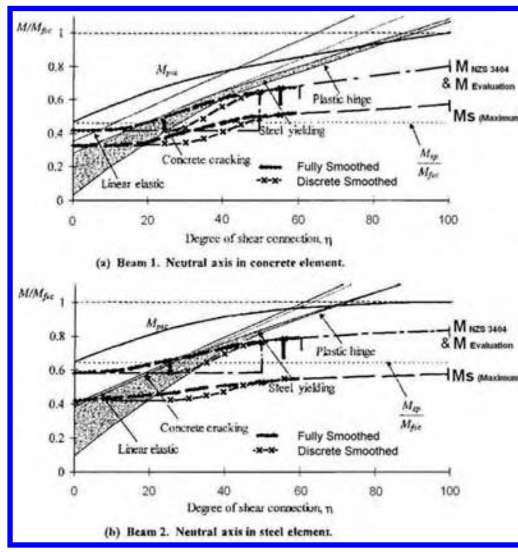

Figure 1. Transition Interaction Curves (Burnet and Oehlers, Fully Smoothed and Discrete Smoothed) for two beam cases.

Calibration of the model is limited and further testing is suggested to confirm the appropriateness of the evaluation procedure.

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Structural health monitoring for bridge management

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ABSTRACT

Structural Health Monitoring (SHM) is a new technology in bridge and structure engineering and its integration into bridge and structures management has not yet been determined. The technology involves tracking of any aspect of a bridges health by using reliably measured data and analytical simulations in conjunction with heuristic experience so that the current and expected future performance of the bridge for at least the most critical limit events can be described in a proactive manner. SHM has clear advantages over traditional Non Destructive Testing and Evaluation because once the monitoring is installed data flow is continuous.

Historically bridge management was focused on defect management with inspection programmes supporting the development of maintenance programmes. Performance measures for bridges are being developed as a management tool allowing the effectiveness of work programmes to be reported. The performance measures developed to date utilize qualitative condition information, rather than defect information, and will continue to evolve as the industry better understands and defines the holistic asset management inputs required to maximise the effectiveness of limited budgets and be able to present the outcomes to the wider community. Performance Indicators for bridge Essentiality, Functionality, Serviceability, Safety and Stability are being considered. Structural indicators being considered include Condition, Reliability, Risk and Work Record. The vision is for structural performance to be based on two prime factors, a risk indicator presenting structure strength and a condition indicator.

It is recognized that SHM is an evolving field and has matured significantly over the last decade with a focus on developing reliable means of acquiring, managing, integrating and interpreting reliable structural data for maximum information gain and at lowest cost. The paper also recognizes the importance of having a comprehensive suite of SHM tools for Bridge Managers to effectively manage their bridge stock. In this paper an example SHM programme is outlined for a typical high risk durability site to show how the SHM could be integrated into the bridge management reporting. From this example it is observed the greatest benefits for a proactive – preventive maintenance bridge management regime are associated with the basic level interrogation of the SHM data. As it is recognized that SHM should not be considered as equating to a structure evaluation assessment it is considered the extent of information gains from SHM needs to be reviewed to ensure SHM is cost effective.

Recent research has shown that risk based condition measures correlate well with bridge funding investment and it has been determined that if basic interpretive data from an SHM programme can be integrated into the performance measures then more robust data will be available and SHM will be more readily justified. The paper considers these matters presenting options for integrating SHM data into the full range of likely performance indicators. With the development of robust performance indicators informed and transparent bridge management decision making will be achieved, and, enhanced bridge management reporting made possible.

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Destructive test of a steel slab-on-girder bridge

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Larson Design Group

ABSTRACT

This paper presents the testing procedures and results from a unique destructive bridge test of a steel I-girder bridge. The subject bridge was in good condition and only decommissioned due to traffic pattern improvements. The goal of the test was to load the structure well beyond its elastic limit in an attempt to observe transverse moment redistribution and investigate the ultimate capacity of the structure. Instead, the test revealed the tremendous reserve capacity of the structure. The bridge was able to withstand a loading mimicking a design truck with a load equal to 17 times an HS-20 vehicle without the data indicating any yielding of the girders or any other significant damage being observed.

A second usefulnesses of the test data is quantifying the load distribution throughout the structure. As an initial step to quantifying the load distribution throughout the structure, the strains through the cross-section of the girders are analyzed. Figure 1 shows such strains at four (nearly even) intervals of increasing load at a given girder cross-section, near the center of the footprint of the applied loading. Each cycle is the loading equivalent of one HS-20 vehicle (or 72 kips). It is hypothesized from this data that there was some loss of composite action between cycles 4 and 8 due to the close proximity of the heavily concentrated loadings to the gauged site. It is then possible that friction between the slab and the girder resulted in the increasing height of the neutral axis between cycles 8 and 17. Future work will seek to more fully understand the specific phenomena and behavior here.

The third application of the field testing data is to validate a finite element model of the structure. Comparisons between the field data and the corresponding FEA results show that (see example in Fig. 2), at low load levels, the finite element analysis is accurate in predicting the bottom flange strains. However, between load cycles 4 and 7(for the example shown), there is very little increase in strain; otherwise the FEA and field data are in good agreement as the slopes of the two lines are similar after load cycle 8. Additional preliminary data analysis suggests that this FEA model is a promising tool for obtaining more detailed information about the bridge and potentially extrapolating the results to determine the bridge's behavior at its ultimate load. Future work will seek to quantify the changes in load distribution or stiffness that were observed during the testing and incorporate these into the FEA to improve its usefulness.

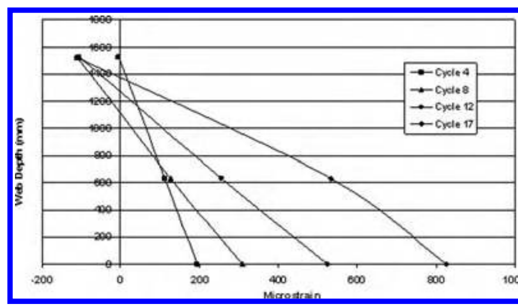

Figure 1. Cross-section strains with increasing load.

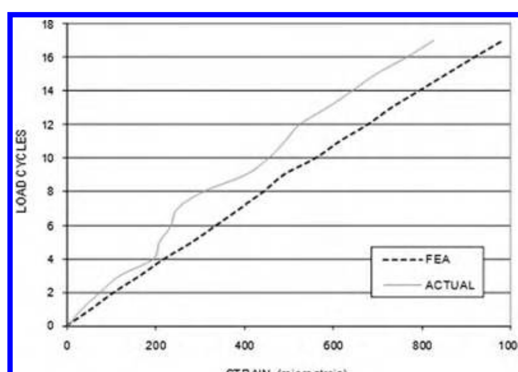

Figure 2. Comparison of actual and FEA strains.

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Bridge inspection and repair technologies on the expressways in Japan

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ABSTRACT

To ensure expressway driving comfort, we NEXCO-Central put extensive effort into inspection, repair and reinforcement work. It is concerned, however, that the damages of the structures by many factors will more increase in the future. Therefore, we need more and more effective maintenance. In this sense, detailed inspection becomes very important, however, we can not enough inspection because of difficulty of approaching to bridge, and lack of inspection engineer and so on. In order to effective maintenance, NEXCO-Central is actively developing inspection systems.

In this paper, I would like to introduce the inspection systems, which is carried out by NEXCO-Central, and the typical example of damage of concrete bridges and steel bridges, and also the technologies of inspection and repair.

1 INSPECTION

The most commonly employed method for concrete structure inspection is visual inspection from the road surface. If any defect is detected by this survey, then, close range visual inspection and hammering test are conducted by engineers. This method, however, is costly, takes a long period of time, and requires traffic regulations. Furthermore, the inspection results could vary depending on the engineer, so there has been a strong demand to develop an effective inspection method. This paper represents several methods being developed at NEXCO-Central by utilizing ICT technologies: one is the infrared concrete defects detection method which can be easily used on sight by engineers, and the others are the digital crack measurement method, and inspection method for PC grouting.

On the other hand, there are several types of deterioration of steel structures, mainly fatigue and corrosion. This paper introduces "Paint View" method for painting corrosion, which is developed by former Japan Highway Public Corporation. This painting diagnosis system applies image-processing technology with the computer, and it is a system evaluating a deterioration phenomenon of the steel bridge painting for the objectivity and fixed-quantity.

2 REPAIR

The repair design is carried out in order to achieve the purpose of repair. It consists of planning and considering the cause of crack, the degradation mechanism and degradation prediction, range and scale of the repair, environmental condition, safety construction period, economic efficiency, degree of importance of the structure, load influence to the environment, reparability, and subsequently, appropriate repair methods and repair materials, repair time, etc are selected.

3 CONCLUSION

The aging structures are increasing and as it turned out more and more expense for inspection and maintenance is needed in near future. Therefore, we must carry out effective inspection and maintenance under the appropriate evaluation. Then, development of inspection and repair technologies is indispensable, and upbringing and improving the skill of the engineer is very important, too.

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Improving transparency of bridge condition for decision-making and analysis in bridge management

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ABSTRACT

Transparency of collected information on bridge infrastructure condition leads to more informed and more effective decisions regarding where and how to appropriate resources so that the maximum benefit is gained for the bridge network. The condition of bridges can be made more transparent to the general public and bridge managers through the use of spatial and spatiotemporal hotspot detection with surveillance geoinformatics. A "hotspot" is an area that is considered unusual, such as an anomaly, aberration, outbreak, elevated cluster, or critical resource area. Based on response properties of geospatial data, spatial and spatiotemporal hotspots can be detected. The goal of hotspot detection is to identify, delineate and assess the significance of hotspots. A scan statistic seeks to identify hotspots of cells that have an elevated response compared with the rest of the region. Hotspot detection can provide surveillance information that can be valuable for mitigation, containment, prevention, and other management strategies.

With the detection of hotspots, it is possible to quickly and graphically identify if there is a region with a concentration of bridges having poor ratings. Preliminary analysis demonstrates how hotspot detection can be used to address bridge management questions related to the scour critical rating and to the overall condition of bridges according to the sufficiency rating in the 2008 National Bridge Inventory for the state of Pennsylvania. The following management questions are explored:

1 Are there hotspots where the bridge scour risk has yet to be determined or where bridge scour risk is high?
2 How well do the hotspots for poor overall bridge condition (based on the sufficiency rating) relate to the hotspots for high scour risk?

The hotspot detection tool identified a region in southwestern Pennsylvania where there is a zone of counties with an elevated rate of bridges that have not been evaluated for bridge scour risk. The detection tools also identified a region in southeastern Pennsylvania where there is a zone of counties with an elevated rate of bridges that are considered to be scour critical. Comparing the scour critical hotspots to the hotspots for poor overall bridge condition (based on the sufficiency rating) shows some agreement between the hotspot for scour critical bridges and the hotspot for bridges with a sufficiency rating below 50 percent.

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Countermeasures for rain- and wind-induced vibrations on the Meiko Nishi Bridge

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ABSTRACT

The Meiko Nishi Bridge was also the first bridge in Japan on which rain- and wind-induced vibration observations were conducted.

A high-damping rubber damper was developed as a method of dealing with rain- and wind-induced vibrations. This high-damping rubber damper improves attenuation performance for cable vibrations by connecting the cable and the girder through rubber that possesses high energy-absorbing performance.

However, the high-damping rubber damper cable had failed. The cause was the fatigue that had accumulated in the high-damping rubber damper. The cause of the failure was the offset deformation. Offset deformation is the relative displacement that occurs between the girder and cable as a result of sunlight and changes in temperature. Accordingly, it was decided to develop a high-damping rubber damper that was not affected by offset deformation.

Existing high-damping rubber dampers are rectangular in shape. As it was recognized that the shape of the damper affects damping and fatigue endurance performance, the shape was changed to a round shape to develop a high-damping rubber damper that would not be affected by offset deformation.

In order to check the performance of the dampers, the cable displacement was measured continuously from April through November 2008. Measurements were conducted for the round high-damping rubber dampers and viscous shear dampers.

