Safety and Reliability of Bridge Structures

Edited by Khaled M. Mahmoud

CRC Press
Taylor & Francis Group
A BALKEMA BOOK

SAFETY AND RELIABILITY OF BRIDGE STRUCTURES

Safety and Reliability of Bridge Structures

Edited by

Khaled M. Mahmoud
Bridge Technology Consulting
New York City, USA

CRC Press
Taylor & Francis Group
Boca Raton London New York Leiden

CRC Press is an imprint of the
Taylor & Francis Group, an **informa** business
A BALKEMA BOOK

Front Cover:

Kanchanaphisek Bridge, Samut Prakan Province, Thailand
Photo courtesy of Parsons Brinckerhoff, USA

Back Cover:
(From top to bottom)

Grimselsee Bridge, Bern, Switzerland
Rendering courtesy of Christian Menn, Switzerland

Kingston-Rhinecliff Bridge, Kingston, New York, USA
Photo courtesy of New York State Bridge Authority, USA

Cover Design:
Khaled M. Mahmoud
Bridge Technology Consulting, New York City, USA

CRC Press/Balkema is an imprint of the Taylor & Francis Group, an informa business

Typeset by Vikatan Publishing Solutions (P) Ltd., Chennai, India
Printed and bound in the USA by Edwards Brothers, Inc, Lillington, NC

Published by: CRC Press/Balkema
P.O. Box 447, 2300 AK Leiden, The Netherlands
e-mail: Pub.NL@taylorandfrancis.com
www.crcpress.com – www.taylorandfrancis.co.uk – www.balkema.nl

ISBN: 978-0-415-56484-7 (Hbk)
ISBN: 978-0-203-86158-5 (Ebook)

Table of contents

Preface

Recent surveys of the U.S. infrastructure's condition have rated a staggering number of bridges structurally deficient or functionally obsolete. While not necessarily unsafe, a structurally deficient bridge must be posted for weight and have limits for speed, due to its deteriorated structural components. Bridges with old design features that cannot safely accommodate current traffic volumes, and vehicle sizes and weights are classified as functionally obsolete. In addition to affecting safe and efficient people mobility and movement of goods and services, these structural and functional deficiencies contribute to traffic congestion. The restrictions may also result in increased response times for emergency vehicles required to use alternate routes. As importantly, such deficiencies may adversely affect the performance of transportation systems in emergency situations or for disaster response. This narrative has become part of the public debate sparked by the collapse of the I-35W Bridge over the Mississippi River in Minneapolis, Minnesota, USA, during rush hour on August 1, 2007, plunging dozens of cars and their occupants into the river. Ever since, numerous technical and news articles have been written to answer the persistent question, *why did the bridge collapse?* Exhaustive examination of the details of a specific bridge failure, typically, reveals the reasons for the collapse and lessons are drawn from the experience. Each bridge failure, since the Tacoma Narrows Bridge disaster in 1940, has served as a wakeup call for the bridge engineering community, initiating radical changes in the design and construction standards. However, a paradigm shift is necessary in the inspection and monitoring practices of the bridge engineering community to provide preventive maintenance and restore the public's confidence in the safety of bridges.

Concerns about bridge safety and reliability are shared by bridge engineers from different countries. This book contains a number of selected papers that were presented at the Fifth New York City Bridge Conference, held on August 17–18, 2009. These papers cover a wide range of topics in the design, construction, maintenance, monitoring and rehabilitation of bridge structures.

The book leads off with a paper by Galambos on "The safety of bridges", in which the author attempts to present an explanation to the tragic events of August 1, 2007, when the bridge carrying the Interstate Highway I-35W over the Mississippi River in Minneapolis, Minnesota, USA, collapsed unexpectedly, plunging the afternoon rush-hour traffic down with the bridge. The author recommends more thorough design checks for new bridges, inspection of older bridges by experienced bridge engineers and the employment of monitoring techniques so that a collapse like the Minneapolis Bridge remains a very rare event. In October of 2007 the AASHTO LRFD Bridge Design Specification became the mandatory design code for highway structures in the United States, to establish better representation of applied loads and strength of materials. In "LRFD versus ASD, the differences between the two standards for retaining wall and abutment design", Esposito and Najm take an in-depth look at the differences between the Standard Specifications and the LRFD Specification and how they apply to earth retaining structures. Calculations for several examples are carried out to examine the effect of the new design standards on structural proportioning and cost. For structures 4.5 meters (15 ft) and taller, the difference between the LRFD and the Standard Specifications was minimal. However, designing shorter walls using the LRFD code can result in an increase in footing length of about 30% and an increase in cost of about 10%. While these cost impacts to a structure can be significant, the new code provides a safe and reliable method of design which is of paramount importance to engineers and the public. Over recent years, the biggest thrust in bridge design has been economy of structure, where the bottom line has become an important criterion. With the recent focus on the aging and ailing infrastructure, coupled with a shortage of funding, this has become a deepened mindset with transportation agencies, designers, and contractors who all believe that they need to do the most for the least cost. In "Providing the best bridges for the best cost", Lawrie presents a viewpoint that optimizes structural efficiency, provides aesthetics and economy to be able to do better work for the same, or even a lesser cost. Constructed in 2007, the $15 million Ramp ‘X’ Bridge project was designed as the final stage of the 5-stage Hawthorne-to-Pleasantville corridor improvement process initiated by the New York State Department of Transportation approximately 25 years ago. The bridge connects southbound Route 9A to Southbound Taconic State Parkway. In “Taconic State Parkway ramp ‘X’ bridge project, Westchester, NY, USA”, Jarosz and Kang present the details of the bridge design. The central feature of the project is a sharply skewed flyover bridge tightly aligned on compound horizontal alignment involving a circular arc segment joined by a tangent. The irregular curvature of the bridge’s steel framing required detailed analysis of stresses and deflections using finite element method. The average span of the curved girders is about 51 meters (167 ft.) as the span follows a pronounced vertical curvature which further complicated the detailing and fabrication of superstructure. The bridge deck crosses the 6-lane State Highway 9A joining its southbound lanes with those of the Taconic State Parkway. In the aftermath of the I-35W Bridge collapse in Minnesota, the Federal Highway Administration issued a technical advisory to bridge owners to check the status of gusset plates on similar bridges. In response, the Pennsylvania Turnpike Commission initiated a screening program for the analysis of the Pennsylvania Turnpike’s Hawk Falls Bridge. Myers presents the “Screening and analysis for the gusset plates of the Hawk Falls Bridge, Pennsylvania, USA.” The 51-year old, 225.5 meters (740 ft) long bridge spans Mud Run and contains a three-span continuous, haunched Warren deck truss. An analysis of the bridge was performed using BAR7. The gusset plates were grouped into representative types and each type was analyzed considering worst-case loads. A finite element model was developed, which verified the results of the hand calculations, finding some stresses above allowable values, but below material yield strength. A monitoring program was subsequently implemented, and data will be collected through sensors and strain gages for one year to accurately evaluate the bridge and make decisions regarding future rehabilitation. The construction of the New St. Anthony Falls Bridge over the Mississippi River in Minneapolis presented many challenges. In coming up with a highly functional and aesthetically pleasing bridge. One of the important aspects of the project that has not commanded a great deal of attention is the design and selection of bearings to accommodate the loads, movements and rotations of this signature bridge. In “Selection and design of the high load multirotational bearings for the St. Anthony Falls Bridge in Minnesota”, Western et al. present the details of the design and selection process for high load multi-rotational devices that ranged in vertical load resistance from 5330 kN (1200 kips) to 25,780 kN (5800 kips).

Suspension bridges with multi main-spans present an attractive solution due to economic, structural and aesthetic advantages. Continuous multi main-spans suspension bridges with each main span of more than 500 meters (1640 ft) are planned in some countries, and a four span continuous multi main-spans suspension bridge is now under construction in China, with about1000 meters (3280 ft) length in each main span. The “Potential of multi main-spans suspension bridge” is presented by Inoue for further applications of the concept as an attractive solution for a long span bridges with safety, reasonable cost and elegant features. The paper shows that the interior tower is a key component that plays an important role not only to the bridge performance but also to the project cost. The paper proposes that the minimum requirement for the interior tower is to have an appropriate equivalent spring coefficient. The author notes that additional equipment may be required to be installed on the saddle of interior tower to provide sufficient cable slippage resistance. With a 500 meter (1640 ft) long main span, the Kanchanaphisek Bridge is the longest span in Thailand. The bridge was designed to require minimal maintenance and provide access to all critical structural components for future inspection and maintenance. Simple low cost ladders and platforms allow ready use access to the stay cable top and bottom anchors for inspection and maintenance. In “Kanchanaphisek Bridge: A cable-stayed bridge designed for a low maintenance budget”, Hsu and Vineyard present details of the concrete counterweights, instead of mechanical tie-downs, that were used to eliminate uplift at the anchor piers of the cable-stayed bridge. The bridge integral anchor piers and innovative “floating” connection at the towers eliminated the need for bearings on the entire bridge. Therefore the need for costly bearing inspection and replacement has been eliminated. To expedite construction, the project for the new bridge carrying the SR1 Coastal Highway across the Indian River Inlet in Delaware, USA was procured as a design-build contract. The main span consists of a concrete cable-stayed bridge with a 290 meters (950 ft) main span and 122 meters (400 ft) back spans. The superstructure components include edge girders, floor beams and a concrete slab. It is supported by two vertical planes of stay cables anchored in the edge girders. The stay cables are anchored in two vertical reinforced concrete pylons using structural steel anchorage boxes to resist longitudinal tensions across the pylon section. The floor beams and edge girders are post-tensioned, as well as the top slab in the longitudinal direction in the vicinity of the transition piers and center portion of the main span. In "Design-build of the Indian River Inlet cable stayed bridge", Butler et al. present the design aspects and discuss several innovative design-build features for the project. Recently, San Diego has commissioned a landmark pedestrian bridge from its new ballpark—over several sets of train tracks, and over a busy downtown thoroughfare—to the convention center and the San Diego Bay. The bridge design is a self-anchored suspension bridge with a single inclined pylon. The main span of the bridge is 108 meters (354 ft) and the pylon is 40 meters (131 ft) tall. The pylon is inclined at a 60 degree angle from the horizontal and leans over the deck to support the single pair of suspension cables. 34 individual suspenders attached to the main cable support the 6 meter (20 ft) wide deck from the top of the railing at only one edge of the deck. The bridge is horizontally curved and a tendon is stressed at the top of the railing. The radial force generated by the tendon above the deck elevation generates a restoring moment which balances the forces in the bridge deck. In "A self anchored suspension pedestrian bridge over Harbor Drive in San Diego, California, USA", Fitzwilliam and Tognoli describe the design process for the bridge from the type selection phase through final design and present different features of the bridge. Stay cable replacement is highly technical work that requires specialized equipment, techniques and engineering at all stages of the operation. Structural assessment and design, construction methods, work schedule and organization are closely interconnected and need to be studied in a very coordinated manner by teams having experience in all of these fields. In "Stay cable replacement, high level engineering for an extended serviceability", Mellier et al. present some of these issues to optimize stay cable replacement operations.

Movable bridges are complex structures due to the necessary integration of mechanical components, control systems and structural members of the bridge. Many of these bridges have been constructed over 100 years ago. Therefore they are in need of serious retrofit to restore to current service standards. The Bridge Street Bridge was constructed in 1880 by the King Iron Bridge Company of Cleveland, Ohio. It survives as a unique example of a nineteenth century hand operated mechanical drawbridge. The lifting system is based on a French design from the early 19th Century. This French design had a unique solution to dealing with the variation of bridge balance required while opening a bascule span. This system allowed the bridge to be opened and closed by hand with a relatively constant effort. In "Rehabilitation of Bridge Street Bridge preserving a nineteenth century historic bascule lift span", van Hagen and Protin discuss some of the unique characteristic of the basic bridge design to maintain an important link to Rockland County's historic heritage. The replacement of a movable bridge in New York City presents a challenge to the owner, designers and constructor. In "Replacement of a rare Hanover skewed bascule—The Hamilton Avenue Bridge, New York City, USA", Griesing addresses the design constraints of this project with respect to the complexity of a skewed bascule bridge replacement in sixty-four days. Cost of bridge retrofits present one of the most significant challenges for bridge owners and designers. After bids came in 73% higher than the Engineer's Estimate, the U.S. Coast Guard elected to perform a Value Engineering Review of a movable rail bridge over the Mobile River in Alabama. In "Innovative redesign for cost savings on a vertical lift bridge", Kelly and Nickoley discuss the original structure configuration, contractor's input regarding the erection, staging, closures to rail and marine traffic associated with the original design, and main details of the redesign which resulted in cost savings.

The advent of construction materials allows weight reduction and enhances structural performance and durability of bridge structures. Lightweight concrete has been used for long span bridges for many years, including its recent use in some major structures in the US and overseas. In "Lightweight concrete for long span bridges", Castrodale and Harmon present an overview of lightweight aggregate and lightweight concrete and the properties affecting construction, and structural performance. The authors discuss selected long-span bridge projects on which lightweight concrete has been used such as the San Francisco-Oakland Bay Bridge and the Chesapeake Bay Bridge in the USA, and some examples of long-span segmental concrete bridges in the USA and Norway. In "Experimental study on hybrid steel-concrete beam connected with perfobond ribs", Kim et al. describe the experimental results of hybrid steel-PSC (prestressed concrete) beam connected with perfobond rib shear connectors. In order to enhance structural behaviors of hybrid beams and convenience of construction, the perfobond rib shear connectors are applied to the upper and lower plate located between a steel girder and a PSC girder instead of using the shear studs. Under the three point loading condition, various beams with a different length are tested to evaluate the flexural characteristics of hybrid beams and joints. The authors conclude that the applied joint type with perfobond rib shear connectors has good functionality on transmitting the load to the steel and PSC parts. The premise of "Rehabilitation of bridges using ultra-high performance fibre reinforced concrete", as presented by Brühwiler, is based on the idea of using Ultra-High Performance Fiber Reinforced Concrete (UHPFRC) to "harden" those zones of the structure that are exposed to severe environment and high mechanical loading. All other parts of the structure remain in conventional structural concrete as these parts are subjected to relatively moderate exposure. The paper describes four applications of the UHPFRC technology for cast in-situ and prefabrication using standard equipment for concrete manufacturing. When built, the Messina Straight Bridge in Italy will be the longest suspension bridge in the world, with a center span length exceeding three kilometers. The use of lightweight materials would therefore be essential to reduce the overall weight of the superstructure. DiGiannantonio presents the advantages of using "Lightweight and recycled materials for the construction of the Messina Strait Bridge, Italy."