Based on the results of past inspections and long-term measurement, the following were determined.

i) Viscous shear dampers and round high-damping rubber dampers exhibit outstanding damping performance with respect to rain- and wind-induced vibrations.
ii) Viscous shear dampers in particular exhibit very exceptional damping performance with respect to rain- and wind-induced vibrations.
iii) Comparatively large displacement occurred on the round high-damping rubber dampers. However, based on the results of the damper performance test, these dampers were judged to possess satisfactory safety with respect to fatigue.

A cable excitation test was conducted on the actual bridge to check the damping performance of the cables on which round high-damping rubber dampers and viscous shear dampers had been installed. Based on the results of measurement, the following were determined.

i) The logarithmic decrement of the cables on which high-damping rubber dampers and viscous shear dampers were installed was 0.02 or greater, and the Scruton number was 60 or greater. Both the design values and the measured values were such that rain- and wind-induced vibrations would be unlikely to occur.
ii) The logarithmic decrement determined from the measured values was only about 60% of the design value.
iii) On the cables that were measured in this test, the viscous shear dampers exhibited better performance than the high-damping rubber dampers.

Judging from the results of long-term measurement and excitation tests conducted under actual conditions, the measures that have been taken on the Meiko Nishi Bridge to prevent rain- and wind-induced vibrations will remain effective as long as periodic inspections are not neglected.

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Proposal for modification of load and resistance factors in the AASHTO LRFD bridge design code

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ABSTRACT

In the new generation of design codes load and resistance factors are determined in the reliability based calibration procedure. A general statement for assuring acceptable safety is that the factored resistance of the components shall exceed the factored load effects. Mathematically it is expressed by basic design equation:

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n \tag{1}$$

where Q_i is the load effect, R_n is the nominal resistance, η_i is load modification factor, γ_i and ϕ are respectively load and resistance factors.

Theoretically the load and resistance factors can take any value. The structural performance can be measured in terms of the reliability index, β. Therefore, the selection criterion for design requirements is closeness to the target β, or acceptable risk of failure. The load factors are expected to be larger than 1, and the resistance factors are expected to be smaller than 1. AASHTO LRFD (2007) load and resistance factors were determined in the calibration process (Nowak 1999). The statistical parameters of load and resistance were determined using the data available in 1980's (Ellingwood et al. 1980 and Nowak 1999). Live load was based on the truck survey in Ontario performed in 1975. Mechanical properties of materials such as concrete and steel were obtained from literature.

Over the last two decades there were significant improvements in numerical and analytical methods resulting in enhanced accuracy in prediction of the actual resistance. In addition, it was observed that the quality of materials improved over the years (Nowak and Szerszen 2000). Therefore, resistance factors can be increased. However, in cases when the resistance factors are already set to 1, such an increase could result in ϕ factors that are larger than 1 which can be philosophically unacceptable.

This study presents a proposal for modification of the load and resistance factors in the AASHTO LRFD (2007). The current Strength I load combination is:

$$U = 1.25DC + 1.5DW + 1.75(LL + IM) \tag{2}$$

The resistance factors that are currently in the AASHTO Code are:

- 1.0 for prestressed concrete girders in flexure,
- 0.9 for reinforced concrete girders in flexure,
- 1.0 for composite and non-composite steel girders in flexure,

The proposed Strength I load combination is

$$U = 1.2DC + 1.4DW + 1.6(LL + IM) + \ldots \tag{3}$$

and the corresponding reduced resistance factors are:

- 0.9 for prestressed concrete girders in flexure,
- 0.8 for reinforced concrete girders in flexure,
- 0.9 for composite and non-composite steel girders in flexure,

The reliability analyses were performed and it was shown that new load and resistance factors will result in slightly higher reliability indices. The proposed changes will allow for future rational modifications of resistance factors reflecting the improved quality of materials and fabrication.

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Recommendations for dynamic allowance in bridge assessment

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ABSTRACT

Correct evaluation of the behavior of highway bridges under heavy traffic loading is extremely important both for the enhancement of design techniques, and also for the assessment of existing infrastructure. It is widely accepted that shortfalls exist in the determination of the traffic load which the bridge may be required to support during its expected lifetime due to inadequate consideration of amongst other factors, the dynamic interaction between the bridge structure and the heavy vehicles crossing it. The latter is addressed in deliverable D10 within the 6th EU framework ARCHES project (2006–2009), which is summarized in this paper. As part of these investigations, lifetime static load effect values are combined with realistic dynamic amplification factors to obtain an overall total lifetime load effect. This process has two distinct parts: (1) The calculation of bridge static load effect due to site-specific traffic flow along with the resultant assessment of bridge lifetime static load effect, and the selection of those loading events that are deemed critical (statically); and (2) The assessment of the levels of dynamic interaction occurring between a bridge and its associated vehicular traffic.

The procedure to obtain a site-specific dynamic amplification factor using numerical simulations and available experimental data is described. It is shown that certain bridges are not susceptible to high levels of vehicle-bridge interaction when loaded by a ‘critically’ heavy vehicle or a ‘critical’ combination of vehicles. Discussions on a range of numerical studies into the site-specific level of total load effect (dynamic + static) and corresponding allowance for dynamics of typical medium span highway bridges are provided.

A procedure to experimentally obtain a site-specific dynamic allowance is justified numerically and then tested experimentally. Initially the approach is applied to a bridge dynamic load scenario defined by weigh-motion based probability distributions which are used to obtain the characteristic value for static load effect. Similarly, but independently, the distributions are applied to a dynamic model to obtain the characteristic total load effect. Comparison between the total and the static results yields the site-specific allowance for dynamic interaction. A method of field measurement is proposed and tested based on these simulations.

Some specific issues concerning the dynamic allowance associated to: (a) pre-existing bridge vibrations; (b) maximum total effects developing in sections different from midspan, (c) the existence of a bump prior to the bridge, or (d) critical loading cases such as cranes, are also discussed in the paper. Finally, ARCHES gives general recommendations on dynamic allowance purely based on the bridge length and road class, which represent a significant reduction in relation to Eurocode values for one-lane bridges and ISO road class ‘A’. These values can be further reduced if a better knowledge of the bridge response was acquired through numerical simulations and field tests. In fact, the analysis of bridge measurements collected during the ARCHES project led to dynamic allowances close to 1.0 for the heaviest loading scenarios in different sites. Site-specific parameters such as a support condition closer to fixed-fixed than simply supported conditions, a critical traffic scenario inducing less dynamics, or a particular profile with an ideal distribution of irregularities within a given road class could clearly lead to smaller dynamic allowances.

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Influence of steel reinforcement corrosion on the stiffness of simply supported concrete beams

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ABSTRACT

The in-service performance of reinforced concrete beams can be severely affected through corrosion of the steel reinforcement when it becomes subjected to harsh corrosive environments containing chlorides and carbon dioxide. In such instances, corrosion is likely to occur in the steel reinforcement, with the expansive nature of the corrosion products likely to induce cracking and spalling of the concrete. A loss of structural integrity (stiffness) will occur and this can severely influence the serviceability of the member.

The purpose of this paper is to investigate the relationship between degree of corrosion and loss of stiffness in corrosion damaged under-reinforced concrete beams. Beams (100mm x 150mm cross section) were subjected to accelerated corrosion in the laboratory and subsequently tested in flexure to failure. The paper reports on the results of these tests and relates the degree of corrosion in the main steel to the percentage loss in stiffness in the concrete beams.

A total of twenty reinforced concrete beams were tested in flexure. The results show that stiffness reduces with increasing main steel corrosion. Referring to Figure A, a linear correlation is evident with $R^2 = 0.6$.

The decrease in stiffness is equal to 2.83 times the degree of corrosion of the main steel. This means that the stiffness of the beam would have to decrease 40% before it exceeds serviceability limits. Therefore the main steel corrosion would have to exceed 14% before deflection criteria becomes an issue.

The main conclusions from the results reported in this paper are as follows and apply within the limit of the parameters covered by the test data in the paper.

– reinforced concrete beams show a loss in stiffness with increasing corrosion of the main and shear steel reinforcement;
– an equation linking the decrease in stiffness to main steel corrosion was determined as follows:
– % decrease in stiffness $= 2.83 \times$ degree of main steel corrosion
– main steel corrosion would have to exceed 14% before deflection criteria are exceeded.

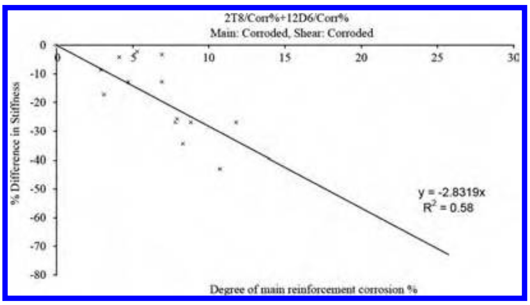

Figure A. Relationship between % loss in stiffness and main reinforcement corrosion.

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Influence of shear reinforcement corrosion on the performance of under-reinforced concrete beams

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ABSTRACT

Reinforced concrete beams are normally designed as under-reinforced to provide ductile behaviour at failure i.e. the tensile moment of resistance, $M_{t(0)}$, is less than the moment of resistance of the compressive zone, M_c. Since it is well established that the steel in reinforced concrete beams is prone to corrosion, the residual flexural strength is normally the main concern of asset managers. However, concrete cover to the shear reinforcement is less than that to the main steel and therefore, may suffer higher levels of corrosion due to chloride penetration and carbonation.

The paper investigates the influence of shear reinforcement corrosion on the performance of reinforced concrete beams. Two groups of eight beams (100mm × 150mm cross section), with two varying degrees of under-reinforcement ($M_{t(0)}/M_c$ ratios) at zero corrosion (control), were tested in flexure. The results show that despite exhibiting varying levels of shear reinforcement corrosion (the main steel remained uncorroded throughout), flexure was still the dominant mode of failure. However, all beams did exhibit a decrease in flexural strength with increasing shear reinforcement corrosion levels indicating that the flexural integrity of the beam was influenced by the shear reinforcement corrosion. This was more pronounced for the beams with a higher $M_{t(0)}/M_c$ ratio (lower degree of under-reinforcement) and this should be taken into account at the design stage.

The relationships in Figure A generally show a linear decrease in $M_{t(ShearCorr)}/M_c$ with increasing percentages of shear reinforcement corrosion. The best fit linear equation for each series of beams is tabulated along with the coefficient of correlation (R^2). A very satisfactory coefficient of correlation exists for both

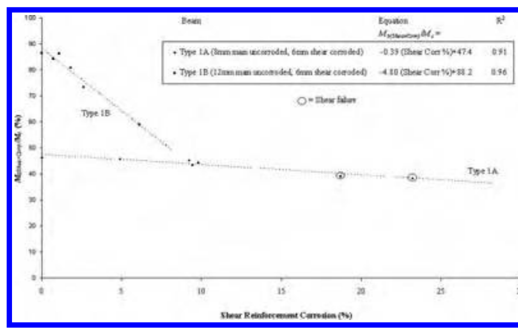

Figure 1. Relationship between $M_{t(ShearCorr)}/M_c$ and the percentage of shear reinforcement corrosion.

tests (>0.91). Shear failure was evident only at higher degrees of shear reinforcement corrosion for Type 1A (>18.7%, shown highlighted in Figure A).