Bridge instrumentation and monitoring technologies play an important role in keeping track of the condition of bridge components. Bridge owners have recently started incorporating instrumentation of sensor technologies to maintain good health of their structures. The Massachusetts Port Authority has recently contracted consultant services for the structural modeling and instrumentation of the Maurice J. Tobin Memorial Bridge. In "Modeling and instrumentation of the Tobin Memorial Bridge in Boston, Massachusetts, USA", Brenner et al. present the analytical structural models and the instrumentation plan that included strain gages, accelerometers, tiltmeters, and temperature sensors. The collected data will be used to verify the analytical structural models. These models and instrumentation plans will be used as part of the structural health monitoring and condition assessment program for the management of the Tobin Memorial Bridge. Acoustic emission (AE) monitoring of steel fracture-critical members and concrete structures provides valuable input to condition-based maintenance planning and the allocation of maintenance resources. The instrumentation used for AE monitoring, however, is generally power hungry, expensive, bulky, and requires extensive cable installation. Merging acoustic emission technology with wireless technology is required to overcome some of these limitations as they relate to long term bridge monitoring. In "Structural health monitoring using wireless acoustic emission sensor network", Hay et al. discuss recent work on the development and application of wireless acoustic emission technology for structural health monitoring of aging bridges. The paper discusses bridging the gap between fracture mechanics and acoustic emission information to assist in predicting crack growth rates and remaining life estimates. The authors also discuss the practical issues associated with wireless sensor integration in terms of power management, information management, and communication. In "Bridge monitoring to measure corrosion rate and concrete resistivity", Moretti and Jerath are set out to establish whether the permeability of concrete bridge decks will be reduced by adding mineral admixtures. Three bridge decks were monitored, one bridge was constructed with conventional Portland cement concrete, the second bridge was constructed with a concrete containing 38% replacement of Portland cement by fly ash , and the third bridge contained 35% ground granulated blast furnace slag (GGBFS) as a partial replacement of Portland cement in its concrete. Corrosion of reinforcement and the concrete resistivity were monitored at the three bridges between 2002 and 2008 by using the Gecor 6 instrument. The authors report that the corrosion rate is the highest in the bridge containing no mineral admixture, the next high values are in the bridge containing GGBFS and the least corrosion rate existed in the bridge whose concrete contained fly ash.

The ability to provide a secure environment in the bridge community today is a daunting task being undertaken by many engineers worldwide. One of the key structural components identified in long span bridges is the steel cable assembly. In his paper "The security assessment of structural cable assemblies in bridges", Klein focuses on identifying the vulnerability of the cable assembly in the event of an accident, disaster, or attack. The paper describes the response of traditional and experimental cable assemblies to moderate and extreme heat. The results from high temperature testing performed on mock cable assemblies under load in controlled test environments are discussed in this paper. The author gives an overview of the new practices being developed in the wire rope industry to increase the security and functionality of cable assemblies currently used in vertical lift bridges. New high performance materials have been shown to drastically increase the functional operating temperature of the cable assemblies without affecting the mechanical properties of the wire rope. The paper also discusses the theoretical and experimental work regarding the operating mechanisms of wire rope held in conventional tapered sockets. Perimeter Intrusion Detection Systems (PIDS) can be deployed as a physical security tool at bridges. If PIDS is deployed as part of an overall multi-stage physical security system, the holistic approach to security will position owners and operators to pro-actively detect unauthorized intrusions, deter attacks, deny access and defend the facility. In their paper, Godoy and Chang present an "Overview of available detection technologies with applications to bridge security." Cracks were found in several floorbeam-to-tie girder connection angles for a tied arch during a biannual inspection. After replacing those damaged angles and frozen stringer bearings, the cracks appeared soon. A series of diagnostic tests were performed in order to identify the crack causes. In "Development of a baseline model for an arch using diagnostic tests", Liu and Shenton develop global and local finite element models and calibrate those models to obtain a baseline model using testing data to find the crack causes.

Bridge inspection, management and assessment represent an important component in the safety and reliability of bridges. Waterways Ireland is responsible for managing and maintaining Ireland's navigable inland waterways, as established under the British-Irish Agreement in December 1999. As part of this cross-border role, Waterways Ireland manages and maintains bridges and an array of other structures on its seven major waterways in order to fulfill their mission of providing a high quality recreational environment on the waterways in their care, for the benefit of all. Currently, there are 580 structures spanning over the 1,000 kilometers of waterways and their associated branches. Of these 580 structures, 360 are owned, managed and maintained by Waterways Ireland. Recently, Waterways Ireland initiated a program to develop a Business Process Model for managing their bridges. In "Developing a business model for bridge management in Europe", Browne et al. describe a Business Process Model, which is comprised of a set of logical diagrams, textual descriptions and data elements. These components provide both an overview and an in depth guide to the procedures involved with managing Waterways Ireland's bridge stock, in a format understood by managers and personnel within Waterways Ireland. With federal budget deficits on the order of $1 trillion, it is unrealistic to expect government financing to appreciably close the infrastructure funding gap. Although private capital is available to this end, there remains considerable resistance in the U.S. to the use of private investment to finance public infrastructure projects despite its general acceptance in other parts of the world. Properly applied and structured, public-private partnerships (PPP) can offer significant advantages to a traditional approach, particularly in the delivery of large-scale bridge projects. In "Use of public-private partnerships in bridge infrastructure delivery", Chang discusses the contemplated use of PPP to replace a number of major bridge crossings in New York. The paper concludes that public-private partnerships have an important role to play in any comprehensive strategy to renew the decaying infrastructure. As the civil engineering profession continues to grow with our ever aging infrastructure, structural health monitoring using nondestructive load testing techniques with comparison to an analytical structural model is rapidly becoming an economical method for decision-making related to asset management. A structural model, verified with collected field data, can provide an objective basis on the decisions to repair or replace bridges and the importance of each action to the safety of the driving public to determine the order in which bridge repairs need to be implemented. Bell and Sipple discuss "Special topics studies for baseline structural modeling for condition assessment of in-service bridges". New Jersey Turnpike Authority's bridge inspection program is evolving to encompass its recently combined inventory of 1,000 bridges on the New Jersey Turnpike and Garden State Parkway, in a challenging and high-usage environment. In "Merging and moving forward with New Jersey Turnpike Authority's bridge inspection program", Laird and Paul highlight recent modifications and innovations, and the implementation of a new computerized bridge inspection and management database that is capable of handling the diverse structure populations on both roadways. The Washington Metropolitan Area Transit Authority (WMATA) "DC Metro" maintains a large network of bridge and aerial structures to support about 170 kilometers (106 miles) of system track. A variety of bridge structures must be maintained and inspected that support over 32 kilometers (20 miles) of elevated track. The Metro system recently celebrated its 30th anniversary of operation and is increasingly dealing with the effects of an aging infrastructure while also planning for major system expansions. In order to ensure the maximal safety of nearly 800,000 daily users WMATA has a dedicated workforce of 16 inspectors to conduct annual inspections of bridges and other structures. In "Washington Metropolitan Area Transit Authority's computerized structure inspection system", Shaffer and Schellhase present an overview of the structures maintained by WMATA, including bridges and tunnels. A significant focus will be on a recent adoption of a new system implemented for conducting the inspections and managing the data utilizing tablet computers in the field and integrated network access.

After a bridge is designed and let for construction, the engineering of the project does not cease. There are many different aspects of construction that should be investigated by a professional engineer, working for a contractor, which will provide for safe and efficient construction. The engineering effort during construction typically consists of both design computations and procedures with figures. In "Construction engineering considerations for highway bridges", Chavel et al. discuss some of the issues a bridge engineer may face when working on the construction side of a project. The paper examines design items typically encountered during bridge construction, including temporary shoring walls, erection and demolition procedures and calculations, construction loading, temporary support structures, girder bracing, cranes and construction equipment. The concept of accelerated bridge construction (ABC) offers significant advantages over the conventional cast-in-place construction in minimizing traffic disruption, improving work zone safety, and reducing on-site environmental impacts. For these reasons, the California Department of Transportation (Caltrans) has embraced the ABC philosophy in bridge design and construction projects. In "Concept and practice of accelerated bridge construction in California, USA", Chung et al. review the ABC practices in California over the last five years. New concepts in developing an ABC project such as construction impact index, impact tolerant index and project delivery index are proposed and analyzed, and the design impact questions are developed for quantifying the indices. The paper also describes a planned ABC project located in Los Angeles urban area. The main expansion joints on the Forth Road Suspension Bridge have been in service since the bridge opened in 1964. Since then, traffic volumes and the weight of vehicles crossing the bridge have increased significantly. The joints are of the rolling plate type whereby a series of plates slide over fixed curved girders. There are eight joints in total and they allow movement of the 1006 metre long main span and 408 metre side spans and are thought to be the largest of their type in Europe. The joints are now considered to have reached the end of their service life and a decision was taken to replace them. It was deemed unacceptable to close traffic lanes for the period required for the work to be carried out. Therefore, temporary ramps were devised to be constructed over the joint locations to allow traffic to pass over the replacement works. However, a combination of a high tender price, due mainly to the high cost of the temporary works required to keep the traffic moving, and the announcement of a proposal to build a new bridge across the Forth to be open in 2016 led to a review of the project to determine if the service life of the joints could be extended until the new bridge opened. Extending the life of the joints until the new crossing was opened would lead to a saving of some £6 million. In "The maintenance of the main expansion joints on the Forth

Road Bridge, Scotland", Colford et al. provide description of the analysis conducted to identify the likelihood and consequences of failure of the various components that make up the joints. Once the failure mode and consequence of that failure of the individual components had been identified, an analysis to determine what mitigation could be put in place was then carried out. The analysis described in the paper concluded that the service life of the joints could be extended by increasing inspection and monitoring and by installing failsafe devices without reducing the current high operational safety levels on the bridge. However, it was recognized that there was a risk of a reduction in operational service levels which had to be balanced against the potential cost savings. The Filetto Bridge is an historic prestressed reinforced concrete bridge near Bologna in Italy. Preliminary studies to investigate the seismic behavior of the bridge showed that the existing structure is not able to support current code seismic excitations. The results obtained from several static and dynamic analyses showed the necessity of a reinforcement design. In "Reinforcement design of the Filetto Bridge on the Santerno River near Bologna, Italy", Gasparini et al. describe the fundamental solutions which provide the structure with the best possible performances such as the insertion of elastomeric isolators between the deck and the piers and the use of highly resistant carbon fiber strips in order to restore the loss of post-tension effect of the beams. The Dumbarton Rail Corridor is located in the southern region of the San Francisco Bay Area, in connecting east and west bay. The crossing of the Bay includes two bridges: the Dumbarton Bridge and Newark Slough Bridge. Portions of these Bridges have timber and steel truss bridge sections that are approximately 100 years old, and each has a 100-year old swing span. The existing movable bridges have not been operational since the mid-1980's and portions of the existing timber bridges were damaged in a fire in the late 1990's. The Dumbarton Rail Corridor (DRC) Project proposes to provide commuter rail service by improving the existing rail infrastructure in the DRC Corridor. In order to reopen the Dumbarton and Newark Slough Bridges to accommodate the planned Dumbarton commuter rail service, several activities were performed in order to help determine if the bridges should be retrofitted or replaced. In "Retrofit and replacement of Dumbarton railroad bridges, California, USA", Go et al. describe the different studies carried out for the retrofit and replacement project including inspections, concrete coring, condition assessment of existing steel and concrete structures, load rating analyses, movable bridge evaluation, seismic analyses, constructability review and cost estimates.

Bridge decks are subject to significant wear and tear and the evaluation of their condition is of paramount importance from the viewpoints of structural integrity and quality of ride. The Kingston-Rhinecliff Bridge is a 1585 meters (5,200-ft) long deck truss, located in Kingston county of the Hudson Valley in New York. The bridge was opened to traffic in February 1957, and has gone through much extensive deck repairs over the years. In 1997, a project began to replace the deck with an HS25 rated system. Based on a cost benefit analysis, a concrete filled grid, known as Exodermic Deck, was selected. In "Understanding the limitations of exodermic bridge decks: A case study on the Kingston-Rhinecliff Bridge, Kingston, NY, USA", Moreau focuses on outlining some of the misconceptions regarding the proper application and performance with Exodermic Deck system. The paper discusses the shear flow mechanism between the steel and the concrete deck and outlines retrofitting measures for the inadequate deck system applications to provide a long and durable life. Wisconsin Department of Transportation bridge maintenance engineers deployed nondestructive test methods on 183 bridge decks during 2007 and 2008 to more efficiently focus preventative maintenance and rehabilitation efforts. In addition to the usual visual inspections, ground penetrating radar (GPR) and infrared thermography (IR) were used to determine bridge deck condition. For each deck a Level 1 evaluation was initially conducted to screen out decks in good condition, and a Level 2 evaluation was then carried out on decks that were likely candidates for future maintenance and rehabilitation. This multi-level approach utilized the strengths of each method to develop a cost effective way of assessing deck condition. In their paper, "Multi-level bridge deck evaluation combining ground penetrating radar and infrared thermography", Maser and Miller describe the methodology, the equipment used, the field data collection techniques, and the analysis procedures used to integrate the data. The prefabricated modular nature of grid reinforced concrete deck naturally lends to reduced construction periods and offers the advantage of shortened work windows—nighttime or weekend—when traffic volumes are lower and the travelling public is less encumbered. In addition, the reduced weight translates into direct savings in the superstructure and substructure for new construction and can minimize superstructure rehabilitation and increase live load capacity of existing structures. A few recent projects accentuate the role of grid reinforced concrete decks as the solution to lightweight, rapid deck replacements as discussed by Gase and Kaczinski , in their paper "The history and benefits of prefabricated grid reinforced concrete decks."