The main conclusions from the results reported in this paper are as follows:

– The predominant failure mode for under-reinforced concrete beams exhibiting low degrees of shear reinforcement corrosion is flexural (<18.7% in this investigation)
– Shear failure occurred only at higher degrees of shear reinforcement corrosion (>18.7% in this investigation)
– Beams with lower $M_{t(0)}/M_c$ ratios suffered a lower rate of flexural strength loss when corrosion was present in the shear reinforcement. Therefore, the recommendation is to design beams with lower $M_{t(0)}/M_c$ for enhanced residual flexural strength but the shear capacity should be increased to guard against the risk of sudden shear failure at higher levels of shear reinforcement corrosion

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Fatigue evaluation of steel finger type expansion joints for highway bridges

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ABSTRACT

A number of fatigue cracks have been found at steel finger type expansion joints of Tomei-Expressway in Japan, recently. Most severe damage was that the face plate of the joint had failed and dismounted from the actual expansion joint as shown in Fig. 1. Fatigue durability assessment is not well evaluated for this kind of joints, although they are subjected to wheel loads directly. Steel finger type expansion joints are consisted of several steel parts—face plate, web plates, rib plates and anchor plates, etc. It is important to evaluate the fatigue strength and fracture mechanism closely for the suitable maintenance. In this study, the observation

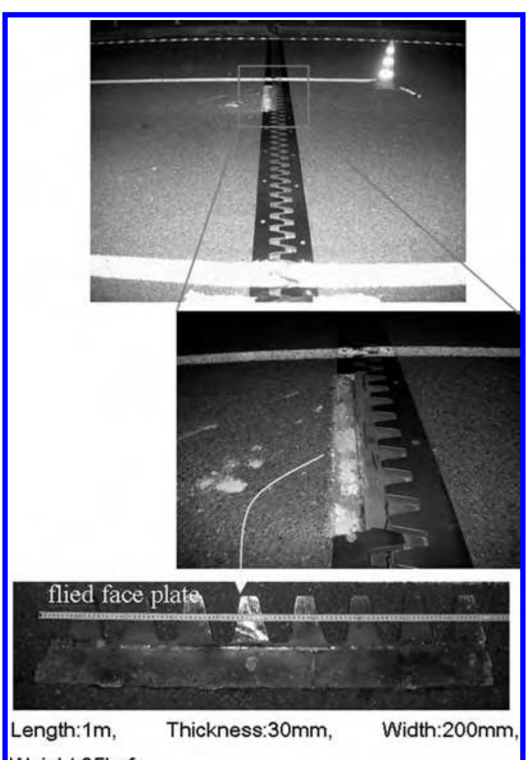

Figure 1. Damage case of finger type joint.

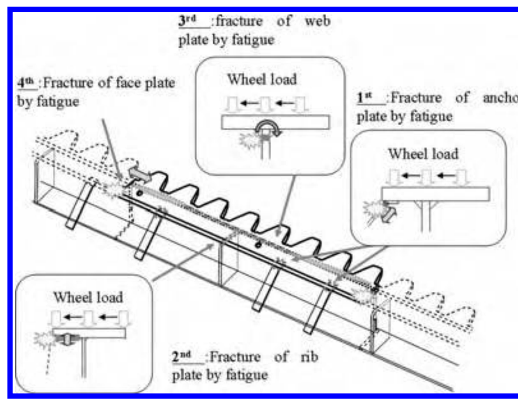

Figure 2. Establish of the fracture mechanism.

of the joints dismounted from existing bridges has been conducted first. The results have confirmed that the cause of damage was the fatigue induced by repetitive wheel loads. Then, the FEM analyses and the fatigue tests with actual size specimens have been performed. The failure process, the fatigue life and the fatigue behavior of this type of the expansion joints have been established finally. The outline of the obtained results of this study for steel finger type expansion joints is presented in this paper.

Fatigue behavior and characteristics of the steel finger type expansion joints in highway have been evaluated by observations of the fractured expansion joints and FEM analyses in this study.

Fatigue life to the fracture of face plate has been estimated by fatigue test for Tomei-Expressway.

The results of this study are summarized as follows.

(1) The fracture of the joints is due to fatigue of applied repetitive wheel load.
(2) The fracture procedure is verified following order anchor plate, rib plate, web plate and face plate as shown in Fig. 2.
(3) The mechanism of face plate fracture is the both-ends-fixed beam bending after the web plate fractured.
(4) The fatigue life of the face plate is predicted approximately 3.2 years.
(5) It is necessary to inspect the joints for fatigue damage at least every 3 years.

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Design of elastic foundation layers for buildings – new design and calculation methods – design examples

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Due to increasing requirements on developed sites with good infrastructure, the buildings move more often near to existing traffic infrastructure. Additionally capital spending in revitalization or renewal of existing traffic nodes (i.e. urban railway stations) leads to new developed sites near existing traffic infrastructure. On the other hand the requirements concerning immission levels (vibrations and re-radiated noise) are increasing rapidly. This leads to the fact that, in order to fulfil these increased requirements, elastic foundation layers for buildings are the only way to achieve these requirements.

Therefore new innovative solutions on designing are required to fulfil on the one hand the technical demands and on the other hand the economical requirements for such projects. Traditionally the design of elastic foundation layers is based on the theory of the SDOF (Single degree of freedom) system. For this the building itself is assumed as a rigid complex. This is generally more or less valid for classical stiff buildings buildings of conrete but leads to an overestimate of the loads applied to the elastic foundation layer for modern buildings with columns bearing the load instead of walls. As a consequence the use of the classical SDOF System for such design calculations leads to wrong estimations for the loads (pressure) on the elastic layer. Thus the demand for an economical design of the elastic layer can not be achieved with the classical SDOF System calculation.

An other important point of interest represents the material used for elastic foundation layers. Generally all of the existing materials come with non-linear material properties. The bedding modulus strongly depends on the load (pressure) on the material. In case of the classical approach a single bedding modulus is assumed for the elastic layer. For buildings where the loads on the foundation varies considerable the classical approach leads to a wrong estimation of the bedding modulus and therefore to a wrong estimation of the system natural frequency. The system natural frequency is however one of the key values for the immissions expected inside the buidling

Thus to meet the immission requirements together with economical requirements new approaches composed of improved FE-calculation considering the non-linear material properties are necessary. Additionally special solutions like expansion joints piping are crossing the elastic foundation layers must be considered.

The present work shows this new calculation and design approach theoretically and on an example. The used example comes from a building site on a new developed area, near a railway tunnel in the city of vienna.

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An attempt of rationalization for maintenance of railway structures using management supporting system

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ABSTRACT

To utilize the existing civil-engineering infrastructures, steady maintenance activities of the structures are quite important. The railway companies in Japan are doing "Regular Inspections" for their own structures by means of the visual survey method periodically. Because these railway companies in the urban areas have a number of structures in spite of their limited manpower, it is very hard for them to keep their structures safe for a long time. Actually, many inspection data were not well organized, stored and utilized depending on the situation. Additionally, highlights of technical viewpoints or rules of judgments by plural inspectors were not well unified.

In order to bridge the gap between the ideal status and the actual one, the information technology has been applied to build up the Structural Management Supporting System (SMS) by the collaboration between 14 railway organizations in the urban areas and a technical research institute. Figure 1 shows the hardware composition of the SMS, and Figure 2 shows the function of drawing the reference chart of the graphical image of the deterioration.

This system has two major technical features; one is that the system has a quite flexible database which is suitable for each individual management framework of 14 railway organizations. The other is that the system has a function to input the grounds of the decisions and output the guide indices of the soundness ranks based on the Maintenance Standards for railway structures in Japan. These features can serve to record and archive the inspection results, and to make maintenance of structures efficient and rational. On the other hand, the negative effect of elongation of the working hours at the inspection sites was produced by using the on-site inspection terminal of this system.

This paper describes outlines and backgrounds of this system in order to give an outlook of the maintenance methodology for the railway structures especially focused on the concrete bridges and viaducts.

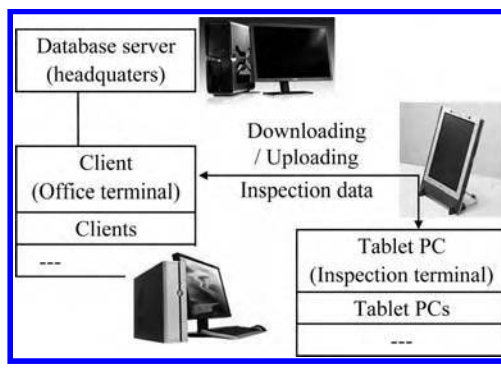

Figure 1. Hardware composition of SMS.

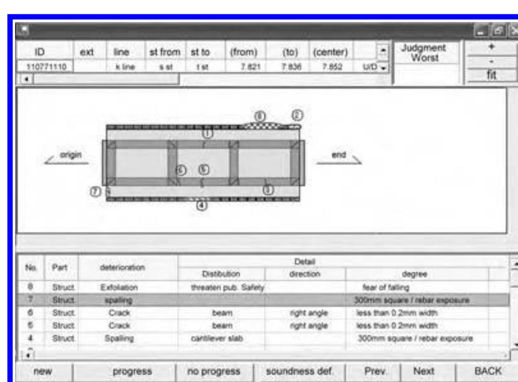

Figure 2. An example of reference chart of deterioration (Translated in English).

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Sensitivity of bridge reliability to parameter variation in systems susceptible to spatially distributed soil liquefaction

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ABSTRACT

The complex characterization and response of coupled soil-bridge-foundation systems in the face of seismically induced liquefaction poses unique challenges when assessing the reliability of critical bridge components. Uncertainty in modeling parameters for bridges susceptible to spatially distributed soil liquefaction range from structural modeling parameters such as material and geometric properties, to soil foundation parameters, such as soil stratigraphy or strength parameters. However, there is a lack of understanding of the relative importance of different variable parameters on the reliability of bridges underlain by liquefiable soils, and a range of levels of fidelity of uncertainty treatment adopted in current bridge fragility modeling practice.

The behavior of a representative multi-span continuous steel bridge (MSCS) typical of the central-eastern U.S. is probabilistically assessed when underlain by liquefiable soils. MSCS bridges are among the most vulnerable bridge classes owing to their bearing and abutment inability to accommodate excessive demands, further exacerbated by soil liquefaction. This study tests the sensitivity of conditional reliability (seismic fragility) estimates for critical bridge components to variation in liquefiable soil, foundation, and bridge modeling parameters.

A two-level fractional factorial design is adopted for the sensitivity study, which requires 32 runs with various combinations of high and low levels of the parameters. In total 13 factors, or modeling parameters, are considered in the sensitivity study, yielding 32 samples of the bridge-soil-foundation (BSF) model with various permutations of the potentially uncertain parameters. Each sample is used in a full fragility analysis rooted in nonlinear dynamic analysis to evaluate the probability of limit state exceedance. The critical components considered in the sensitivity study include the fixed bearings, expansion bearings, columns, and abutments in active action (tension) and passive action (compression) at four damage states. The resulting lognormal fragility curves vary considerably with each combination of high and low modeling parameters as shown in Figure 1. An analysis of variance is conducted to assess the statistical significance of modeling parameter variation on the resulting fragility estimates (median value). Table 1 summarizes the results of the analysis, which found that the significance of parameters remained consistent across the four damage states. The top seven modeling parameters tend to have a statistically significant impact on the sensitivity of the most vulnerable bridge components, as well as other components in the bridge. Modeling parameters that do not tend to have a significant impact on any of the highly vulnerable bridge components considered in this study include fixed bearing initial stiffness, yield strength of steel, soil contraction parameter, and thickness of liquefiable soil layer. However, the fragility of the piles and the bents and abutments are not tested in this analysis and will be a focus of future studies.

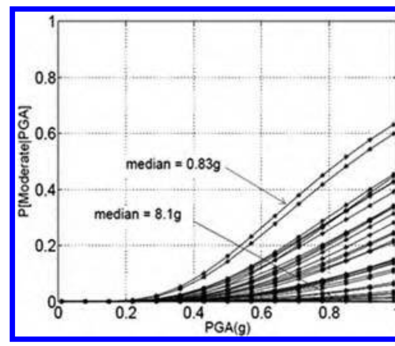

Figure 1. Column fragility curves at the complete damage state from the 32 runs of the sensitivity study.

Table 1. Summary of sensitivity study for median value fragility ranking the modeling parameter signifi-cance across all components and damage states.

Rank of Most Significant Parameters
Undrained shear strength
Damping ratio
Shear modulus
Deck-abutment gap
Ultimate capacity of soil
Expansion bearing COF
Fixed bearing COF*
Friction angle

*Equally as important as preceding parameter.

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An enhanced XML Schema matching technique for checking of the missing items in construction document delivery

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ABSTRACT

An enhanced method for XML Schema matching is provided for developing a tool checking information items during a process of electronic document delivery. This research has adopted an XML Schema matching technique developed by Lee *et al*. (2006) because most construction documents are in an unstructured format. Basic process of the schema matching process follows a work done by Yi *et al*. (2005), and the matching process consists of similarity measurement and relaxation labeling process. Main drawback of previous method is that the computing time for relaxation labeling dramatically increases as the number of document element increases because 4th order matrix should be calculated during the relaxation labeling process. In order to overcome this drawback, this research proposed a simplified compatibility model for the relaxation labeling process.

Distribution of accuracies according to variation of weight values used in similarity measurement was analyzed to evaluate accuracy when the new simplified compatibility model was applied. As shown in Figure 1, the maximum mean value of accuracy for the proposed method achieved about 93.61% where the ratio of weight values on element name, sibling, child and parent was $\omega_{NE} : \omega_S : \omega_C : \omega_P = 1:1:1:1/2$. When the previous method was applied, the maximum mean value of accuracy was 93.62%.

Whilst the accuracy of new method achieved as much as the accuracy of previous method, computing time was dramatically reduced. The simplified compatibility model could reduce the computing time of relaxation labeling process to 460 times where the number of total elements was 218 (see Figure 2).

Validation of the proposed method also performed with twenty of sample schemas that were extracted from structural calculation document of steel girder bridge. The total numbers of document elements were 29,525. Final accuracy of the proposed method was 91.0%, and it took 118 seconds per document in the relaxation labeling process.

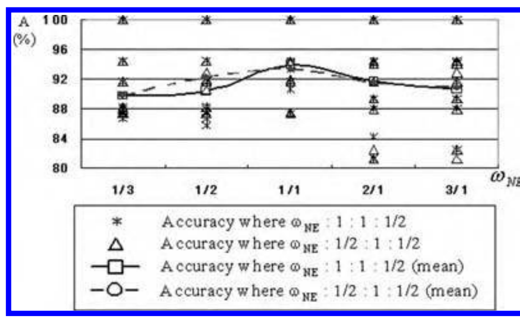

Figure 1. Accuracies according to change of ω_{NE} ratio in simplified module.

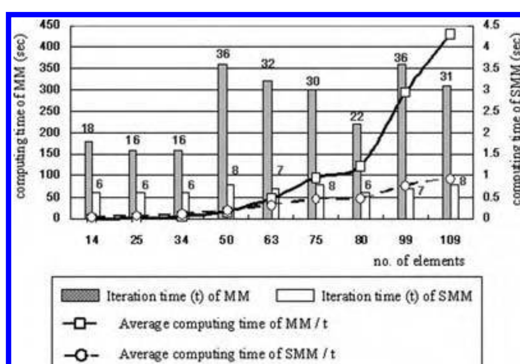

Figure 2. Comparison of computing time due to number of element of model.

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Structural analysis and fatigue life assessment of the Paderno Arch Bridge

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ABSTRACT

According to a study conducted by the ASCE Committee on Fatigue and Fracture Reliability (1982), 80–90% of failures in steel structures are related to fatigue and fracture. A follow up paper of this study is reported in Byers et al. (1997). At the same time, iron historical bridges represent a relevant category of the international cultural heritage, being the evidence of the modern industrial technology, particularly those intended to accommodate activities of an industrial or transport infrastructure. Many of these structures require particular rehabilitation due to design defects, basic elements deterioration, variation of use or change of the intensity of the imposed loads. With regard to Italy, the historical heritage is rich of significant metal structures, which played an essential role in the growth of industrial civilization: the most part of this heritage is represented by bridges, and the 60 per cent of Italian railway steel bridges has about one hundred years, as they were built between 1900th–1920th.

The Paderno bridge, an arch metal historical structure is part of the Monza-Bergamo line and is in service from 1889. A stepwise and practical approach for evaluating the structural integrity of historical and deteriorated steel bridges, incorporating analytical, mechanical and structural characterizations, is presented. Critical regions of hot-spot members were identified using structural finite element analysis, and remaining fatigue life estimation has been performed. In this paper the Paderno steel bridge, a typical arched railway of the mid-nineteenth century, is studied according to a step-level assessment procedure proposed in Pipinato (2008). First the bridge is geometrically described and a literature material investigation is carried out. Then, a linear FEM model is used to find out critical hot spot stress. Finally hot spot stress data are used in order to perform the reliability fatigue assessment.

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New high endurance sliding material for bridge bearings

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ABSTRACT

PTFE–steel sliding bearings are among the most popular bearings currently used in bridge construction because of their ability to support heavy loads and accommodate large movements and rotations.

Thanks to its low friction and good resistance to environmental influences, since its introduction in the ‘60s PTFE has represented the state of the art low-friction material in sliding bearings, despite of its moderate load carrying capacity and low wear endurance, which cause the PTFE element to be the dimensioning and most stressed part of the bearing.

However in the recent years developments in bridge design and increasing traffic conditions have posed new and more severe requirements to sliding bearings. In very long span bridges and in structures subjected to very frequent loads the expected movements at the supports during the service life of the structure are expected to exceed by far the wear life of PTFE, and PTFE bearings should be replaced several times during the lifetime of the structure, resulting in excessive costs and an extremely high maintenance burden. Further, the coefficient of friction of PTFE increases at low temperatures, and the European standard EN 1337 prescribes a minimum operating temperature of −35°C for PTFE sliding bearings, so excluding the use of these bearings from regions with continuously low temperature characteristics.

To overcome these limitations, a special sliding material has been developed at Politecnico di Milano and proposed as an alternative to PTFE in sliding bearings. The trade name of the special sliding material is Xlide®. An extensive experimental campaign has been carried out to assess its functional properties, and the main results are summarized as follows:

1 the compressive strength of Xlide® at ambient temperature is 190 MPa, which is two times larger than PTFE’s;
2 sliding elements made of Xlide® plates lubricated with silicon grease can sustain, with negligible wear, accumulated movements up to a total distance of 50,000 m which is five times larger the distance covered by PTFE elements under similar conditions (Fig. 1);
3 Xlide® offers minimal resistance to sliding even at very low temperatures, out of the range of application of PTFE;
4 the dynamic coefficient of friction of Xlide® holds constant over the typical pressure and velocity ranges of bridge bearings;
5 Xlide® has a good resistance to chemical and environmental influences.

In conclusion, Xlide® is suitable to replace PTFE in sliding bearings subjected to high loads, long distances of movements during the service life of the structure, and very low environmental temperatures. Currently use of Xlide® in sliding spherical bearings is covered by an European Technical Approval.

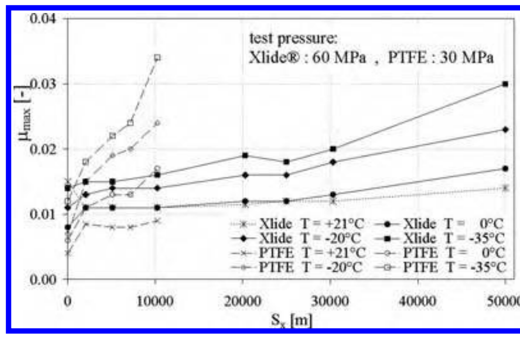

Figure 1. Friction profiles of dimpled and lubricated sliding elements using either Xlide® or PTFE at different temperatures and accumulated sliding distances.

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What is bridge durability – official regulations and reality

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ABSTRACT

Bridge durability has became during the last decades one of the most important technical, economical and social problems in many countries including the most developed ones. It results mainly from the fact that the durability of a large number of the existing bridge structures has shown to be not enough and they show to be structurally deficient and, therefore, require to be rehabilitated. The durability is multi-component problem concerning both the material and structural solutions of the bridge structures as well as their maintenance. Moreover, it concerns the adequate methods of design and erection of the new bridges as well as the rehabilitation of the existing ones. In many cases the existing bridges are also functionally obsolete and demand to be modernized. Therefore, bridge durability can referred also to the modernized structures.

According to the author's concept illustrated in Figure 1, the bridge durability can be defined as the period of time when the bridge (or its structural member or detail) achieves certain acceptable level of decreasing its required technical and/or functional characteristics, such as e.g., load-carrying capacity, stiffness, crack widths, too limited vertical clearance under the structure or too small width of the bridge deck, etc.

Officially required bridge durability (i.e., their service life) in the relevant regulations does not result from the bridge design methods applied so far. These traditional methods do not allow to ensure the bridge durability demanded by the bridge administration because they are based on the quite different philosophy of design than durability design. The durability design is the newest trend and allows to harmonize the methodology of design with the required durability of the bridge structures with its technical aspect. The fundamentals of the above design method are presented in the paper and exemplified by the case of carbonation of concrete and corrosion of the steel reinforcement as the factors influencing the bridge durability.

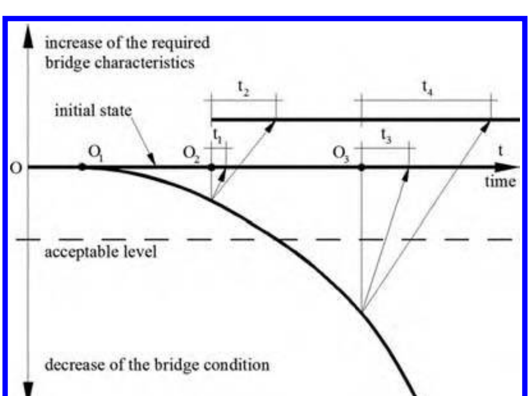

Figure 1. Bridge characteristics in a function of time.

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Effective management of concrete assets – concrete durability: Achievement and enhancement

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ABSTRACT

Effective management of concrete assets is noabout delivering best value return on investment. The safety of highway users places an additional burden on bridge asset managers, who must ensure their assets remain safe, predictable and resilient to changing materials and environments.

To achieve safe and predictable bridge assets, owners and operators need a detailed asset management strategy, encompassing a database inventory, inspection reports operation and maintenance plans, a valuation for each asset and an understanding of the level of service expected. Many industries are already highly familiar with these concepts, including Aviation, Water, Rail and the Military. In the UK, even the humble car parks now need an asset management plan (the Life Care Plan), following the high profile car park collapse in Wolverhampton.

The Bridges and Highways sector has lagged behind other industries in the implementation of a full asset management strategy. In the UK, bridges are inspected regularly and inspection information is stored on a central database, along with notional asset value information. This is moving towards the concept of a full depreciated asset valuation, based on actual condition, but it is a long way off.

The situation for the local roads and bridges is far less consistent or as well advanced. Local Governments (run by County, District or City Councils) have been forced to comply with UK Government requirements to carry out asset management and valuation to all of their assets. Many Local Government groups are well down the road to preparing Highways Asset Management Plans (HAMPs) – but there remains a wide gulf between knowing what bridge assets you have and the depreciated value of that asset: a gulf that can only be closed by expensive testing, assessment and optioneering.

Those tasked with managing bridges, as well as engineers and concrete technologists, need a working understanding of asset management plans, objectives and valuation principles if they are to engage effectively in this field. Equally, asset management practitioners need assistance from concrete specialists with to predict future deterioration rates for concrete assets, identifying time-to-spalling, reductions in structural capacity with time if they are left unrepaired, and ultimately, the point in time when the asset will be cheaper to demolish and replace, rather than retain and repair, in whole life value terms.

This paper discusses some of the issues identified when considering bridge asset management, in a climate where owners are trying to get longer lives and reduced maintenance for concrete assets. The paper focuses on bridge asset valuation in particular and its relationship with whole life valued costing of refurbishment options for bridge assets. Examples are given for cost effective management of bridges.

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Condition inspection, analysis and maintenance of three prominent railroad bridges

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ABSTRACT

During 2005 and 2006, Hardesty & Hanover performed bridge condition inspection and load rating analysis of three prominent railroad bridges. The first bridge is Amtrak's Little Hell Gate Bridge in New York City. The Little Hell Gate Bridge, completed in 1917, is a three-track, 1,156-foot long, unique underdeck inverted bow string steel truss structure with pins and eyebars, soaring high above the urban landscape of New York, designed by the renowned Gustav Lindenthal and Othmar Ammann. The second and third bridges are the Tunkhannock and Martin's Creek Viaducts near Scranton, Pennsylvania, inspected for Canadian Pacific Railway. The viaducts were constructed circa 1915 and are massive, world-record, reinforced concrete structures of 2,375 feet and 1,600 feet length respectively, that carry a single-track railroad as high as 240 feet above rural, forested mountain valleys of northeastern Pennsylvania. Each bridge posed challenges to the engineers conducting the inspection, as there were unique access requirements, specialty testing, operating railroad accommodation, and issues of special interest involved with each. This presentation provides information on field inspection access, specialty testing, and recommended maintenance programs on these types of bridges. This presentation will describe the inspection plans used to thoroughly and efficiently conduct these bridge inspections, the specialty testing used on each, and the safety procedures implemented to mitigate the higher risks faced on these bridges. This presentation will also describe the observed maintenance issues and recommendations to owners/managers to enhance their maintenance plans in order to increase the lifespan of their structures and minimize future rehabilitation and replacement costs.