Bridge structures are symbols of their representative cities and countries and they represent icons of aesthetics in the landscape. Beautiful bridge designs achieve a delicate balance between function and form, and aim to optimize the flow of forces with the objective to reduce structural elements and their dimensions to a minimum. In "Christian Menn's recent bridge designs—reducing structural elements to the simplest solution", Brühwiler presents the conceptual designs by Christian Menn of four landmark bridges. The four bridges express technical efficiency by slenderness and transparency and emphasize the importance of understanding the functioning of the structural systems. A sound engineering concept is thus the solid basis for a far-reaching aesthetic quality and for the conceptual design of simple but elegant structures that offer a great crossing experience while respecting the functional requirements and providing technical performance. Gustave Eiffel and Othmar Ammann each designed great two-hinge arch bridges that have been named landmarks by the American Society of Civil Engineers. Eiffel's Maria Pia Bridge exhibits the crescent form with an aesthetic of lightness. Ammann's Bayonne Bridge is of the spandrel form and is influenced by an aesthetic of monumentality. Referring to archival research and technical analyses, in "A study of the evolution of arch forms: Eiffel's Maria Pia Bridge and Ammann's Bayonne Bridge", Thrall and Billington show that each is a major work of structural art by adhering to the follow tenets: efficiency (minimum material consistent with satisfactory performance and assured safety), economy (competitive construction cost consistent with minimal maintenance requirements), and elegance (aesthetically striking consistent with efficiency and economy). The authors present a quantitative comparison between the crescent and spandrel forms of two-hinge arch by optimizing each with respect to member cross-section. The paper concludes that the crescent form is more efficient based on a lower self-weight after optimization. The "Design and construction of the Third Millenium Bridge over the Ebro River in Zaragoza, Spain", by Arenas de Pablo et al. is described from conception to construction. The Third Millennium Bridge is a bowstring arch entirely made of high strength, self-compacting white concrete and located in an urban environment. The Poughkeepsie-Highland Railroad Bridge was originally opened to railroad traffic in 1888. At the time of its opening, and at 2 kilometers (1.25 miles) in length, the Poughkeepsie-Highland Railroad Bridge was touted as the longest bridge in the world. Following a fire in 1974, the bridge was closed to all traffic. Walkway Over the Hudson, a private non-profit organization, envisioned a re-birth of this historic structure as the world's longest pedestrian/bike bridge. This concept gained traction in 2007, as the idea of opening the world's longest pedestrian bridge was linked to the Fall 2009 Quadricentennial of Henry Hudson's exploration of the Hudson Valley. New York State Office of Parks Recreation and Historic Preservation will operate the bridge as New York's newest State Park. To meet this target event, the project had to progress on a very aggressive accelerated schedule. Project processes and determinations had to comply with a myriad of public/private funding partners. And, concerns of a host of third party permit grantees and stakeholders had to be addressed. The private sector 'owners' of the project were focused on the budget and the schedule. In "Walkway over the Hudson—Fast track design and construction", Melewski et al. describe the public-private partnership fund mechanism and different features that helped the project advance. Henry Meiggs of San Francisco, a fugitive from the U.S. Government, signed a contract with the Peruvian Government in 1869 to build a 219 kilometers (136 miles) railroad connecting Callao, the main port of Peru and Lima, the Capital, with Oroya, a small town close to the mining district of Cerro de Pasco in six years for U.S. $25,875,000. Mr. Meiggs was the biggest employer of U.S. engineers next to the U.S. Army at that time. The Verrugas Viaduct was 83 kilometers (51.8 miles) from Lima and 1783 meters (5,850 ft) above sea level in the Andes mountains. The wrought iron viaduct was designed, fabricated and shipped by the Baltimore

Bridge Company via land and sea to Callao from where it was transported to the site on rail and mules. The viaduct was opened to traffic on January 8, 1873. In March 1889, heavy floods and rock slides in the gorge pushed the central pier causing collapse of the bridge. Mr. Leffert L. Buck was retained for the replacement bridge and he designed a cantilever bridge with two piers, two side spans of 43 meters (140 ft) each, and a central opening of 72 meters (235 ft). Construction started on July 1, 1890, and the bridge was completed on January 1, 1891. In "Verrugas Viaduct and its reconstruction, Peru, South America", Gandhi describes details of both bridges, difficulties encountered during their constructions, and people connected with them. Finally, Capellán Miguel et al. present on the "Design and erection of four signature urban bridges recently built in Spain." Every design has special architectural and urban values and all of them were developed searching for new structural and aesthetic solutions to solve each different kind of crossing. The four designs aim to represent the cities where they are located.

The editor is indebted to the authors and reviewers of the papers contained in this volume. The presented work is an invaluable contribution to the safety and reliability of bridges.

Khaled M. Mahmoud, PhD, PE
Chairman of Bridge Engineering Association
www.bridgeengineer.org
President of Bridge Technology Consulting
www.kmbtc.com
New York City, USA

New York City, August 2009

1 *Bridge safety, analysis and design*

Chapter 1

The safety of bridges[1]

T.V. Galambos
Department of Civil Engineering, University of Minnesota, Minneapolis, Minnesota, USA

ABSTRACT: On August 1st, 2007, the bridge carrying the Interstate Highway I35W over the Mississippi River in Minneapolis, Minnesota collapsed unexpectedly, plunging the afternoon rush-hour traffic down with the bridge. Traffic management cameras recorded the demise of the bridge, leaving the whole world witness. The collapse left everyone wondering how something like this can happen, and questioned the safety of driving over any bridge. These thoughts presented a need to be answered by the bridge engineering community. This paper is an attempt to provide a personal response, as an engineer, to the questions of why bridges collapse, and what engineers should do about it within the context of what is already being done.

1 INTRODUCTION

At approximately 6 PM on August 1st, 2007, the bridge carrying Interstate Highway I35W over the Mississippi River in Minneapolis suddenly collapsed, plunging the rush-hour traffic some 35 meters down with the bridge or into the river. Many of the people in the cars, buses or trucks survived the unexpected drop, however, there were 13 fatalities and some 80 injuries. Rescue response was almost immediate and many heroic acts were performed by the surviving passengers, the police, and other rescue workers, demonstrating that the community was prepared for an emergency wherever it was to come from.

The collapse of this bridge was sudden, unexpected, and complete. The structure essentially disintegrated within seconds. The demise of the bridge was recorded by traffic management cameras and within minutes the whole world witnessed the catastrophe. The questions on the minds of everybody were: "How can something like this happen?"; "Is it safe to drive over any bridge?"; "Who is responsible?" These thoughts are entirely reasonable and they need to be answered by the bridge engineering community in order to re-establish confidence in this part of our infrastructure. This essay is an attempt to provide my personal response to the questions of why bridges collapse and what we as engineers should do about it in addition to what is already being done.

Bridges define the character of a city. Whenever I think of New York, San Francisco, London, Paris, Lisbon or Budapest, the bridges come to mind. Bridges are the pride of the citizens. They not only provide transportation links, but they are part of the architectural and literary heritage and the aesthetic landscape of civilization. A Nobel Prize in literature was even awarded for a novel of the life of a bridge (Andric 1977). There are many thousands of bridges in the world, and there have been thousands more in the past. It is hardly surprising that among these bridges there were partial failures, or total collapses. But most of these occurred in the past or in somebody else's country, and so the general public is not worried about routinely crossing bridges. However, if the failure happens almost next door (the I35W Bridge is a 20 minute walk from my office at the University of Minnesota), then we take note! This is especially so if the failure comes out of the blue on a clear warm summer day on a bridge that has done its job for half a Century. Drivers on the Minneapolis Bridge often were not even aware they were on a bridge. A minute or so, and they were across. No wonder that the citizens were upset when this permanent, solid, dependable, bridge decided to give up the ghost.

[1] Reprinted from The Bridge 38(2): 20-25 with permission of the National Academy of Engineering.

People often take great risks in their every-day life. Driving a car, crossing a street, smoking, overeating, getting married, etc. involve the taking of chances. These actions could possibly have dire consequences. However, it is the individuals who voluntarily put themselves into situations that could have a bad outcome. They are in the driver's seat! *The collapse of a bridge is not a voluntary risk.* It is not supposed to happen, and hardly anybody in the world will ever perish in a collapsing bridge. Pugsley (1966) in his pioneering book on the safety of structures relates that fighter pilots in WWII went up against the enemy even if there was a high chance of getting shot down, but these pilots would demand design changes if the structural failure rate was more than 5×10^5 per flying hour. So to the public the collapse of the Minneapolis Bridge is a very serious event that must somehow be explained.

2 HISTORICAL PERSPECTIVE

It is an almost unheard of event that a bridge that is judged to be sound, suddenly and totally disintegrates in a matter of seconds. This is rarer than a rare event, yet it does happen from time-to-time and the memory will last the lifetime of the generation that experienced it, if even remotely through reports of the media. One such event will be presented next, because it has many similarities to the Minneapolis bridge failure.

The example is the collapse of the Point Pleasant Bridge over the Ohio River on December 15, 1967 at Point Pleasant, West Virginia. My knowledge of this collapse comes from the final accident report of the National Transportation Safety Board (NTSB 1970) and from my brother Charles Galambos who worked on the investigation. This bridge was constructed in 1929 and had been in service continuously for 38 years. The structure suddenly collapsed during the evening rush hour. Forty-six persons died, nine were injured and scores of vehicles fell into the river. This unfortunate collapse was thus very akin to the event in Minneapolis on August 1, 2007.

The Point Pleasant Bridge was a novelty in its time, and few other such bridges were constructed during the lifetime of it, and none after its collapse. The sketch in Figure 1 was scanned from the NTSB report and it shows the general features: it is a hybrid between a truss bridge and a suspension bridge. In the middle of the central span the tension in the suspension bars is counteracted by the compression in the top chord of the truss. The two forces somewhat cancelled each other, to the economic advantage of this type of bridge. The suspension system consisted of sets of two eye bars connected by pins at the junction between adjacent segments. (see Figure 2, which was also scanned from the NTSB report). The material of the eye bars was a new tempered high-strength steel. Failure was initiated by the brittle fracture in the eye-bar material at the first joint to the north of the tower on the Ohio side in the suspension structure, as denoted in the sketch of Figure 1. The causes of the failure are given on p. 126 of the NTSB report, which is quoted next:

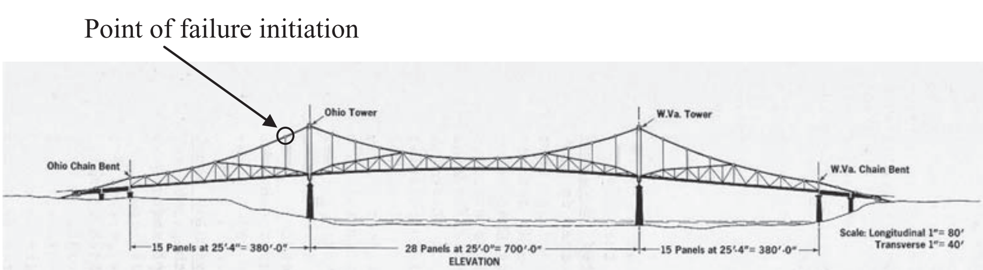


Figure 1. Sketch of the Point Pleasant Bridge (from NTSB 1970).

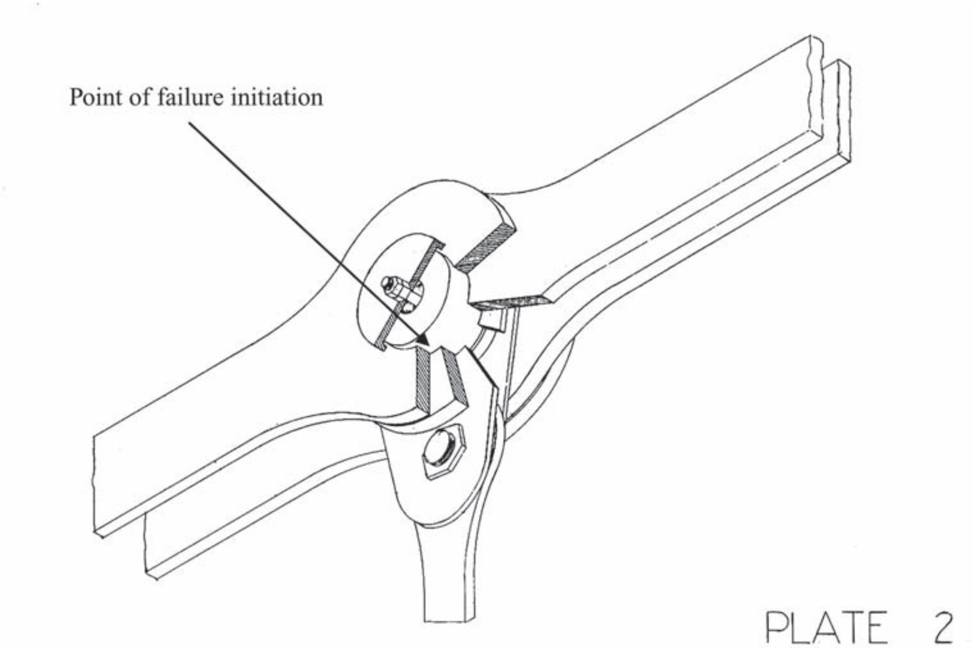


Figure 2. Sketch of the eyebar arrangement of the Point Pleasant Bridge (from NTSB 1970).

> "The Safety Board finds that the cause of the bridge collapse was the cleavage fracture of the lower limb of the eye of eyebar 330 at joint C13N of the North eyebar suspension chain in the Ohio sidespan. The fracture was caused by the development of a critical sized flaw over the 40 year life of the structure as the result of the joint action of stress corrosion and corrosion fatigue.
>
> Contributing causes are:
>
> 1. In 1927, when the bridge was designed, the phenomena of stress corrosion and corrosion fatigue were not known to occur in the classes of bridge material used under conditions of exposure normally encountered in rural areas.
> 2. The location of the flaw was inaccessible to visual inspection.
> 3. The flaw could not have been detected by any inspection method known in the state of the art today *(1970, added by author)* without disassembly of the eyebar joint."

This disaster was a "wake-up" call for the bridge engineering community, and the design, construction, inspection, and maintenance of bridges has radically changed as a result. Bi-annual inspections and material fracture toughness requirements were mandated, as well as other changes over the years, especially as regards the considerations of fatigue and brittle fracture.. The one change of most interest for this essay was the requirement that the bridge must be *robust*, that is, it should not totally collapse when a local joint or a member fails. In the jargon of bridge design standards, the system must not be prone to "progressive collapse". The forces that a failed part was designed to be carrying must be able to be rerouted to another path: the structure must be "redundant". Had the Point Pleasant Bridge been built with many eyebars in each chain link, as shown by the photo of the Budapest Chain Bridge (Figure 3) instead of just two, then it would still be in service today, with possibly fractured eyebars replaced as needed. The real cause of failure was thus not the fracture of the eyebar, but an error of judgment during design.

Figure 3. Eyebars of the Budapest Chain Bridge.

3 CAUSES OF BRIDGE FAILURES

A bridge is all structure. It does not have any hidden sources of strength as exist in buildings where walls, stairs and elevator shafts, slabs, etc. usually provide uncounted strength and stiffness to the structural system. Every member in a bridge is there for the structural purposes of strength and stiffness.

Most bridge failures occur during construction, when the structure is most vulnerable. Many of the problems of undesirable bridge behavior get ironed out during construction. Once the bridge is fully erected, one can confidently expect that the structure will last for its intended life span doing its intended job. There are, however, rare collapses during the service life of bridges. Fortunately most of these are partial collapses, and most are discovered in good time and repaired. Following is a short list of causes of bridge collapse among many others that could be cited:

- *Previously unimagined effects of natural forces*: One example is the partial collapse of one segment in the traffic lane of the San Francisco-Oakland Bay Bridge during the Loma Prieta Earthquake. Out of phase movements of supports as the seismic wave traveled along the bridge caused a span to be dropped. Similar failures occurred in freeway bridges during the San Fernando earthquake. As a result, designers now make sure there is an unbroken path for the flow of forces and that members and supports are solidly tied together.
- *Deliberate destruction in war*: Most of the bridges over the rivers of Central Europe were destroyed during the Second World War. While this was unfortunate at the time, it had a very beneficial effect on the future of bridge engineering, because the reconstruction brought a renaissance to the art of bridge building. Out of economic necessity new architectural forms evolved, such as plate girder bridges, cable-stayed bridges, prestressed concrete bridges, to mention only a few instances. Over the years since the middle of the last century this rejuvenation has moved from Europe to America, to Japan, and presently to China. An example of a lovely cable-stayed bridges is shown in Figure 4.

Figure 4. Cable-stayed bridge over the Savannah River.

- *Careless or accidental human acts*: One could cite here the destruction of the original Sunshine Skyway of Tampa, FL by the collision of a ship with one of its piers. As a consequence, piers in waterways are now surrounded by strong protecting barriers.
- *Decay by cracking or corrosion*: Often this signifies neglect of proper maintenance and inspection. One of the major assignments of bridge inspectors is to keep an eye on such deterioration. Repair for this malady is a large part of the maintenance budges of transportation authorities.
- *Fatigue and brittle fracture of structural members and connections*: This type of failure is frequently due to the misunderstanding of connection details or the use of wrong materials. While design practice knows how to design against such problems, cracking and corrosion are part of aging, and should be constantly watched for and repaired when discovered.
- *Unfamiliarity with new materials, details, and structural systems at the time of design*: This was one of the problems with the Point Pleasant Bridge discussed above.
- *Unseen hazards*: Here the most current concern is scour at base of bridge piers where the structure interfaces with the riverbed. This problem is judged presently to be the most common cause of bridge failure, and one of the active areas in bridge research.
- *Hazards that exist but we have no idea they are there*: These are the rarest and most dangerous causes of collapse. One of the most complex issues here is the extension of the science of structural engineering into situations for which there is no previous experience, where research should have preceded design, or where the designers were unaware of research done elsewhere. The classic cases of such catastrophes are the Failure of the Firth of Tay Bridge in Scotland on December 28, 1979 in a great windstorm (Prebble 1956), the collapse of the Quebec Bridge over the St. Lawrence River on August 29, 1907 during construction (The Government Board of Engineers, 1919), and the failure of the West Gate Bridge in Melbourne, Australia on October 15, 1970 (Royal Commission 1971). In the case of the Scottish bridge, the designers did not consider the effect of the wind forces on the design of the structure. One of the causes of the collapse of the Quebec Bridge was the neglect of a term in a differential equation of column strength. This effect was negligible for columns of solid cross section, but it was significant for the type of latticed open section columns used on this structure. The West Gate bridge failed for a variety of reasons unrelated to theory, but research conducted later by Professor Noel Murray of Monash University in Melbourne showed that the designers took the prevailing theory of the behavior of stiffened panels welded from thin steel plates beyond where it was applicable.

4 CONSEQUENCES OF BRIDGE DISASTERS

Bridges fail as a consequence of intended or unintended human acts or omissions, and due to unexpected natural disasters. Every bridge failure has provided society and its builders and engineers

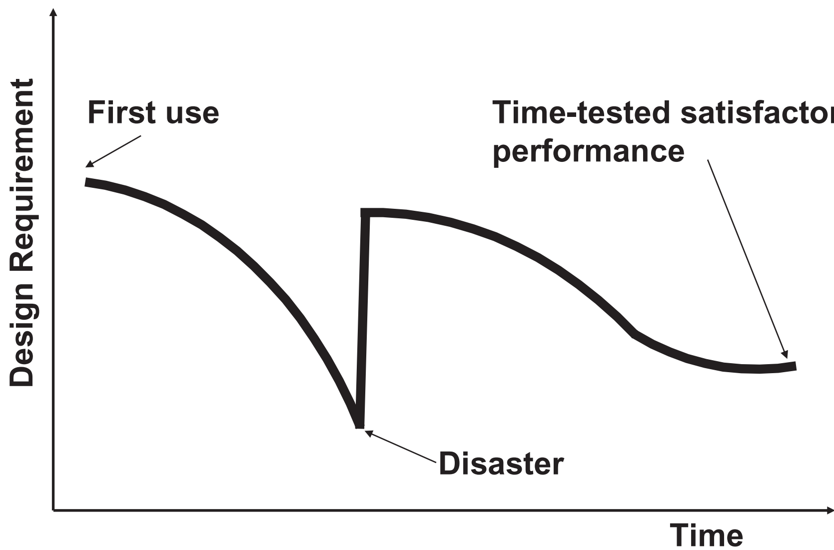


Figure 5. Cartoon of bridge design evolution.

the opportunity to do better next time, to learn, and apply this knowledge to the next design and to the retrofit of past designs. The process is illustrated in the cartoon in Figure 5.

When a new configuration of bridge geometry or a new combination of materials is used for a bridge, designers proceed conservatively. Continued success with the system encourages them to be less conservative, eventually passing into the range where they run out of the elbow room provided by the factor of safety. The result can be failure of the system. Society then reacts by returning to more conservatism in the next design, or, the type of bridge is judged to be unsuitable and the type vanishes. The cycle starts up again as new confidence is gained. This confidence is now also buttressed by research, and eventually a good balance is reached between safety and economy. Many examples could be cited. The classic case is the story of the stiffened roadway on suspension bridges: it became less and less stiff with each new bridge, until the Tacoma Narrows disaster. Research and an understanding of aerodynamic phenomena has now progressed to the stage where designers all over the world are spanning greater and greater suspension bridges. The story could be repeated many times over for other types of bridge structures. Current bridge art is based on the vast past experience of the designers, planners, fabricators, and builders, and on collaboration with research in academic, governmental, and industrial organizations and laboratories. While much has been learned from studying the causes of bridge failures and the research conducted after the catastrophe, it is far better to do the research first, and then dispense with the bridge collapse altogether.

5 BACK TO THE QUESTIONS

If we have the above touted experience with science, theory, design, and construction techniques, as well as thoroughly researched studies of all past failures, why did the bridge in Minneapolis fall down? This particular bridge was watched over by inspectors and engineers from the Minnesota Department of Transportation, thorough examinations were made frequently by a consulting firm, and graduate students under the guidance of structural engineering professors at the University of Minnesota studied the bridge several times in the last decade. To use a medical analogy: the patient died of unknown causes in the hospital while medical specialist were standing around the bed and studying the symptoms. The precise causes of this collapse are still under investigation by NTSB, and no definitive conclusions have been published at this time (April 2008).

The following scenario is my own guess as to what could have happened. It seems now (April 2008) that all the investigators and inspectors who checked the health of the bridge prior to collapse had somehow missed the fatal points of weakness. A few days after the disaster it was clear to those making preliminary examinations of the wreckage that it was one particular node, repeated symmetrically in four places in the trusses of the center span, had insufficiently thick "gusset" plates. These plates are riveted to the members that enter the joint, thus holding them together to form a node in the two trusses. These plates were reported by NTSB to be half as thick as they should have been. The deck of the bridge was being replaced at the time of collapse. There was a lot of construction material and equipment on the lanes where the deck replacement was taking place. The resulting extra force on the four presumably half strength joints could have caused one or more of the gusset plates to fracture, yield or/and to buckle. While the structure had some redundancy, there was probably no time for redistribution of forces to take place, or there was a loss of enough strength in the critical joints to transform the center span into an unstable mechanism and so the collapse occurred. The suddenness of the collapse can be explained by the fact that gravity acts very fast. I recall other instances where the removal of one member in a statically determinate truss in a collision between tractor and bridge caused an almost instantaneous collapse. I also remember recently slipping on an icy sidewalk and immediately finding myself on the ground. The suddenness is thus no mystery, and the cause of the collapse is simply explained by the loss of stability because of overload and under-strength. Gravity took care of the rest! The NTSB report will surely have specific details and scenarios for possible additional contributors to failure.

What can we learn from this collapse and will there be other bridge collapses in the future? There surely will be other failures in the future, as long as there are humans in the world. However, these events will be rarer now because of the lessons learned. The expected recommendations by NTSB in its final report will reduce this tiny probability even more, but whatever we do, there is always a remote chance of missing something.

What can we do better, or in addition to, what is being done now in order to reduce the risk of bridge collapse? Following are some suggestions:

- Older bridges and larger bridges should be kept under observation by experienced bridge engineers who through the years have gained acquaintance with the set of bridges under their supervision. Original designs of these bridges should be periodically examined and reanalyzed with the current state-of-the-art computer methods and against the current design codes.
- The design calculations, drawings and contract documents of new bridges should be reviewed carefully by experienced bridge engineers who did not perform the original design. The bridge documents should also be "peer reviewed" by an agency not associated with the original design or the bridge authority who own the structure.
- Robustness and redundancy should be incorporated in the bridge design as a matter of course to counteract unexpected and unimagined hazards, especially the chance of progressive collapse.
- The players in the design and construction teams should convene a "hazard scenario" exercise to discover possible problems prior to the start of construction.
- There are available many devices and instruments for monitoring critical parts of bridges, such as accelerators to discover changes in the dynamic signature, strain gages and deflection gages, and many types of cameras. Many other, even exotic, high technology devices could be employed. Data from these instruments then can be electronically transmitted, stored and finally evaluated by inspectors and experienced engineers. We will see more of these monitoring schemes installed as a consequence of the Minnesota bridge collapse. However, we must keep in mind that it will take sustained discipline to store and evaluate the data for the rest of the life of the bridge, when it is highly likely that for the large majority of structures there will be no noteworthy event ever to be observed.

I am sure that these and other measures are now being implemented. Instructions from government agencies to bridge owners are now (April 2008) being formulated.

6 CONCLUSIONS

Will these and the not yet invented precautions entirely prevent future bridge failures? There is always the human element: we make mistakes and we miss things, but, we also have the resolve to learn, think and act. There is no absolute safety in bridge performance. However, the bridge engineers are taking every possible measure so that a collapse like the Minneapolis Bridge remains indeed a very, very rare event.

REFERENCES

Andric, I., 1977, *The Bridge on the Drina*, The University of Chicago, IL.

National Transportation Safety Board, 1970, *Collapse of US 35 Highway Bridge, Point Pleasant, West Virginia, December 15, 1967*, Washington, DC, Report Number: NTSB-HAR-71-1.