Figure 1. End Span of Amtrak Little Hell Gate Bridge.

Figure 2. Canadian Pacific Railway Tunkhannock Viaduct.

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Full scale laboratory testing of replacement orthotropic deck for Verrazano Narrows Bridge

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ABSTRACT

After a comprehensive study of the site specific loading, it was recommended to replace the 45 year old concrete filled steel grid deck at the upper level of the Verrazano Narrows Bridge with a steel orthotropic deck that is integral with the floor system and the stiffening truss. The new lightweight deck system with a thin wearing surface and concrete barrier demonstrated better structural efficiency and life cycle performance. The replacement deck designed by the Parsons Transportation Group of New York City consists of closed trapezoidal ribs. To accommodate the limitations on vertical clearance, the deck panel incorporates a shallow sub-floor beam (diaphragm) with a cutout at the junction of the pass through ribs. The ribs are provided with a full depth internal bulkheads at the sub-floor beam locations. One of the primary reasons for selecting an orthotropic deck is that the deck system, if properly designed, can provide more than 100 years service life with minimum maintenance. However, the major concern related to orthotropic deck is the relatively high initial cost owing to intensive fabrication, and higher possibility of in-service fatigue cracking from a large number of welded connections. Particularly severe of these details is the bulkhead/sub-floor beam to rib connections that are subjected to complex in-plane and out-of-plane deformations under localized wheel loads. Other critical detail is the longitudinal rib-to-deck partial penetration weld that is known to develop fatigue cracking at the lack of penetration at the weld root.

Fatigue performance of the replacement orthotropic deck for the Verrazano Narrows Bridge is being evaluated on a full scale prototype at the ATLSS Engineering Research Center, Lehigh University. The prototype deck is about a quarter of the deck between the panel points of the stiffening truss, and includes one floor beam and two stringers. The fatigue testing is being conducted using six actuators loaded in series, simulating the passage of tandem axles. In addition, global displacement of the deck is simulated by three actuators located under the floor beam and the two stringers. The deck response is also determined under a rolling

tandem axle bogie load moving across the deck at a slow rate. The deck is instrumented using more than 300 sensors at critical locations. In addition the deck performance is also monitored using acoustic emission technique. A finite element model of the prototype deck is analyzed to understand the complex behavior of the deck under localized and moving wheel loads, to verify the measured response under static and dynamic loading conditions and to correlate the deck response with the observed cracking mode. The prototype testing and the analytical studies is providing critical information on issues related to fabrication and deck installation, design improvement and optimization, expected response of the orthotropic deck in service, and the effectiveness of the replacement deck design in achieving more than 100 years service life.

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Structural hardening for cable elements of cable supported bridges

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ABSTRACT

The necessity of ensuring transportation structures are protected from threats on all levels is an ongoing concern for those involved in the design, construction, operation and maintenance of critical infrastructure. Cable-supported bridges can be particularly vulnerable to a variety of threats, including large blast and precision demolition explosives, as well as fire and a variety of cutting threats. With the proper analysis of these threats and the execution of a thorough planning process, these structures can be effectively protected. This presentation will examine the details involved with each step of this process, including:

- Objective evaluation of the vulnerable elements against generally defined threats through a thorough risk analysis and detailed structural analysis of the bridge.
- Definition of the specific threat or threats the hardening solution will be required to defeat to allow for objective testing and approval.
- A discussion of how to select the best structural hardening alternative for a particular structure based on specific evaluation criteria such as price, performance, weight, durability, maintainability, and aesthetics. The presentation will examine how existing products and practices may be adapted to bridge protection.
- Establishment of the proper performance tests and acceptance criteria that should be used to evaluate hardening alternatives against specific threats.
- Recommendations for the procurement process, including the development of secure plans and specifications, protection of the design threat(s) and hardening alternative composition.

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Performance assessment of bridges under progressive damage and abnormal actions

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ABSTRACT

This paper presents a framework for evaluation of bridge performance under progressive damage and abnormal actions. Evaluation of bridge response under damage requires the use of proper performance measures. Selected performance indicators are explained and used in a numerical example of a truss bridge undergoing time-variant damage. Lifetime performance of the bridge is evaluated using both deterministic and probabilistic approaches. In the probabilistic approach, uncertainties are related to loading and member strength. The first yielding probability, the system failure probability, and the vulnerability and damage tolerance profiles considering 75 years lifetime of the structure are obtained. The reserve strength factor, residual strength factor, redundancy factor and margin of redundancy profiles are also obtained.

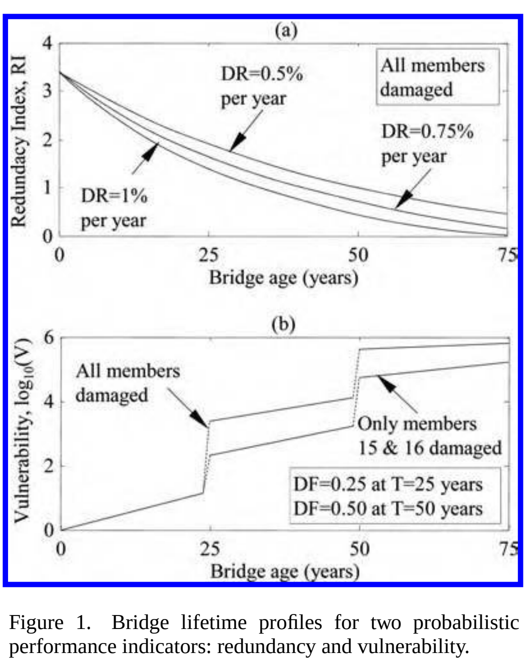

Figure 1. Bridge lifetime profiles for two probabilistic performance indicators: redundancy and vulnerability.

Alternative damage scenarios including slowly arising damage, sudden damage, damage occurring in all members and damage occurring in critical members are considered.

Time variant parameters in the analysis are the resistance of each member and applied load on the structure. Alternative resistance degradation rates and damage factors are considered while load increase rate is kept constant in all analyses.

Probabilistic lifetime profiles of redundancy and vulnerability of a truss bridge under different damage scenarios are shown in Figure 1.

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Detection of fatigue crack of steel deck plate by ultrasonic nondestructive testing

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ABSTRACT

Steel orthotropic deck systems have been applied in many countries. However, in recent years, many cases of fatigue cracking of the deck systems have been reported in Japan. This study focuses on a rib-to-deck fatigue crack that is initiated at the weld root and propagated in the deck plate, as shown in Figure 1. Considering inspection of cracks shown in Figure 1, visual inspection is nearly impossible due to the accessibility. The objective of this study was to investigate the applicability of the ultrasonic testing to detect those fatigue cracks. Fatigue tests were carried out. During the fatigue tests, three ultrasonic nondestructive techniques were applied. Those were creeping wave, horizontal shear (SH) wave, and surface (Rayleigh) wave techniques. Detectability of each technique was investigated.

The specimen was fabricated by welding two plates, considering a deck plate and a rib plate. Cyclic loading of 200,000 cycles was applied to the specimen. After loading, the specimen was taken away from the loading site, and the ultrasonic testing was carried out. That procedure was repeated until failure. At each time of cyclic loading, the amplitude of the loading was changed to make beach marks. Fatigue failure from the weld root into deck plate occurred after more than 900,000 cycles.

Figure 2 shows one of the wave forms obtained by the ultrasonic testing. Relation between echo heights and crack depths was obtained. Actual crack depth could be obtained from the beach mark observation. Figure 3 shows the echo height and crack depth plot. The SH wave technique showed the best detection ability among three techniques. It was possible to detect crack longer than 2 mm completely. The creeping and surface wave techniques also showed a good performance. The ratios of detection for a crack of 2 mm or longer were 80 and 75%, respectively.

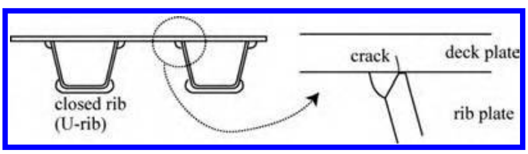

Figure 1. Fatigue crack focused in this study.

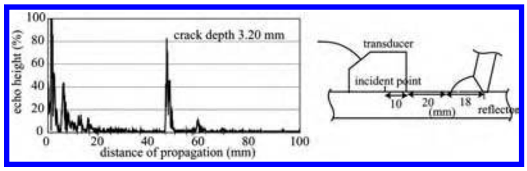

Figure 2. Wave form obtained by SH wave technique, when the fatigue crack was 3.20 mm deep.

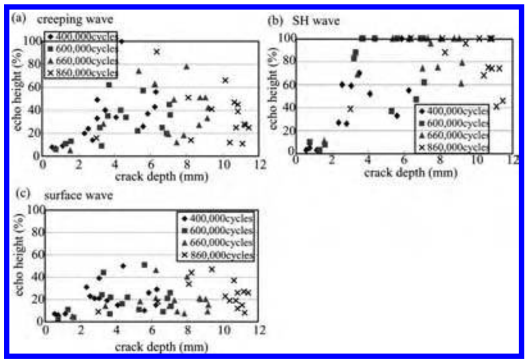

Figure 3. Echo height and crack depth in cases of (a) creeping wave, (b) SH wave, and (c) surface wave methods.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)

Fiber-reinforced polymer composites: An effective solution to our nation's crumbling infrastructure

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ABSTRACT

The poor condition of our nation's bridges has been well documented. According to the National Bridge Inventory, there are over 600,000 bridges in the United States, and 25% of them are determined to be either structurally deficient or functionally obsolete.

The age, environment, and increasing load requirements are factors contributing to the deterioration of our nation's infrastructure.

For years, engineers have been looking for a solution to the above issues, to repair and strengthen our bridges quickly, in a cost effective manner, and with minimal disruption to traffic. One of the most tested products over the past 20 years is Fiber Reinforced Polymer (FRP) Composites. FRP's are one of the tolls that will help us to catch up to the maintenance needs of thousands of our bridges.

Research and testing of FRP composites began in the 1980's to strengthen structural elements to withstand the forces associated with earthquakes, and strengthening of structural elements that are either deteriorated, under-designed or damaged from impact. The long term durability of FRP composites will also be discussed in greater detail.

There are independent design guidelines published by the American Concrete Institute, and the International Code Council. The Transportation Research Board has also published a useful guide for design and inspection. AASHTO is developing a design guideline as well. There will be a discussion on how FRP composites are designed.

Today, there are hundreds of installations of FRP composites that have been used to strengthen bridge columns, beams, pier caps, walls and slabs. Several case studies will be shown to emphasize the effectiveness of strengthening with FRP composites, and the cost savings for the owner. Many installations are over 10 years old. FRP Composites have proven to be very effective in strengthening structural members, and extending the life of our nation's bridges.

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Bridge Maintenance, Safety, Management and Life-Cycle Optimization – Frangopol, Sause & Kusko (eds)
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Seismic assessment on the existing highway bridges in Taiwan

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ABSTRACT

The current engineering technology as well as economy-benefit policy to enhance the seismic capacity of the bridges in Taiwan is introduced in this paper. A plan of seismic retrofitting for more than 2,200 highway bridges all over Taiwan performed through 2006 to 2008 is the focus to present and discuss. Fifteen types of bridge in which were categorized in advance according to their different seismic responses, based on the lessons learnt from the experienced catastrophic Figure 1. Analytical bridge model used in SAP-2000 earthquakes. Some representative bridges in each type were preceded with either pushover analysis or time history analysis to get the seismic capacity. Accordingly, the seismic fragility curves for each group can be established statistically. The feasible retrofitting approaches and the cost required for the grouped bridges with insufficient seismic capacity were evaluated to serve as the basis of seismic retrofitting for the numerous existing bridges. As a result, the possible resulting benefits of social economy can be explored and discussed to support the optimal retrofitting strategy.

A simplified and novel seismic assessment procedure has been introduced. A state-of-the-art pushover analysis method is adopted to calculate the seismic resistance capacity of the bridge. A few modifications have been made in the pushover analysis to obtain more reasonable results. The case study of the Li-Kun bridge shows the efficiency of applying the retrofitting measure to improve the seismic behavior in terms of the return period of the earthquake or anticipated service life. Analytical result shows that the Li-Kun bridge can sustain an earthquake with PGA of 0.488 g and can function effectively for more than 107 years after performing the seismic retrofitting work. Therefore, the goal of extending the structural longevity is achieved. This paper is useful for engineers to define a higher performance objective when setting a realistic target in the performance-based design approach.

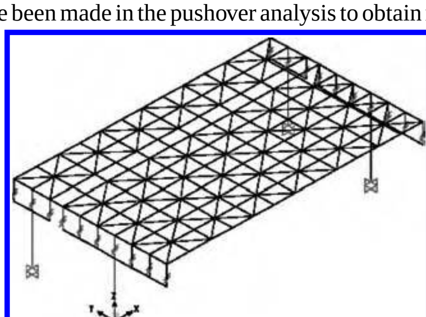

Figure 1. Analytical bridge model used in SAP-2000.

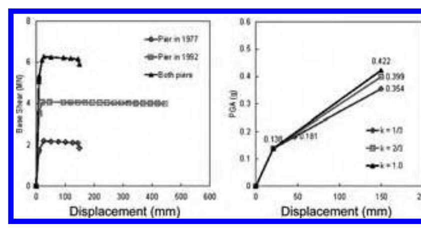

Figure 2. Pushover curve in the longitudinal direction.
Figure 3. PGA assessment result in the longitudinal direction.

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Prediction of restrained shrinkage stresses and premature deck cracking in composite bridges

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ABSTRACT

Reinforced concrete bridge deck slabs made composite with supporting steel or pre-stressed concrete girders are widely used structural systems in highway bridges. Current practice in operation shows that service life of a reinforced concrete deck is much shorter than the design life of the bridge, Krauss & Rogalla (1996). Observed deterioration scenarios indicate that degradation process of bridge decks is very often initiated by hydration heat and shrinkage cracks. Despite of many modifications made in standards of bridge deck design and curing transverse cracks have continued to appear, French et al. (1999), Hadidi & Saadeghvaziri (2005).

Paper deals with the development of a simple and practical method to estimate stresses due to restrained shrinkage. Proposed approach is based on flexure analysis of a composite beam's section assuming slip at interface of the deck and girder as well as slip between concrete deck and reinforcement. Displacement and strain discontinuity condition at the interfaces, and curvature compatibility for the deck and girder of composite section are utilized. Linear elastic properties of constituent materials and linear slip constitutive laws are assumed. Obtained results indicate the strong possibility of cracks initiation in the deck slab due to restrained shrinkage development especially in early age of concrete. The bottom part of the deck section is mostly exposed to cracking. The ratio of deck-to-girder stiffness strongly affects the final value of the restrained shrinkage stresses. The analytical expression for the coefficient of restraint level is given in closed form,

$$\eta_{Db} = \frac{1}{1 + \dfrac{B_D}{B_G} + \dfrac{H_C^2 B_D}{D_D + D_G}} + \frac{z_{Db}}{H_c + \left(1 + \dfrac{B_D}{B_G}\right)\dfrac{D_D + D_G}{H_C B_D}}$$

and can be easily used in design practice to measure cracking tendency of concrete in bridge decks.

The restraint level caused by the steel girders ranges from 25% to 60% of the full restraint. In case of typical pre-stressed AASHTO girders the constraint to the shrinking bridge deck is less than for steel girders, Saadeghvaziri & Hadidi (2005).

A method to reduce early-age deck cracking is proposed. A general idea behind the method is to initially create stress pattern in the slab of opposite sign to that caused by restrained shrinkage. It is proposed to apply externally controlled forces to the girders to create the initial camber. In that manner, tensile stresses will be induced at the top part of the girders. After the first phase of concrete maturing, the camber will be gradually relaxed. A part of tension stress caused by shrinkage will be compensated by the composite action of bridge cross-section. As an example of application of compensating load simply supported beam (pin-roller) of a bridge is analyzed, see Figure 1.

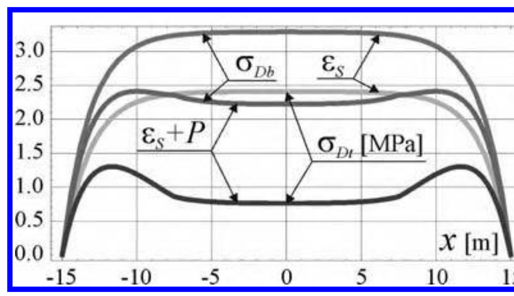

Figure 1. Relaxed stresses at the top and bottom fibers of the cross-section in the deck of composite bridge.

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Evaluation of corrosion deterioration of weathering steel bridge under the environmental corrosiveness

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ABSTRACT

Weathering steel bridge had been exposed for 28 years in Okinawa Japan. Corrosion environment for steel material in Okinawa is a severe accelerated corrosion environment because of high humidity with high temperature, and huge amount of the air-born salt for islands region. As for this bridge, sever corrosion was happened the end of girders. Then it collapsed in July 2009.

In this study, we carried out evaluation of corrosion deterioration for steel girder bridge using this weathering steel bridge under the real severe accelerated corrosion environment.

In this report, the outline of investigation results will be presented and discussed based on the decrease of the main girder's web plate thickness and the environmental corrosiveness.

Photo 1. Panorama bridge (Okinawa in Japan).

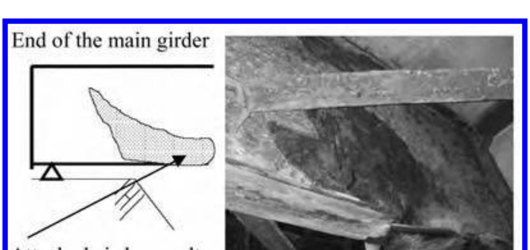

Figure 1. Corrosion situation (the end of the girder).

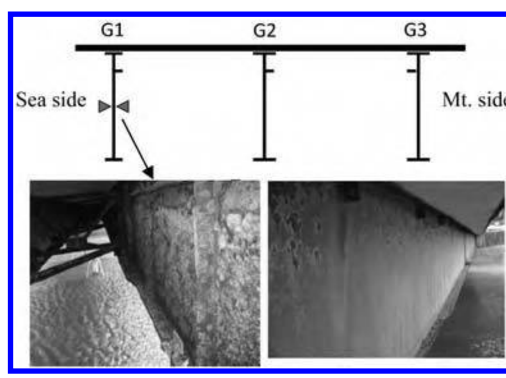

Figure 2. Corrosion situation (inside and outside).

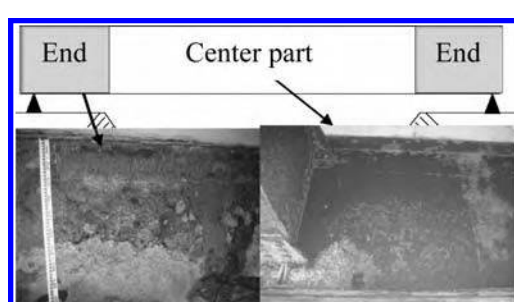

Figure 3. Corrosion of the center part and the end.

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Study on the cause of brittle fracture during earthquakes in steel bridge bent focusing on stress triaxiality

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ABSTRACT

During the Northridge Earthquake in 1994 and the South Hyogo prefecture Earthquake in 1995, unexpected brittle fractures were observed in some steel structures. A corner of steel bridge bent was also cracked seriously.

Based on the multiaxiality, this paper reveals the effect of some weld bead shape on the risk of brittle fracture during earthquakes on real structure. The multiaxiality was evaluated by stress triaxiality which was computed through nonlinear FEM analyses applying an earthquake waveform. The target structure is steel bridge bent which was damaged during the South Hyogo prefecture Earthquake.

Stress triaxiality is often defined as

$$T_1 = \frac{\sigma_h}{\bar{\sigma}}, \tag{1}$$

where σ_h and $\bar{\sigma}$ are respectively the hydrostatic stress and the von Mises stress. On the other hands, triaxiality T_2 is also defined by Schafer et al. (2000). The Definition of T_2 is as follows:

$$T_2 = \frac{\sigma_{max}}{\bar{\sigma}}. \tag{2}$$

where σ_{max} is the maximum principal stress.

T_1 and T_2 are indexes which have a similar tendency for estimating the triaxiality of multiaxial stress state (Tamura et al. 2009).

In this study, FEM analyses considering geometrical and material nonlinearities were conducted using the analysis program Abaqus 6.7. In the analyses, a zooming method was applied to obtain the local stress state of fracture origin during earthquakes.

As one of the result of this study, the dependency of triaxiality on the weld bead radius is shown in Figure 1. The horizontal axis of the figure is the depth from the weld toe in which the fracture of real bent started. From the figure, the difference of triaxiality distributions among four kinds of bead shape (the toe radii are 0.5 mm, 1.0 mm, 2.0 mm, and 5.0 mm) can be confirmed. The degree of triaxiality increase with the depth depends strongly on the toe radius.

The triaxiality increases the risk of brittle fracture, because it increases the local normal stress of there. Figure 2 shows the effects of triaxiality on the maximum principal distributions at the fracture origin. It

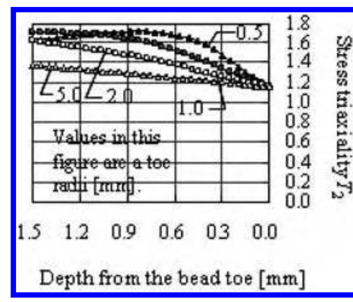

Figure 1. Triaxiality distributins.

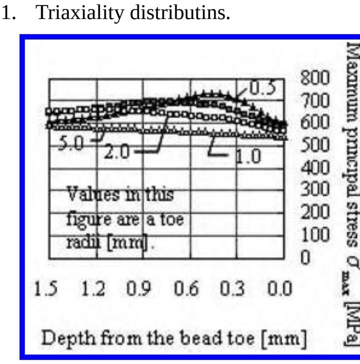

Figure 2. Maximum principal stress distributions.

can be found the remarkable difference. This is because the effect of the triaxiality which is strongly affected by the weld bead shape. It implies one of the main causes of brittle fracture in the real bent was possibly the triaxiality generated by a small radius toe (approximately 1 mm) of unequal leg bead.

Similarly, the effects of unequal leg bead and cracking were also investigated. As a result, it was indicated that there is a possibility that fracture occurs due to these weld bead shape effect.

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A simplified approach to calculate the secondary moments of continuous prestressed concrete bridges

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ABSTRACT

This paper provides practical engineers with an equation to calculate the secondary moments of prestressed concrete continuous beams. Normally the prestressed secondary moments are calculated by deriving it from the net moment and the primary moments. This involves extensive calculations and sometimes complex integration techniques but the proposed equation described in this paper will give the secondary moment directly. The secondary moment calculated using the equation is compared with the secondary moment calculated using various existing methods such as support displacement method, moment distribution method and stiffness method, and also using the structural analysis software STAAD. For the purpose of analysis, two span, three span and four span beams are considered. This equation can be applied to multiple span beams/girders. Finally the equation is used to show the principle of concordant profile where the secondary moments are zero at the intermediate supports.

The equation for determining the secondary moment is derived from the basic principles of moment area and support displacement. For the purpose of analysis a two span continuous prestressed prismatic concrete girder over supports A, B and C is modeled and shown in Figure 1. The physical parameters of the girders are shown below.

L = Span length of AB & BC in ft
P = Prestressing force on the tendons in ksi
e_1 = Eccentricity at mid span in inches
e_2 = Eccentricity at intermediate support in inches

Extensive work has been done to study the behavior of prestressed concrete members in the past and very few of them deal with the calculation of secondary moments. A simplified method to determine the secondary moments has not yet been explored. The following methods are the key methods used in this paper to substantiate the validity of the equation.