Petroski, H., 1995, *Engineers of Dreams*, Alfred A. Knopf, New York.

Prebble, J., 1956, *The High Girders*, Secker and Warburg, London.

Pugsley, A., 1966, *The Safety of Structures*, Edward Arnold Publishers (LTD), London.

Royal Commission, 1971, *Report of Royal Commission into the Failure of Westgate Bridge*, C.H. Rixon, Government Printer, Melbourne, Victoria, Australia.

The Government Board of Engineers, 1919, *The Quebec Bridge Over the St. Lawrence River,* Printed by order of the Governor-General in Council, Canada.

Chapter 2

LRFD versus ASD, the differences between the two standards for retaining wall and abutment design

S. Esposito
Parsons Brinckerhoff, Princeton, New Jersey, USA

H. Najm
Department of Civil and Environmental Engineering, Rutgers University, New Brunswick, New Jersey, USA

ABSTRACT: In October of 2007 the AASHTO LRFD Bridge Design Specification became the mandatory design code for highway structures in the United States. This has introduced a number of changes in the design and analysis procedures that engineers have to follow. This paper takes an in-depth look at the differences between the Standard Specifications and the LRFD specification and how they apply to earth retaining structures. Calculations for several examples were carried out to examine the effect of the new design standards on structural proportioning and cost. For structures 15' and taller, the difference between the LRFD and the Standard Specifications was minimal. However, designing shorter walls using the LRFD code can result in an increase in footing length of about 30% and an increase in cost of about 10%. While these cost impacts to a structure can be significant, the new code provides a safe and reliable method of design which is of paramount importance to engineers and the public.

1 INTRODUCTION

The AASHTO LRFD (Load and Resistance Factor Design) Bridge Design Specification has recently become the mandatory design code for highway structures in the United States. As a result all engineering design calculations must conform to these standards. Many elements of bridge design have adopted the LRFD method from the start while others have lagged behind. Retaining walls and bridge abutments are two structural elements that have typically been slow to make the transition to LRFD design. These two types of structures behave similarly. They both resist earth pressure loads, make use of vertical wall elements to retain soil and rely on footings to add stability to the structure.

The resistance to make the switch for these structures has been so great that it has become common for engineers to submit calculations utilizing both of the design methods; stability related calculations would typically conform to the AASHTO Standard Specifications (which utilizes Allowable Stress Design) whereas structural capacity and reinforcing calculations would use the LRFD method. Now the LRFD must be used universally for any state funded projects. As a result, many engineers must catch up to learn the new method and execute LRFD design as required.

This paper examines the differences between the two design standards and how they affect earth retaining structures. Conclusions applicable to typical designs were drawn based upon calculations which consider the design standards independently. These examples elucidate the effect of the new design standard on these structures.

2 CHANGES IN DESIGN STANDARDS

The first Standard Specification for Highway Bridges was introduced in 1931 using allowable stress design (ASD). Since then interims have been added to the specifications regularly. In 1977 Load Factor Design was introduced to the specification. This addition was eventually viewed as patchwork for the existing code and it became apparent that a separate standard would be required to advance the design method. In 1993 Load and Resistance Factor Design (LRFD) was adopted by the American Association of State Highway and Transportation Officials (AASHTO). In 1994 the first edition of the LRFD code was published as an optional specification. The latest Standard Specification was published in 2002 and represented the 17th edition of the code. During the same time period the LRFD code was updated and in 2007 the 4th edition of the LRFD code replaced the Standard Specifications. The Federal Highway Authority (FHWA) and state agencies had agreed that beginning on October 1, 2007 all new bridge designs must conform to the latest LRFD design code.

Under the ASD Specifications all loads were considered equally and as such a comfortable 'safety factor' was established by maintaining stress levels below an acceptable level. Therefore the safety of a structure designed using ASD was built into the resistance of the structure as opposed to the applied loads.

The LRFD is a probability based approach as the design considers the variability of structural member resistance in addition to the unpredictability of the applied loads. For example, live loads are considered to be much more variable than dead loads and as such they have significantly larger load factors. In the LRFD the factored (reduced) resistance or capacity of a structure must be greater than the factored load effects. As a result the LRFD is considered to create a more uniform margin of safety through a large range of structure types and materials.

Since the LRFD utilizes a probability design, there is a wide range of load types which are considered separately by the code. This allows the different loads to have significantly different load factors dependent upon their variability and the loading condition, or limit state, that is being considered. There are more than 25 load categories in the LRFD and the magnitude of their load factors vary between limit states. This is due to the probability of certain loads to exist during a specific load condition. For example the likelihood of heavy live loads during an extreme event such as an earthquake is relatively low. As a result, the load factor for live loads is significantly reduced when earthquake loads are analyzed.

To complicate matters further, certain load factors can vary within the same limit state. The code refers to this category of loads as permanent loads. The variation of these load factors correspond to a maximum and minimum value based on the application of the load. If the force effect of the load is detrimental to the stability or the capacity of a structure it receives the maximum load factor. However, if the force effect is beneficial to the structure then it receives the minimum value. The load designation and definitions for these permanent loads are as follows:

- DC – dead load of structural components and nonstructural attachments
- DD – downdrag
- DW – dead load of wearing surfaces and utilities
- EH – horizontal earth pressure load
- EL – locked in force effects resulting from construction
- ES – earth surcharge load
- EV – vertical pressure from dead load of earth fill

Five of the loads listed above are critical to retaining wall and abutment design. As such, it is plain to see that this aspect of the LRFD has a profound effect on earth retaining structures.

Consider the following example for retaining wall stability analysis. The maximum load factor for horizontal earth pressure must be used as this creates a horizontal thrust on the wall which reduces stability. Note that this force effect is a function of the soil weight and height. When computing the weight of the soil resting on the footing of the wall the minimum load factor must be used as this load adds to the weight and resistance of the structure. The AASHTO LRFD provides the following illustrations to help engineers understand the dynamic effect for these types of loads.

As one would imagine, this leads to a complicated analysis of the loads imposed on such a structure and their application in the various limit states. In general, engineers prefer calculations to be simple and intuitive. The load factors in the LRFD are perceived to be somewhat confusing. Many engineers consider the load due to horizontal earth pressure to be very predictable and feel that it should not include such a high load factor. Furthermore reducing the resisting load created

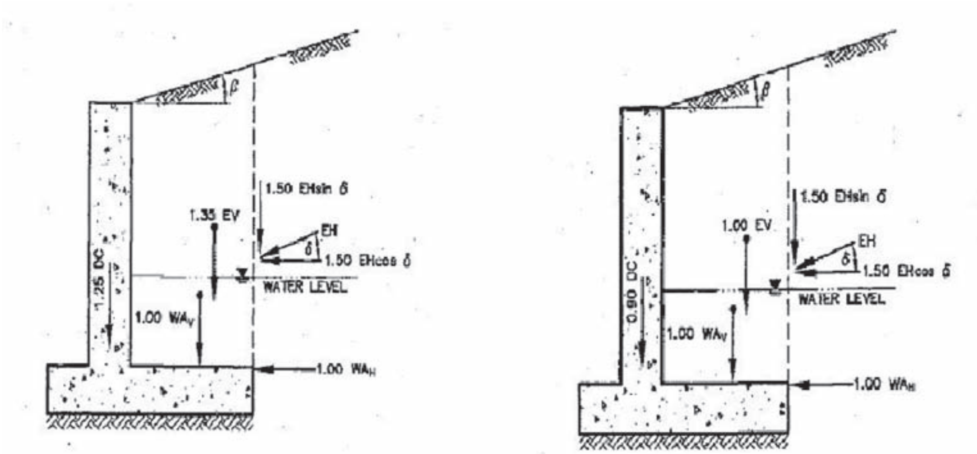


Figure 1. Left illustration shows maximum load factors required for bearing pressure calculations. Right illustration shows application of load factors for wall stability analyses. (AASHTO LRFD Specifications, 2007).

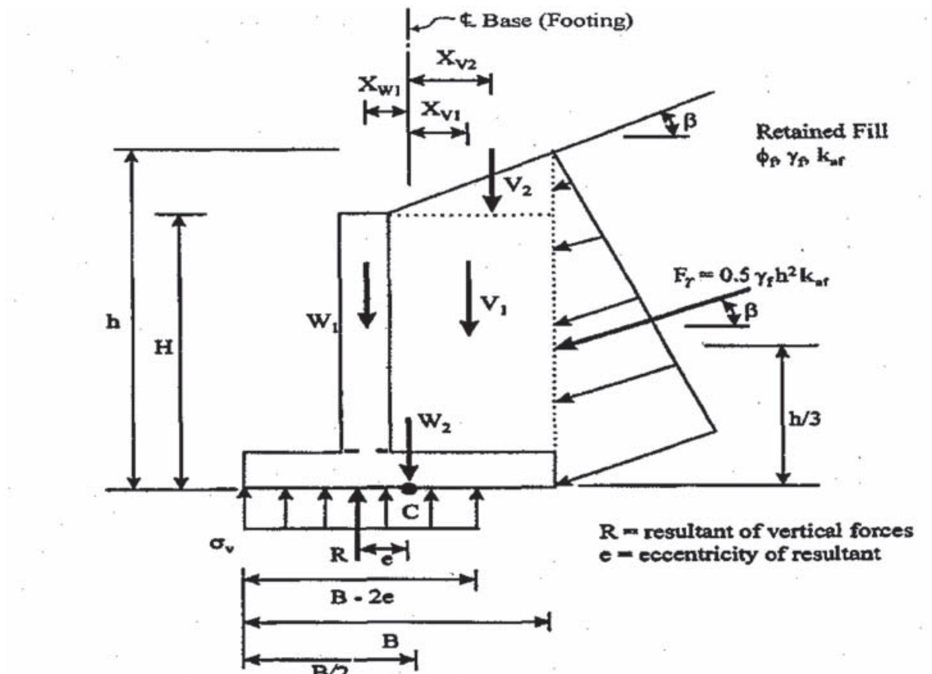
by the self weight of a structure below its nominal value (load factor = 0.9 for DC as shown in Figure 1) is another change for seasoned engineers.

3 KEY DIFFERENCES IN WALL DESIGN CHECKS

In addition to the method of computing the load effects on earth retaining structures, there are a number of other significant changes to the stability analyses of these structures in the LRFD. The most significant change in the stability evaluation is the elimination of the overturning check. This has been one of the basic stability checks under the ASD. The LRFD considers this check to be unnecessary as long as two other constraints are met. The first is that the bearing pressure under a footing cannot be less than zero, which would create an uplift force on the structure. This was also a requirement of the ASD. The second condition requires that the location of the resultant of vertical forces acting on the structure, known as the eccentricity of the loads, must remain within the allowable limits. This leads us to another change brought about in LRFD analysis. While an eccentricity check was required in ASD, the allowable limit has been modified. In LRFD the eccentricity is permitted to fall within the middle third of the footing, whereas ASD is more restrictive, requiring the eccentricity to remain within the middle quarter of the footing. This change was made due to the effect of the load factors used in the LRFD. Increasing this limit was necessary to avoid excessively large structures. As discussed previously, the goal of LRFD is not to add more safety to structures, but to establish a more uniform level of safety. There is sound justification for eliminating the overturning analysis because it should not control under the new constraints of the LRFD. Nonetheless, removing such a basic design check was a marked deviation from the ASD approach.

Another significant change in the way structures are analyzed is the method used to compute their sliding resistance. In ASD a table was provided that the engineer could use to select a friction coefficient based on the type of material a foundation was bearing on. Now, the friction coefficient is directly related to the internal friction angle of the foundation material. This is certainly a more intuitive and less subjective method. In addition, a resistance factor must be applied to the new friction coefficient to reduce the stabilizing forces. This strength reduction coupled with the increased driving forces due to factoring loads often makes the sliding check a controlling design consideration.

The assumption used to calculate the bearing pressure distribution for footings on soil has also changed. For footings bearing on soil or rock the ASD considers a trapezoidal shape with the minimum bearing pressure at the heel of the footing and the maximum at the toe. This assumption



Summing moments about Point C:

$$e = \frac{(F_T \cos\beta)h/3 - (F_T \sin\beta)B/2 - V_1 X_{V_1} - V_2 X_{V_2} + W_1 X_{W1}}{V_1 + V_2 + W_1 + W_2 + F_T \sin\beta}$$

Figure 2. LRFD bearing pressure diagram for footings on soil. (AASHTO LRFD Specifications, 2007).

only remains the same under the LRFD design of footings bearing on rock. When footings bear on soil the bearing stress is assumed to be rectangular over an effective length. The effective length is equal to the length of the footing minus two times the eccentricity of loading. This change was necessary to eliminate the presence of uplift at the heel that would often occur under LRFD design, but not ASD design, as a result of the increased loading due to load factors. Again, this change was made to avoid a major difference in the proportioning of the structure across the two design codes. Figure 2 below shows the assumptions adopted by the LRFD design manual for footings bearing on soil.

Another modification to resistance is evident in bearing capacity computations. Under the LRFD bearing capacity now has a mandatory resistance factor used to decrease the allowable load transferred from a footing to the underlying soil. However, bearing capacity calculations for LRFD typically result in much larger values, therefore such a resistance factor is necessary to maintain the desired level of safety against the failure of foundations.

4 DIFFERENCES IN LOADS

It is also important to highlight the areas where major changes have been brought about from the load effects themselves. The first load we will consider is the live load surcharge. This load is used to simulate the effect of trucks passing near a structure. The load is applied through an equivalent height of soil that is expected to create the same force effects. Previously, ASD applied a surcharge load equal to 2 feet of soil under all design conditions. The LRFD has replaced this load with a value which varies depending on the height of the structure and its orientation relative to vehicular traffic. For short walls, a surcharge load as much as 5 feet of soil must be applied. Intermediate walls will have a force effect near 3.5 feet of soil. When the wall height is equal to or greater than 20 feet the height of soil applied as the surcharge load is 2 feet.

The vehicular impact load, perhaps one of the most challenging load cases for wall designers, even in the ASD specs, has become more difficult to satisfy in the LRFD. In ASD design, an equivalent static lateral load of 10 kips is applied to a structure that could potentially be exposed to a collision. Without specific guidance provided, a rational was developed by engineers to resist this load. It had been typical practice to distribute this load over a structure's expansion joint spacing, considering the unit to act as a whole. In many cases this would relate to a 30 foot length, resulting in a distributed load of 0.33 kips per foot. Another typical application was to distribute the force over a length equal to the height of the structure, representing a 1:1 failure plane from the point of impact. Either of these two methods were used at the designer's discretion to obtain the desired level of safety for the situation at hand.

In the LRFD, this load has been significantly increased. The test-level 4 crash rating, which AASHTO considers applicable to the majority of highway bridges with moderate to high speed limits, will be considered. The lateral load for this test level is 54 kips, more than 5 times the load used in the ASD. Furthermore, the LRFD requires this load to be distributed over 3.5 feet at the parapet or barrier curb. This results in a loading in excess of 15 kips per foot. When calculating the overall stability of a structure, it is still typical to use the distribution methods which were adopted in ASD design. However, it is very difficult to dissipate this load when considering the internal or structural capacity. Since the LRFD outlines a specific distribution for the load, engineers must ensure that a structure has enough capacity to avoid a local failure at the point of impact. While this change may not have a profound effect on the overall dimensions of a structure when coupled with certain engineering rationale to distribute the load, the reinforcement for the structure and the anchorage will undoubtedly be significantly greater compared to the ASD requirements.

5 THE NEED FOR DESIGN COMPUTATIONS

Now that many of the differences between the two standards have been addressed, it is necessary to see the effects they have on actual designs. To obtain a truly accurate, detailed and tangible picture of their effect it is necessary to develop design calculations that will put these concepts to the test. Although the key differences between the two codes were highlighted previously, the LRFD still utilizes a number of the same principles inherent in the ASD. Yet with all of the load combinations, types of loads and load factors, the procedures for design have certainly been expanded upon. It has never been the intent for the LRFD to have a significant impact on the final dimensioning or performance of a structure. Service load design was effective and proven to be a valuable system for design. The LRFD attempts to lead engineers to a similar end product as the ASD, however, the primary goal is to offer a more uniform factor of safety for a variety of conditions and structures.

Therefore, the following questions arise. Does LRFD produce similar designs for earth retaining systems when compared to the ASD? Does LRFD succeed in providing a uniform safety factor over a range of conditions? And finally, are the changes to the new design standard easy to comprehend or likely to cause problems for engineers?

To answer these questions a complete and thorough set of design calculations have been developed for both design standards for use in conjunction with this paper to gain a true understanding of the changes brought forth in LRFD. A spreadsheet was developed which fully utilizes and meets all the requirements of the current ASD code and the new LRFD standard for conventional semi-gravity cast-in-place concrete retaining walls. It includes loading calculations for each of the AASHTO load combinations relating to wall design; even those which are not expected to control the design must be considered. The spreadsheet was used to carry out a complete design using both methods to allow for an accurate comparison of the two design standards. Conclusions about the cost effectiveness and design effort of the LRFD design standard versus the standard specifications were drawn along with discussions of strengths and weaknesses between the two design methods.

6 SUMMARY OF RESULTS

Three different structures have been analyzed in this study; two retaining walls and one abutment. The choice was made to consider the first design case under two separate load conditions. The first condition, design case 1A, considers a moderate retaining wall height with no impact force applied to the wall. The second condition, design case 1B, adds the vehicle collision force to the structure analyzed in case 1A to study the impact this load has on design. Design case 2 studies a relatively short wall height. Design case 3 depicts a typical abutment design.

Emphasis for comparison below is based on the required footing dimensions of the structure across the two design methods. This has the largest effect on cost considering the concrete volume for the stem of the wall will not vary; it is solely dependent on the height of the wall. Changing the dimensions of the vertical wall element does not impact the stability performance of a structure; minimum thicknesses specified by the owner are typically used by engineers. When a wall fails to meet the stability requirements it is always the footing of a wall that is made larger to increase the resisting loads. As the length of a footing increases, more soil rests on the structure adding weight and resistance to the system. In addition, less reinforcement is expected to be required for LRFD but the cost implications will be minimal.

For each case examined, the critical design parameters are summarized in the tables below. The retaining wall or abutment height includes the thickness of the footing and the water table height is measured from the bottom of the footing.

6.1 *Design case 1A*

Design condition 1A proved to be a case where LRFD and ASD design methods were similar in required geometric properties for the retaining wall. The required footing thickness was equal under both codes. The minimum footing length for LRFD was found to be somewhat larger, but a difference of 9” is not very significant. As predicted, the reinforcement required for LRFD is less than ASD. The reinforcement at the bottom of the stem will require #8 bars with a spacing of 9” for ASD; the spacing can be increased to 12” for LRFD.

The two design methods produce similar results for this design case and the effects on cost are minimal. A cost comparison analysis will be presented at the end of design case 1B.

6.2 *Design case 1B*

The design for case 1B did not vary much in terms of stability and geometric proportioning when compared with case 1A. Therefore, we can conclude for a wall with a moderate height of retained earth, impact forces do not have a profound effect on the general stability of a retaining wall. The only change in the footing length due to the collision force is an additional 3” necessary to satisfy the LRFD. The eccentricity of load was the controlling factor for the design, primarily due to the impact force.

With the inclusion of the collision force in case 1B, the difference in footing lengths between the two design methods is now 1’-0”. This could be considered as a substantial difference, and it is certainly recognizable, however the cost implications are still small. We will consider the impact on concrete cost of the two design methods for a 30’-0” long wall segment. Two separate concrete quantities and costs are computed for wall design; one for footings and one for the stem portions. Typical construction costs for these items are currently near $600/cubic yard for footings and $900/cubic yard for wall stems. The primary reason for the difference in costs is due to the formwork and additional labor required for the stem construction. The unit prices used have been obtained from the bid prices of the most recent construction projects awarded by the NJDOT.

This results in a cost difference of $1,500 for case 1A between the two design codes. This correlates to a 2.5% increase in cost when transitioning from ASD to LRFD. Such a change in cost is generally not considered to be of much importance for highway construction projects.

Therefore the total difference in concrete cost for design case 1B is $2,000 which adds about 3.3% to the cost of construction. This is still an insignificant addition to the cost of the wall.

Table 1. Design case 1A parameters.

Design parameter	LRFD	Standard specification
Retaining wall height	20’-0”	20’-0”
Water table height	4’-0”	4’-0”
Impact force	n/a	n/a
Surcharge load	2’-0”	2’-0”
Minimum footing thickness	3’-0”	3’-0”
Minimum footing length	12’-3”	11’-6”

Table 2. Design case 1B parameters.

Design parameter	LRFD	Standard specification
Retaining wall height	20’-0”	20’-0”
Water table height	4’-0”	4’-0”
Impact force	54 kips/20 ft	10 kips/20 ft
Surcharge load	2’-0”	2’-0”
Minimum footing thickness	3’-0”	3’-0”
Minimum footing length	12’-6”	11’-6”

Table 3. Standard specification design cost estimate—case 1A and case 1B.

	Minimum	Maximum	Average
Stem width	1.50 ft	2.83 ft	2.165 ft
Stem height	17.00 ft		
Wall length	30.00 ft		
Stem volume	1104.15 cf		
Stem volume	40.89 cy	× $900/cy	= $36,805
Footing length	11.50 ft	3.00 ft, footing thickness	
Toe length	2.75 ft	0.50 ft, additional toe thickness	
Footing volume	1076.25 cf		
Footing volume	39.86 cy	× $600/cy	= $23,916
Total cost			= $60,721

Table 4. LRFD design cost estimate—case 1A.

	Minimum	Maximum	Average
Stem width	1.50 ft	2.83 ft	2.165 ft
Stem height	17.00 ft		
Wall length	30.00 ft		
Stem volume	1104.15 cf		
Stem volume	40.89 cy	× $900/cy	= $36,805
Footing length	12.25 ft	3.00 ft, footing thickness	
Toe length	2.75 ft	0.50 ft, additional toe thickness	
Footing volume	1143.75 cf		
Footing volume	42.36 cy	× $600/cy	= $25,416
Total cost			= $62,221

Table 5. LRFD design cost estimate—case 1B.

	Minimum	Maximum	Average
Stem width	1.50 ft	2.83 ft	2.165 ft
Stem height	17.00 ft		
Wall length	30.00 ft		
Stem volume	1104.15 cf		
Stem volume	40.89 cy	× $900/cy	= $36,805
Footing length	12.50 ft	3.00 ft, footing thickness	
Toe length	2.75 ft	0.50 ft, additional toe thickness	
Footing volume	1166.25 cf		
Footing volume	43.19 cy	× $600/cy	= $25,916
Total cost			= $62,721

Table 6. Design case 2 parameters.

Design parameter	LRFD	Standard specification
Retaining wall height	10’-0”	10’-0”
Water table height	2’-3”	2’-3”
Impact force	n/a	n/a
Surcharge load	3’-6”	2’-0”
Minimum footing thickness	2’-0”	2’-0”
Minimum footing length	7’-9”	6’-0”

The internal stability, or reinforcement requirements, of the stem did increase significantly for the LRFD method due to the large impact force. As discussed previously, the impact force is more than 5 times greater for LRFD than ASD. Under a crash loading the reinforcement required for LRFD is more than double what is needed for ASD. LRFD requires No. 8 bars at 12” at the top of the wall; ASD only requires No. 5 bars at 9”. The wall stem height is 17’-0”, the difference in bar weights is about 1.5 lb/ft. Therefore, assuming the 30’ wall length would result in 17’ × 30 bars × 1.5 lb/ft = 638 lb × $2/lb = $1,530. Again this is a relatively small change in cost compared to the total cost of the wall.

6.3 *Design case 2*

Design condition 2 represents a major difference in design between LRFD and ASD. The length of footing required for LRFD is nearly 30% greater than what is needed to satisfy ASD. Furthermore, the footing length for LRFD is almost 80% of the height of the wall, which would be a painful sight for any engineer and definitely create an area of concern for anybody reviewing such a design. The cause of this large footing discrepancy is the surcharge load, which is equivalent to 3’-6” of soil under the LRFD code for a wall height equal to 10’-0”. ASD utilizes 2’-0” for all load cases, and it is not until you reach a wall height of 20’-0” that LRFD allows use of the more accepted ASD value (see Figure 3 below). To make matters worse, the maximum LRFD load factor for live load surcharge is 1.75. After the load factor is applied, the equivalent height of soil for the LRFD version of this design case surpasses 6’-0”.

This change of the footing has a major impact on the cost of this relatively small retaining wall. The reinforcement required for the two methods is identical and therefore has no impact on the cost of the wall. Below we will take a look at the impact on concrete cost of the two design methods for a 30’-0” wall segment.

Retaining Wall Height (ft.)	h_{eq} (ft.) Distance from wall backface to edge of traffic	
	0.0 ft.	1.0 ft. or Further
5.0	5.0	2.0
10.0	3.5	2.0
≥20.0	2.0	2.0

Abutment Height (ft.)	h_{eq} (ft.)
5.0	4.0
10.0	3.0
≥20.0	2.0

Figure 3. LRFD surcharge loads in equivalent height of soil (LRFD Tables 3.11.6.4-1 and 3.11.6.4-2). (AASHTO LRFD Specifications, 2007).

Table 7. Standard specification design cost estimate—case 2.

	Minimum	Maximum	Average
Stem width	1.50 ft	1.50 ft	1.50 ft
Stem height	8.00 ft		
Wall length	30.00 ft		
Stem volume	360.00 cf		
Stem volume	13.33 cy	× $900/cy	= $12,000
Footing length	6.00 ft	2.00 ft, footing thickness	
Toe length	1.25 ft	0.50 ft, additional toe thickness	
Footing volume	378.75 cf		
Footing volume	14.03 cy	× $600/cy	= $8,416
Total cost			= $20,416

Table 8. LRFD design cost estimate—case 2.

	Minimum	Maximum	Average
Stem width	1.50 ft	1.50 ft	1.50 ft
Stem height	8.00 ft		
Wall length	30.00 ft		
Stem volume	360.00 cf		
Stem volume	13.33 cy	× $900/cy	= $12,000
Footing length	7.75 ft	2.00 ft, footing thickness	
Toe length	1.25 ft	0.50 ft, additional toe thickness	
Footing volume	483.75 cf		
Footing volume	17.92 cy	× $600/cy	= $10,750
Total cost			= $22,750

The difference in cost between the two designs is roughly $2,200. Now this is not a huge number in itself, relatively speaking though, the impact is very significant. This represents more than an 11% change in cost for the structure and if there is a long length of such a retaining wall on a project the difference can become much more of an issue.

6.3.1 *Discussion of the effects of vehicular impact forces on case 2*

It is not within the scope of this paper to go into a detailed analysis of collision forces; this is due to the highly subjective application of these loads. As a result, calculations for case 2 which consider crash loadings will be omitted from a detailed discussion, but will be briefly mentioned here. One would expect the effects of the collision force to be much greater for a short wall which has a small amount of soil weight above the footing to keep the wall stable. This, in fact, is true. If the joint spacing, length of the wall and the engineer's judgment will allow the designer to distribute the impact loads over a length of 30’-0” the change in the required footing length is 6” for both LRFD and ASD which is not particularly overwhelming. Although, with LRFD’s footing already on the long side, the addition of the impact will begin to give the wall a foolish appearance compared to what engineers typically see.