1. Moment area method
2. Support Displacement method

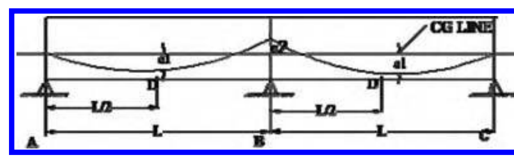

Figure 1. Two span girder showing span length and eccentricities.

3. Moment distribution
4. Stiffness method
5. STAAD

To summarize the secondary moment of prestressed concrete can be determined using the derived expression so that the designers can use this equation.

- The proposed equation can be used to accurately calculate the secondary moment using the eccentricities and span length.
- The span length can be modified to any length but the secondary moment of the system is not affected due to this change and when the eccentricities are modified there will be a significant change in the secondary moment
- With these analyses, it has been noticed that the concordant profile can be achieved when the eccentricity at the center of span is twice of eccentricity at the intermediate span. (i.e. $e_1 = \frac{e_2}{2}$ and the secondary moment will be zero).

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Fatigue testing of stay cables at resonant frequency

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ABSTRACT

Dynamic fatigue tests are used to evaluate the fatigue behavior of materials and structures. The cycling load for dynamic fatigue tests is generally applied by means of servo hydraulic controlled jacks. The testing setup is very simple but on the other hand the testing frequency is low for high test loads (max. 1 Hz). The Institute for Structural Engineering has developed a new method for dynamic fatigue tests on large specimens like stay cables (Kollegger, Köberl, Pardatscher & Vill).

Due to the novel method it is possible to increase the testing frequency up to 40 Hz, also for large specimens.

After tensioning a specimen and an auxiliary cable, anchored at a coupling unit by means of a hydraulic jack an unbalanced vibration generator, attached at the coupling unit applies the vibration loading. In case the testing frequency of the specimen is equal to the first eigenvalue of the testing setup the applied load of the vibration generator is multiplied depending on the damping of the system. So the novel fatigue testing facility requires less time and a lower demand of energy than conventional servo hydraulic controlled testing units.

Due to promising results of preliminary tests and the support of Vienna University of Technology a testing facility for large specimens was built in the laboratory of the Institute for Structural Engineering. The u-shaped reaction frame is 16 m long, 2.8 m wide and 2 m high with a total weight of 1500 kN. The reaction frame is made of reinforced high strength concrete and is post-tensioned in three directions in order to remain free of cracks. This is very important to achieve low damping. The testing facility is mounted on spring bearings and is therefore isolated from the foundation.

The new testing method decreases the duration for fatigue tests dramatically and fatigue tests can be carried out more economically. The testing unit is dimensioned for a static tensile load up to 20.000 kN, an upper load for fatigue tests up to 12.000 kN and a vibration range up to 2.500 kN.

Figure 1. Testing facility.

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Durable connections between precast bridge components: Fundamental approach

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ABSTRACT

The Federal Highway Administration statistics indicate that more than 20 percent of the National Highway System (NHS) bridges and 27 percent of non-NHS bridges are structurally deficient or functionally obsolete. Prefabricated bridge elements/systems are widely used to accelerate the construction and assure the quality; thus, increase service life. Accelerated construction as well as the use of prefabricated elements has many inherent advantages including minimum traffic disruption and increased work zone safety. Yet, the failure of empirically designed field implemented joint details for connecting precast components is a major durability concern. Development of joint details to assure monolithic structural behaviour with no cracks and no water leakage under service loads require fundamental understanding of the joint behaviour against design parameters. The common connection details are to use grouted keyways or butted joints in conjunction with posttension.

The objective of this study is to develop an understanding of posttension stress distribution at the joints between structural components such as beams, panels, etc., through a fundamental analytical approach. Understanding of basic relations for stress distribution in panel systems under concentrated loads is vital to develop a fundamental approach for posttension design.

The study evaluates the effects of modular ratio of grout and precast member material and interface friction on posttension stress distribution. The paper also deals with regions of uniform compression under multiple posttension loads. Based on the results of the study, following conclusions can be drawn:

When the grout layer has a substantially small elasticity modulus compared to precast components, post-tension can be distributed at far distances from the load point. By doing so, longitudinal tensile stresses at the edges of the component also increase considerably. Selecting a grout material that has a comparable elasticity modulus to that of precast members is desirable. If grout layers are stiffer, this time, compressed region boundaries get narrower. Hence, use of customized grout properties for specific applications is recommended.

When two surfaces are debonded, both normal and longitudinal stresses increase due to a narrower bearing area. When *component-to-grout thickness* ratio increases, the effects of friction are minimized. Bonding between adjacent precast members is important when dry joints are used to connect members of similar dimensions such as full-depth deck panels.

The joints closest to the load application points would be the most critical due to lack of clamping in between the posttension locations. Using wider fascia members would provide adequate distance for posttension stress distribution even with large duct spacing.

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Bridge strengthening by structural change: from continuous beam to network arch

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ABSTRACT

The paper focuses on solving problems in bridges due to damage suffered in intermediate supports, either by damage to the element, or by undermining or scour problems. These pathologies can be a problem if the conditions of repetitive runoff are aggressive, generating poor security conditions and significant cost in reinforcements.

Traditional repair and strengthening techniques often are ineffective. Therefore the objective of this paper is to evaluate an alternative to these standard solutions. The main idea is to remove the intermediate supports that are damaged by changing the bridge typology, incorporating arches with hangar's arrangement that hold the deck structure, thus allowing to remove the piers.

For the study, a continuous reinforced concrete bridge, built in the town of Chimbarongo, Chile in 1930, is used as application example. The bridge shows severe damage from erosion, corrosion and scour in piers. The methodology is the modeling of the original bridge in a structural analysis program, comparing it with a model of the bridge arches with vertical and network hangar's arrangements.

This procedure confirms the possibility of redress through the network arches, identifying elements and technological procedures for the site, as the use of external brace and construction of the arch on the original deck using it as temporary support during construction and permanent tie in service.

Additionally a transformation of the type of bridge and the optimization of the geometry and materials are obtained.

This new alternative repair is sustainable, because reduces the risk for future problems of infrastructure costs and avoids repetitive and expensive reinforcements. Additionally, gives a value to the concept of repair, providing a new aesthetic concept to the structure.

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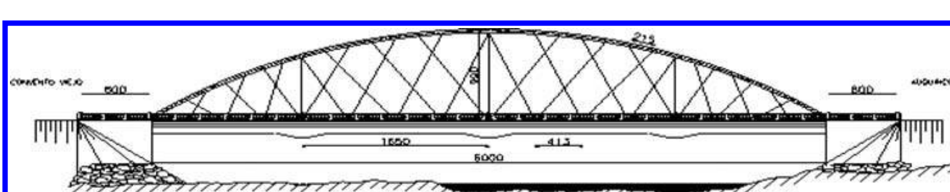

Figure 1. Longitudinal view of San Luis strengthened arch bridge.

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Innovations in fabrication of the self-anchored suspension span of the San Francisco-Oakland Bay Bridge

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ABSTRACT

The 2.2-mile-long East Span of the San Francisco-Oakland Bay Bridge (SFOBB) is currently undergoing a seismic retrofit that will completely replace the existing steel truss bridge that opened in 1936. Approximately 280,000 vehicles cross the SFOBB daily and the bridge corridor is being brought up to current seismic safety standards by replacing the existing East Span. This work is being accomplished while keeping the existing bridge open to traffic. A key component of the New East Span will be a Self-Anchored Suspension (SAS) bridge chosen by the region and legislated to be the signature span in the retrofit of one of the nation's busiest bridges. When complete, this structure will be the world's largest SAS. American Bridge/Fluor Enterprises (AB/F) (A Joint Venture) won the bid to build the SAS. Both companies have impressive portfolios combining years of experience that include constructing the original SFOBB. AB/F determined that the structural steel portion of the SAS would be fabricated in China at the Shanghai Zhenhua Heavy Industry Co. Ltd. (ZPMC), a subsidiary of the China Communication Construction Company (CCCC) on Changxing Island just outside Shanghai. ZPMC is the largest heavy-duty equipment manufacturer in the world and is famous for producing more than 75 percent of the world's port cranes used in the shipping industry as well as other large scale steel bridges such as the Golden Ears Bridge in Vancouver, Canada. ZPMC implemented a number of innovations in order to expedite the fabrication process and to assure meeting the stringent quality requirements of what will soon be a world-renowned architectural icon while remaining the backbone of regional transportation in the San Francisco Bay Area.

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Fatigue life evaluation of existing highway reinforced concrete bridges

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ABSTRACT

Generally, fatigue has not been considered as a key problem in the design of reinforced concrete bridges. Until the 1960s, reinforcement was mild steel and the stresses permitted in the steel bar and the concrete were such that fatigue and fracture failure was believed to be impossible. With the developments of reinforced concrete structures, higher working stresses were permitted and, in particular, high yield reinforcing bars were introduced. In recent years, some studies showed that fatigue could occur in reinforced concrete structures in combination with other causes of deterioration. In the past ten years, considerable increases in traffic volume and wheel loads have caused obviously fatigue damage in existing highway reinforced concrete bridges in China. Some highway reinforced concrete bridges were damaged seriously, leading to the whole bridge collapse sometimes because of overloading and oversize trucks. So how to evaluate the fatigue safety of existing reinforced concrete bridges is an urgent problem in China. In the current paper, the assessment models of existing reinforced concrete bridges based on S-N curve and linear elastic fracture mechanics (LEFM) were proposed. The two methods were applied to the case study bridge, and the fatigue life of the bridge was estimated using the monitoring strain data and considering traffic volume increase rate. In short, the following conclusions could be gained: (1) The S-N curve and LEFM approaches could be effectively applied to evaluate the fatigue safety of actual RC bridges. (2) Combined with other factors of deterioration such as corrosion, corrosion fatigue could cause the remarkable reduction of fatigue life for reinforced concrete bridges. (3) Furthermore, the effect of overloading and oversize trucks should be investigated in-depth.

ACKNOWLEDGEMENT

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Durability rehabilitation of a reinforced concrete bridge damaged by corrosion and overload

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ABSTRACT

Cracking and spalling of concrete cover caused by corrosion of steel rebar and overload is the major damage of reinforced concrete (RC) bridge regarding it's durability in the coastal area. To improve performance of RC structures and prolong their service life, the cracked members must be repaired effectively. The conventional method is removing the cover and recasting it with new concrete, which is destructive and chlorides in the residual concrete can still increase the risk to the further corrosion of steel rebar. Considering the corrosion mechanism of RC structures, a non-destructive electrochemical technique was carried out to rehabilitate a RC bridge damaged by chloride and overload after 30 years' service in the coastal region. Some researchers have focused their interests on the indoor experiments about this method, but there is a gap among indoor experiments, design and practice. In order to make a good understand of this method, this research presented a work on the engineering applications.

The rehabilitation process was introduced in this study. The effectiveness of this strategy was assessed by inspecting the initial and the final chloride ions in concrete together with the carbonated depth. The tested results showed that about 22% to 74% of the initial chloride was removed from the concrete after 40 days' treatment, and the chloride contents close to the rebar were reduced by 49% to 81% of the initial values according to the different treatments. In addition, the carbonated concrete cover had been re-alkalized quickly.

Cracks and repair materials in the damaged zone had significant effects on the desalination efficiency. An effective path for the transportation of chloride must be established between the anode and the rebar in concrete. Therefore, the cracks should be sealed before the application of this technique in order to avoid the short circuits, and the repair materials should be conducive to the transportation of chloride.

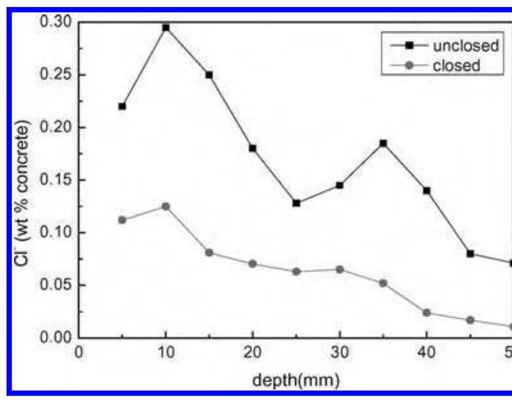

Figure 1. Effect of cracks' treatments on the chloride removal.