However, if the length of the wall is short or joints are spaced closely and the maximum justifiable distribution length is only 20’-0” the effects are drastic under the LRFD code. Such a design would result in a 9’-6” footing length for LRFD compared to 7’-3” for ASD. This means that under the ASD design code, one could design a footing for a 10’-0” high retaining wall with an impact load to be smaller than a design using LRFD under static conditions.

6.4 *Design case 3*

Design case 3 relates to abutment design. The stability of the abutment wall was checked without including any vertical loads from the superstructure of the bridge. Such large vertical loads would create resistance to sliding and overturning and including these loads could potentially lead to failure during construction or if the superstructure is ever removed from the abutment.

When checking the maximum bearing pressures, the maximum vertical loads transferred from the bridge are included. This will obviously create the critical design condition for the bearing check. The vertical bridge loads obtained for this example pertain to a 100’-0” single span, simply supported bridge with 10’-0” long approach slabs at each abutment.

Table 9. Design case 3 parameters.

Design parameter	LRFD	Standard specification
Abutment height	18’-0”	18’-0”
Water table height	4’-0”	4’-0”
Impact force	n/a	n/a
Surcharge load	2’-0”	2’-0”
Minimum footing thickness	2’-0”	2’-0”
Minimum footing length	8’-8”	8’-8”

Table 10. Standard specification and LRFD design cost estimate—case 3.

	Minimum	Maximum	Average
Stem width	4.00 ft	4.00 ft	4.00 ft
Stem height	12.00 ft		
Wall length	30.00 ft		
Stem volume	1440.00 cf		
Stem volume	53.33 cy	× $900/cy	= $48,000
Backwall width	1.50 ft	4.00 ft	2.75 ft
Backwall height	4.00 ft		
Backwall volume	330.00 cf		
Backwall volume	12.22 cy	× $900/cy	= $11,000
Footing length	8.67 ft	2.00 ft, footing thickness	
Toe length	2.00 ft	0.00 ft, additional toe thickness	
Footing volume	520.20 cf		
Footing volume	19.27 cy	× $600/cy	= $11,560
Total cost			= $70,560

For the design example considered, the required footing dimensions are identical across the two design standards. It is worth noting that the stability check controlled for LRFD design, whereas the allowable bearing pressure was the limiting design consideration for ASD.

The reinforcement requirements were also the same for the two design methods. This is due primarily to the dimensioning of the structure. The abutment wall section is very deep and therefore requires minimal reinforcing for either design case. Also, the length of the heel and toe for the footing are very short which prevents the internal moments from reaching any real significant levels. A cost breakdown for a 30'-0" section of this structure is shown below.

It can be seen that the cost is generally controlled by the stem portion of the abutment. This is due to the large width required for the bridge seat section which produces a substantial volume of concrete. The stem and backwall account for 84% of the cost in this design example. As a result, even if there were a difference in footing requirements for the abutment the cost impact will be small. For example, a 1'-0" or 12% increase in the length of the footing would only increase the cost of the structure by $1,350 which is less than 2% of the total cost.

7 CONCLUSIONS

There are a number of significant differences between the LRFD and the ASD design requirements for retaining walls and abutments. Prior to October 2007, the ASD has been implemented successfully for these structures and considered simple and intuitive. Nonetheless, engineers must now change their approach for the design of earth retaining structures.

In review of the design examples, the differences between the two design methods became evident. Arguably, the main difference is the complications introduced by the LRFD method. Under LRFD, many elements of design need to be examined more closely to determine which condition applies, such as determining the appropriate surcharge load to impose on a structure. Furthermore, the correct use of load combinations and load factors is more complex in the LRFD compared to the ASD. Interestingly enough, the strength of LRFD lies within the same concept. The fact that the calculations are much more complicated is due to the level of detailed research and analysis that has gone into the loads. The strength of the ASD, on the other hand, is its intuitive feel and use of stability checks which are trusted and well understood. While the changes presented by the LRFD are not very difficult to comprehend, it will take time for engineers to research and implement them into their analyses.

The findings discussed in this paper indicate that the LRFD will produce longer footings for short walls and similar footing lengths for wall heights greater than 15'-0" when compared to ASD. Furthermore, the total cost of construction is expected to vary slightly for walls taller than 15'-0" and to increase as much as 15% for shorter walls designed using the LRFD standard. This is primarily due to the increased surcharge loads imposed on shorter walls. These results can make it seem that LRFD design may provide excessive safety factors for short walls. However, it is difficult to dispute the justification provided by the code, see the commentary provided in section 3.11.6.4 of the LRFD code.

Based on the observed structure proportioning, designers who are confident with service load design should find comfort that the LRFD does not fall short with respect to sizing requirements for stability. However, when engineers are judged for their bottom line cost of a project the LRFD approach may not be favorable—especially since the ASD has been proven to be dependable over time. Overall, the LRFD provides a detailed, reliable and safe design method over a broad range of design cases. This is the primary concern of a designer whose main focus is on the continued safety of the public today and in the years to come.

ACKNOWLEDGEMENTS

We would like to thank Tom Fisher from Parsons Brinckerhoff for helping us move this project forward and providing valuable guidance.

REFERENCES

American Association of State Highway and Transportation Officials. AASHTO LRFD Bridge Design Specifications. Customary U.S. Units, 4th Edition, 2007.

American Association of State Highway and Transportation Officials. Standard Specifications for Highway Bridges. 17th Edition, 2002.

New Jersey Department of Transportation. Bridges and Structures Design Manual. Fourth Editions, 2002.

Chapter 3

Providing the best bridges for the best cost

R.A. Lawrie
Lawrie & Associates, LLC, Alexandria, VA, USA

ABSTRACT: Over recent years, the biggest thrust in bridge design has been economy of structure. "The bottom line" is the most important criteria. With the recent focus on the aging and ailing infrastructure, coupled with a shortage of funding, this has become a deepened mindset with transportation agencies, designers, and contractors who all believe that they need to do the most for the least cost. This paper presents the philosophy that we really need to provide the very best bridges for the best cost. Optimizing structural efficiency will provide aesthetics and economy. With a program including innovative thinking and an extensive development program for construction mechanization and equipment, we should be able to do better work for the same, or even a lesser cost. I believe it is our professional responsibility to advance our thinking and technology so that we can, once again, produce the finest bridges in the world.

1 THE NEED FOR EXCELLENCE

As Professional Engineers, I believe we all desire to produce the best bridges for the best cost. However, we live in a society that is ruled by the "bottom line," which appears to be at the cost of quality in many instances. The emphasis on structural efficiency and excellence is losing ground due to the emphasis on doing the most for the dollar. I have seen that this is not only true in the United States, but also in Europe and in Asia. Economics are certainly important; however, it seems that we should not exclude structural excellence when developing economics.

The goal of this proposal is to change our direction as an industry to accomplish structural excellence. We, in the United States, need to show the world how this can be done by working together as a team to bring this to fruition. This team should consist of government agencies, design engineers, architects, contractors and construction engineers. We all must work together to gain the common goal of structural excellence.

To focus our attention in the right direction, we should look at a number of bridges from the past that have brought prestige to the American bridge industry. Three samples are provided:

1. Verrazano Narrows Bridge (Figure 1), New York, with a 1300 m main span, was designed by Othmar Ammann. It was built in 1964.
2. Golden Gate Bridge (Figure 2), in San Francisco, CA, was designed by Joseph Strauss in 1937. This bridge has a main span of 1280-m.
3. The Eads Bridge (Figure 3), St. Louis, MO, was designed by James Eads. It has three main span arch spans, 156 m each. It also has a very innovative caisson foundation system. It was built in 1874.

As you look at these three bridges, you can see there is a certain magnificence of structure that exhibits itself within the structure itself. This magnificence is the very evidence of structural efficiency. This efficiency is the key to this proposal.

Considering the age of the Eads Bridge, even it demonstrates this efficiency with a unique combination of small structural elements used to provide a very significant, efficient span. The Verrazano Narrows Bridge and the Golden Gate Bridge are of a later vintage; they show their efficiency in the graceful way they carry the load from the long spans.

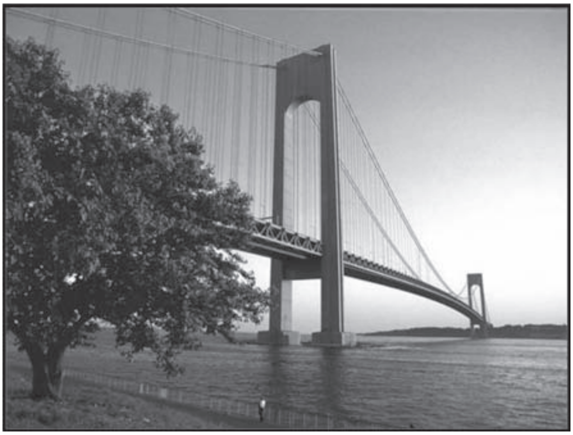


Figure 1. Verrazano Narrows Bridge, New York.

Figure 2. Golden Gate Bridge, San Francisco, California.

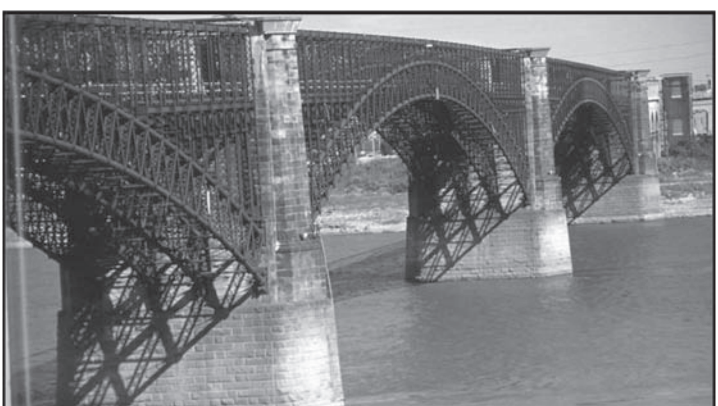


Figure 3. The Eads Bridge, St. Louis, Missouri.

Structural efficiency is important and is an element of design that needs to be brought back to the forefront in developing structural solutions for bridge structures. Most structure type selection today is based on historical cost records. We propose that a process be used where an efficient concept design is developed by the design team and, after the concept is set, the economics are developed by the complete design and construction team, which develops construction procedures, construction equipment, and design details in order to produce efficiency in construction as well as in structural design. We would further suggest that the special equipment be funded under a separate program that minimizes first time equipment costs and allows the equipment to be made available to other projects by any contractors who might use such.

Professor T.Y. Lin, one of the world's greatest bridge engineers, taught that all bridges should be made structurally efficient. He further taught that if they are structurally efficient, they will naturally have an aesthetic that is pleasing to the public eye. I believe that this is demonstrated in the three bridges above. He also taught that if a bridge is structurally efficient, it should be economical. An important feature is how to get the economics. I believe the economics should be developed, not based on historic cost, but rather by developing construction methods and equipment, along with design details, that will provide the necessary economics. This process is quite feasible and possible. It needs to be instituted into procurement rules.

2 EXAMPLES OF EFFORTS FOR EXCELLENCE

The possibilities for innovation that result in structural efficiency are demonstrated in the following projects. It needs to be noted that a number of these projects were developed under the supervision of Professor T.Y. Lin at T.Y. Lin International. I had the good fortune of working with the firm during those days.

There are many thousands of overpass structures throughout the United States. These structures generally are not very large, but being in such abundance, they result in a significant cost to the public. It is important that a quality product and a structurally efficient solution with economics be developed for this type of structure as a major improvement to the public investment of funds.

This is a structure that was likely laid out and developed using an historic cost analysis for this type of structure. We believe the structure is inefficient and consequently not necessarily cost effective. (Figure 4)

This I-95 overpass bridge is typical of that being constructed throughout the State of Maryland. (Figure 5) It appears to us that the State of Maryland is taking a bold step forward in developing efficiency for its overpass structures that also demonstrates a degree of economics. The fact that it is currently building a number of these structures does indicate that they have found a certain structural efficiency combined with aesthetics, resulting in economics. The gentle haunch, the thin girders close to the abutment, and the shape of the abutment offer considerable aesthetics

Figure 4. Typical "bottom line" overpass structure.

Figure 5. King Avenue over I-95, Maryland.

Figure 6. Strait of Gibraltar, Span – Morocco.

and efficiency to provide the public with a good cost effective solution that is based on structural efficiency.

The details of the proposed bridge crossing the Straits of Gibraltar (Figure 6), a 9-mile bridge connecting Spain and Morocco, were developed for the Gibraltar Commission by T.Y. Lin International under the leadership of Professor Lin. A point of efficiency that was developed by Professor Lin is that the spans were 5 km, resulting in a difficulty for the suspension span cables that they could barely carry their own weight. By introducing the rigid stay support, Professor Lin was able to shorten the effective length of the suspension cable to approximately 2/3 of its original value. This resulted in considerable and improved efficiency resulting in economics for the proposed crossing. This bridge is one of several alternates that have been considered for the Gibraltar crossing, not yet built.

The Ruck-A-Chucky Bridge (Figure 7) was to span the American River, located over the proposed lake for the Auburn Dam. Due to earthquake considerations with the Auburn Dam, this bridge was never built, but the design was done offering a definite number of structural efficiencies that are evident after some study. The cable arrangement is such that it creates a longitudinal thrust down the center line of the curved structure, offering a horizontal arch in axial compression offering very significant efficiency to the structure support system. At this site, the lake would have been 450 feet deep and 1200 feet wide. A straight bridge, with limited curvature at the ends, would require heavy approach cuts and tunnels, and a conventional curved bridge would require several high piers, a major expense due to the deep water, hillsides and the high seismicity of the site. Professor Lin's solution was a "hanging arc" design, which avoided piers and was on a tight, 1500-ft radius. Although a new concept at the time, it utilized well-established technology

Figure 7. Ruck-A-Chucky Bridge, CA.