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Where are the sustainable bridges in the United States?

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ABSTRACT

Far from the abstraction of a wooden crossing tucked away in a national park, the concept of sustainable bridge design is one that needs further investigation and application by the bridge design, maintenance, and ownership communities.

Sustainable bridge design can be defined as an engineering project that is conceived, designed, constructed, operated, maintained, and eventually put out of service in such a fashion that these activities demand as little as possible from the natural, material and energy resources of the surrounding supporting community. In order to achieve this goal, the designers, owners, and maintenance professionals need to approach each aspect of their craft from new perspectives during each phase of a proposed structure's lifespan.

From this basic definition, a series of standard bridge metrics is proposed, and compared against the benchmark U.S. Green Building Council's benchmark LEED® standard (U.S. Green Building Council, 2006):

Each of these equivalent bridge design goals is delved in further detail, and a series of possible design goals and objectives in each of these categories is posed to form in concept a complete standardized sustainable bridge metric.

Hard and soft advantages for pursuing a sustainable bridge design are presented. While this question is one deserving of further study, hard benefits derive directly from the proposed metric, while the soft benefits of this system range from public approval to the opening up of federally earmarked dollars for sustainable infrastructure and energy savings.

After defining sustainable design and outlining a proposed quantifiable metric, a serious look at the current standard of sustainable bridge design in the United States is examined. Without a national standard or best practice defined, it is difficult to properly compare any two structures that claim the title of sustainable. Both academic study of the benefits and a national standard spearheaded by AASHTO and embedded in their current design and maintenance guides in the United States would be appropriate for any meaningful discussion or practice of sustainable bridge design. This work would also have the additional benefit of reigning in the now wide open field of "green" material vendors and processes.

Table 1. LEED Design Goals vs. Sustainable Bridges.

LEED Design Goals	Equivalent Bridge Design Goal
Sustainable Sites	Sustainable Sites
Water Efficiency	Water Efficiency
Energy and Atmosphere	Energy and Transportation
Materials and Resources	Materials and Resources
Indoor Environmental Quality	N/A
Innovation in Design	Innovation in Design

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A heuristic approach for optimizing bridge inspection route

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ABSTRACT

Bridge inspection, which can identify deteriorations and damages, is a major task of bridge maintenance. In Taiwan, three types of inspection are performed for roadway bridges: (1) daily patrol, (2) regular inspection, and (3) special inspection. All of them are visual inspection without using heavy equipments or instruments.

Daily patrol and special inspection are quick and simple inspection and usually don't need to schedule in advance. Compared with daily patrol and special inspection, regular inspection requires much more time to perform detailed visual inspection. In addition, there are usually time limitations for bridge management agencies to complete their regular inspections, such as before rainy season.

The goal of this research is to establish a two-stage heuristic approach that can help bridge management agencies quickly schedule an optimal inspection plan for daily route. The first stage develops a heuristic procedure to quickly identify a feasible initial solution; the second stage then takes the initial solution from stage one and improves it utilizing Genetic Algorithm (GA).

There are six constraints and assumptions, including (1) the inspection crew will start off from the main office called O, (2) only one inspection crew, (3) the number of work-hour per day is fixed, (4) every bridge's GPS coordinates are known, (5) the travel time between two bridges is calculated by the direct distance, and (6) the inspection of a bridge is inseparable unless the bridge is too long to complete in one day.

The bridge inspection time can be estimated by Equation (1) derived from a multifactor linear regression analysis.

$$Y = 0.02X_1 + 1.14X_2 + 2.58X_3 + 11.47 \tag{1}$$

Where Y = bridge regular inspection time (minute); X_1 = slab area (m^2); X_2 = number of deteriorated component (according to last inspection record); X_3 = number of span.

In this research, a heuristic rule was established to identify a feasible initial route. First of all, select bridges located on main roads and find the direct distance from O based on their GPS coordinates. Second, choose the nearest bridge from O as the first bridge on schedule. Use vehicle speed set by inspector can calculate travel time between O and Bridge 1 (R_{O1}). Meanwhile, one can use Equation 1 to estimate the inspection time of Bridge 1 (T_1). For the same work day, subtracting R_{O1} and T_1 from remaining work hours to see how many hours left. If there is enough time travel to next bridge and complete the inspection, then move to the next bridge. Otherwise, the day is done and start from O to schedule for the next day.

Initial route is a good solution but not optimized. The second-stage utilized GA to improve the initial solution. A real world case of 68 bridges were put into the program and studied. The result of the study case is satisfied. The program rearranged the sequence of bridges, reduced the duration from 15 days to 13 days, and has 6.31 hours remain for the last day.

This research established a model to find an optima route for inspecting a group of bridges. Not only does it find the shortest path that connects every bridge, but also assign all bridges into a certain work day. Multifactor linear regression analysis was applied to estimate the bridge inspection time. There are some plans for future research. The current model uses direct distance between two bridges to calculate the travel time. The future model will incorporate this model with GIS and navigation system to estimate more accurate travel time among bridges.

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Assessment of the operation level of a bridge network in post–earthquake scenarios

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ABSTRACT

This paper presents the results of seismic evaluations at bridge level and network level, carried out on the road network of the Autonomous Province of Trento (APT). The APT Transportation Department operates approximately 2340 kilometers of roads and 1000 bridges, using a real-time internet based Bridge Management System (BMS). This system (for more details of the APT stock, see Zonta et al. 2007) incorporates seismic vulnerability analysis capability based on the definition of fragility curves for all the bridges inventoried. Fragility curves are conditional probabilities that a bridge will reach a certain limit state (operational, damage control, life safety, collapse) under a given ground motion intensity level (Nielson 2005).

Currently in the APT-BMS, fragility curves for a specific bridge are developed by capacity-spectrum approach according to HAZUS guidelines(HAZUS-MH MR3 2003; Basöz & Mander 1999), which is a rapid approach seeking to establish dependable fragility curves. In contrast to other methods that have been used in the past, such as empirical fragility curves or analytical fragility curves that require much previous damage data or extensive computation, only limited information is needed for this model.

Using this approach, the system can generate a probabilistic damage scenario over the whole network, given a simulated design earthquake. We show that, in the case of the APT stock, the direct seismic risk involving collapse or loss of life is moderate. By contrast, we expect a critical problem in network operation in a post-earthquake situation, when it will be necessary to identify the safest path between any two places in APT region.

This problem is addressed using Dijkstra's graph search algorithm (Dijkstra, 1959) to find the shortest path between a source-destination pair for a non-directional and non-negative cost path graph. Giving the simulated network graph of APT region, this method can show the best path rapidly and clearly between any two places on Google Earth, as in figure 1. The results can be used by decision makers to prepare a pre-earthquake plan, and for the post-earthquake emergency response. Google Earth and GIS tools are used to present the results of seismic evaluation and network prioritization. This combination can offer straightforward visualization, and is a great help to government officials and non-professionals in understanding research results.

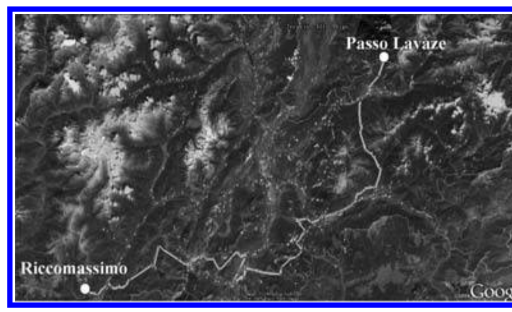

Figure 1. Simulated network and the safest path between Lavazè Pass and Riccomassimo.

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A distributed application for infrastructure risk prevention

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ABSTRACT

Risk mitigation during the life cycle monitoring of critical infrastructure usually involves an expert screen-ing for a proper evaluation. This activity is needed for the risk management in both pre- and post-disaster occurrence. The present paper illustrates a distributed application taking into account the criteria and approaches of the risk management of built infrastructure.

The software was created to code the importance of a report in accordance with the associated risk. So, at any time, the user can see the most important reports or a class of importance selected at his/her choice.

This will help to easily track the most needed infrastructure investment that must be made to avoid for example the associated risks if an earthquake with a certain magnitude occurs. Therefore, this helps in the dis-aster prevention phase. When a disaster occur the most important problem is usual that at the command center mostly disparate and incomplete information will came. As a result, the efficiency of central coordination is highly decreased due to inherent distribution and isolation of intervention teams. By using this tool, any expert that will have either a mini PC or a PDA with GPS connected to the Internet can give all needed information about all kinds of damage and estimation in accordance with his/her domain. The software is created in such as way that any expert has a tracking route that can help a rescue team to save him/her if he/she suddenly disappears without a trace.

From the point of view of the expert, the application is created to be as simple as possible to be used. A login process is required due to the importance of the processed data. After that, the user will have the possibility to see all the reports that are already made, to search from the reports and to modify application settings such as the language, for example.

The proposed methodology is able to provide the critical information for rescue teams, considering also the unsafe position of expert immediately after an extreme seismic event, during the recovery phase.

The paper also illustrates the deployment of information into a geographical map where the information, available in real time & instantaneously for a Central/Regional Command Office, will put in action an alert signal, in emergency situations in case of some critical stages for structures immediately after the seismic event.

The software is a web-enabled GIS-based application. Because it was developed using free maps, it also has lower implementations costs. Some similar applications are already developed but none has the combina-tion of facilities centered to the field report and the real-time transmission using GIS in infrastructure risk prevention. The approach is feasible because the new Internet access for mobile devices seems to be cheaper but also resilient at various types of normal disasters.

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The relevance of whole life costs for infrastructure buildings

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ABSTRACT

Since several years calculation of life cycle costs becomes more and more important and the appraisal of these costs is discussed in an enormous number of papers. But life cycle costs of a bridge are normally defined as direct costs of construction, operation, maintenance, rehabilitation and removal. In contrast whole life costs additionally consider costs generated due to time-delays of the infrastructure users, an increasing number of accidents and cumulative air pollution. These costs are summarized in the category externalities. Furthermore non-construction costs and benefits resulting from improved time of travel, reduced traffic hazard and therefore reduction of accidents have to be taken into account. A classification considering the different cost components is given in ISO 15686.

Since the 1980s different studies for identifying and assessing externalities as well as for the determination of external costs were performed in the traffic sector. Up to now no detailed analysis of external costs from bridges exists. If external costs are to be calculated and internalized a monetarily quantification is required. In the course of cost calculation it is necessary to find a proper procedure for the assessment of goods which are not part of a market and for whom no sales values are existing.

In most cases external costs arising during the whole life cycle of a bridge exceed the original building costs by far. The biggest impacts result from time delay in the course of an obstruction of traffic. Other relevant external costs are basically related to the components accidents, air pollution and climate change. However, the amount of costs shows a significant variety in different studies and guidelines. Depending on the assumptions, considered externalities and the kind of approach for a consideration of long term effects the results vary within a broad range. The clear results point out that an approximate calculation of external costs is better than neglecting them. Therefore a consideration of external costs is indispensable to achieve efficient macroeconomic decisions.

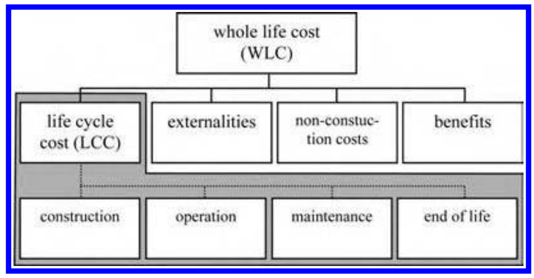

Figure 1. Classification of costs according to ISO 15686.

Due to the great importance to consider all categories of the whole life costs and the uncertainties described in this paper, further examinations especially for building activities in the infrastructure sector have to be realized. Therefore a structural bridge design has to be developed, which is not only minimizing manufacturing and maintenance costs but also arising external costs. The goal must remain to create a holistic assessment system for bridges. The challenging problem is to perform a quantification of external costs and benefits.

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