Figure 8. Route 50 over Ohio River, WV – OH.

and methods. Although never built, it was one of the few bridges to receive awards from the architectural community.

This Route 50 crossing near Parkersburg (Figure 8), WV crosses two legs of the Ohio River and spans the historical Blennerhasset Island. The original environmental requirements dictated that the bridge must span the island since it was a historic site. This is a preliminary alternative developed to meet this requirement. The truss arch system was prestressed to a high level by jacking the bearings at the two piers over the span. This jacking prestressed the arch so that much of the dead load stress was negated due to prestressing the arch upward. This force was locked into the bearings on the piers and further locked in by a tie system in the arch itself which took the tensile force created by the jacking. This enabled a very slender arch which was very efficient and would have allowed considerable economics to be accomplished. Later in the project, the environmental requirements were changed and a somewhat different structure was designed and used for this site.

The High Bridge spans the Mississippi River in Minneapolis (Figure 9). It also has jacked bearings similar to the Route 50 bridge, but in addition, it also has a post-tensioned tie system offering redundancy in the tension tie that greatly reduced fatigue requirements and thereby, improved efficiency and offered additional economics.

Figure 9. High Bridge, MN.

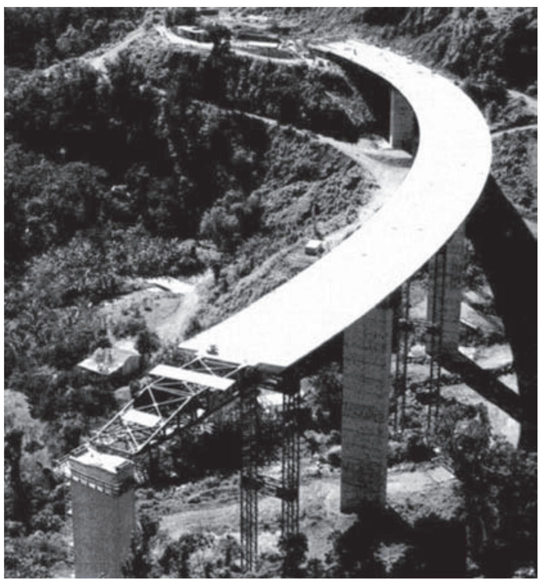


Figure 10. Cagunas River Bridge, Puerto Rico.

The Cagunas River Bridge (Figure 10) was launched from the far side of the photograph. It offered significant erection efficiencies that could not be accomplished by other means. This is a case where a construction method was proposed as a value engineering alternative by the contractor in an effort to provide a more efficient means of building the structure.

The Sacramento River Trail Bridge (Figure 11) demonstrates a method of constructing in a canyon or other in environments where work beneath the structure is very difficult. This type of structure simply takes a cable which is anchored at each end of the bridge. Then the precast superstructure is rolled on the cables into its proper location and post-tensioned into a rigid structure. It offers an efficient economical solution for crossing difficult site locations. Of note to this

Figure 11. Sacramento River Trail Bridge, CA.

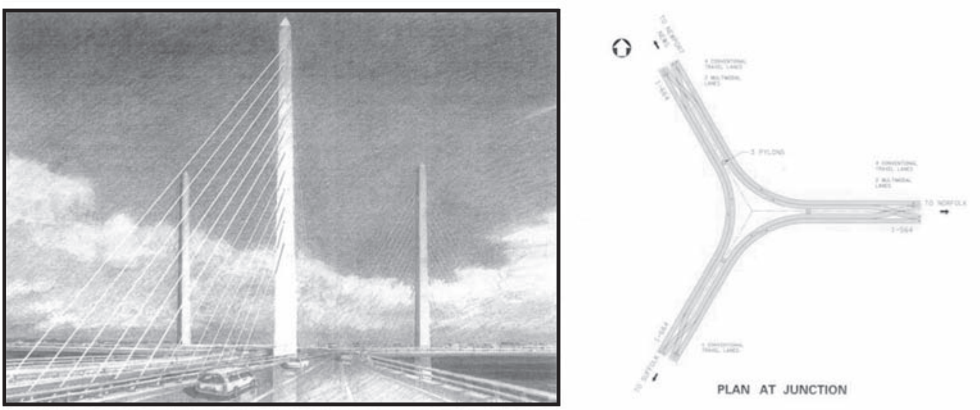


Figure 12. Third Crossing, VA.

structure, it is very slender in elevation. At a moderate distance away, it is almost a line crossing the river.

The Third Crossing (Figure 12) was a proposal developed by Lawrie & Associates that was presented to the State of Virginia and several contractors. It is yet to be funded; however, we believe it offers a solution to a major challenge. It is an interchange of two interstate routes, I-64 and I-664, located in the middle of the water at the mouth of the Chesapeake Bay on the James River. A traditional interchange at this site would consist of approximately 60 piers in the relatively deep water. This solution is comprised of three pylons, supporting the entire interchange in a pattern that is similar in all three legs of the interchange. Flyovers are used beyond the cable stayed support system to switch traffic to the proper location on the highway approaches.

3 CONCLUSIONS

Over the years we have seen many competitive challenges from nations abroad. One time, people used to ask, "Have you seen what they are dong in Europe?" Later, this was followed by "Have you seen what they are doing in Japan?" and then "Have you seen what they are doing in China?" We hope by means of adding a plan for structural efficiency in design with its resultant aesthetics and by developing efficient construction methods through a system supported by partnering and federal assistance to gain the comment from the worldwide profession "Have you seen what they are doing in the United States!"

Our philosophy must change. Rather than "Do the Most for the Least Cost," we must "Do the Best for the Best Cost!"

In order to provide the "Best Bridges for the Best Cost" we must make our goals for the 21st century to provide:

- Magnificence
- Splendor

for all the world to see!

REFERENCE

Graf, Bernhard, 2002. *Bridges that Changed the World*, Munich, Germany: Prestel.

Chapter 4

Taconic State Parkway Ramp 'X' Bridge project, Westchester, NY, USA

E.S. Jarosz & J. Kang
Gandhi Engineering, Inc., New York, USA

ABSTRACT: Constructed in 2007, the $15 million Ramp 'X' Bridge project was designed by Gandhi Engineering, Inc. as the final stage of the 5-stage Hawthorne-to-Pleasantville corridor improvement process initiated by the NYSDOT approximately 25 years ago. The project is located in a narrow north-south oriented ecologically sensitive segment of the Pocantico River, and is hemmed in by State Route 9A/100, the Taconic State Parkway, a wide bikepath, and Beech Hill Road, all of them framed by the surrounding Pocantico Hills, in the town of Mount Pleasant, Westchester County, NY, about 31 miles north of Times Square in New York City. The bridge connects southbound Route 9A to Southbound Taconic State Parkway. The central feature of this project is a sharply skewed flyover bridge tightly aligned on compound horizontal alignment involving a circular arc segment joined by a tangent. The irregular curvature of the bridge's steel framing required detailed analysis of stresses and deflections using finite element method. The average span of the curved girders is about 51 meters (167 ft) as the span follows a pronounced vertical curvature which further complicated the detailing and fabrication of superstructure. The bridge deck crosses the 6-lane State Highway 9A joining its southbound lanes with those of the Taconic State Parkway. In addition to the bridge structural work, the project required the construction of long box culverts adjacent to bridge foundations and extensive use of tall retaining walls faced with granite veneers that secure the fills of bridge approach embankments and the embankments of relocated bikepath.

1 PROJECT BACKGROUND AND HISTORY

The project is located in a narrow north-south oriented ecologically sensitive segment of the Pocantico River flood plain, and is hemmed in by State Route 9A/100, the Taconic State Parkway (TSP), a wide bikepath, and the Beech Hill Road, all of them framed by the surrounding Pocantico Hills, in the town of Mount Pleasant, Westchester County, NY, about 31 miles north of Times Square in New York City (Figure 1).

In the past two decades, the Taconic Parkway corridor has experienced significant increase in traffic volume due to economic expansion and associated residential developments. On March 13, 1987 the New York State Department of Transportation (NYSDOT) and the Federal Highway Administration (FHWA) approved a Final Environmental Impact/4(f) Statement (FEIS/4(f)) (NYSDOT/FHWA 1987), which addressed the proposed improvements of the Taconic State Parkway from Hawthorne Interchange to Campfire Road.

As part of the preferred alternative, Ramp 'W' was constructed to connect the northbound TSP with the northbound State Route 9A/100 about 15 years ago. A companion Ramp 'X' connecting southbound Route 9A/100 with the southbound TSP was also included in this alternative. The original location of Ramp 'X' was approximately 245 meters (804 ft) north of the exit ramp to the Saw Mill River Parkway at the Hawthorne Interchange. The configuration and location of Ramp 'X' as originally proposed is shown in Figure 2.

However, further analysis and investigation of the original Ramp 'X' location identified undesirable operational traffic characteristics associated with the closely spaced weaving sections and the use of a left side exit. To resolve these issues, NYSDOT decided to shift the proposed location

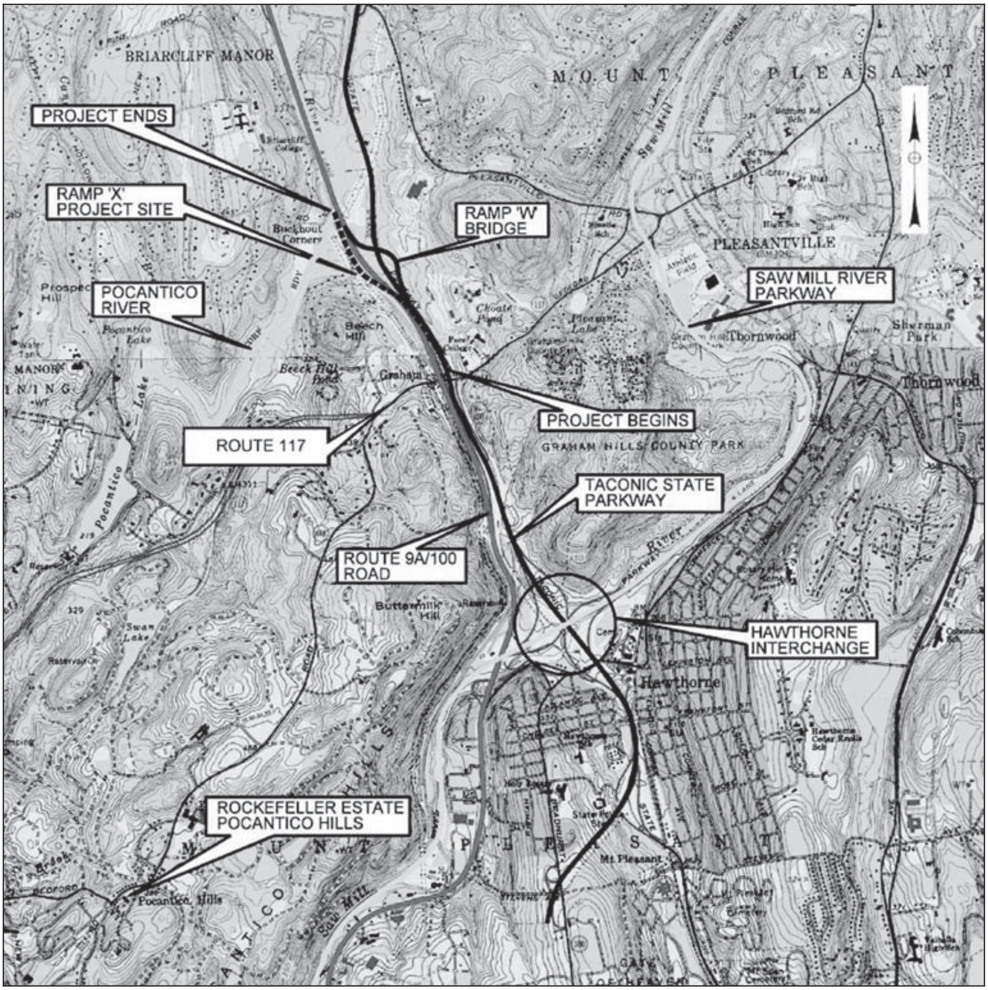


Figure 1. Location map.

of Ramp 'X' to an area north of Route 117 adjacent to the existing northbound Ramp 'W'. At this location the ramp junctions would operate free from the influence of adjacent ramp terminals, left side exits and weaving areas. The relocation would also provide standard acceleration and deceleration lanes for Ramp 'X'. To accommodate the new ramp, a 285 meter (935 ft) long segment of the existing North County Trailway (NCT) situated on the western side of Route 9A/100 was relocated. The original proposed FEIS Ramp 'X' location and the new Ramp 'X' location are shown in Figure 2.

Gandhi Engineering Inc. (Gandhi) was retained by the NYSDOT—Region 8 for the fifth and final stage of the actions approved under the FEIS/4(f). Gandhi prepared a Reevaluation Statement (NYSDOT/FHWA 2004) addressing proposed changes to the preferred alternative and performed environmental analysis covering a wide range of issues including air quality, noise, water quality, wetlands and ecology, stream channel realignments, flooding, neighborhood impacts, residential impacts, Right-of-Way requirements, parkland impacts, construction impacts, economic activity, scenic resources, energy study, cultural resources, endangered species and hazardous waste. The FHWA approved the Final Reevaluation Statement on June 25, 2004. Gandhi was authorized by the NYSDOT to proceed with the detailed design.

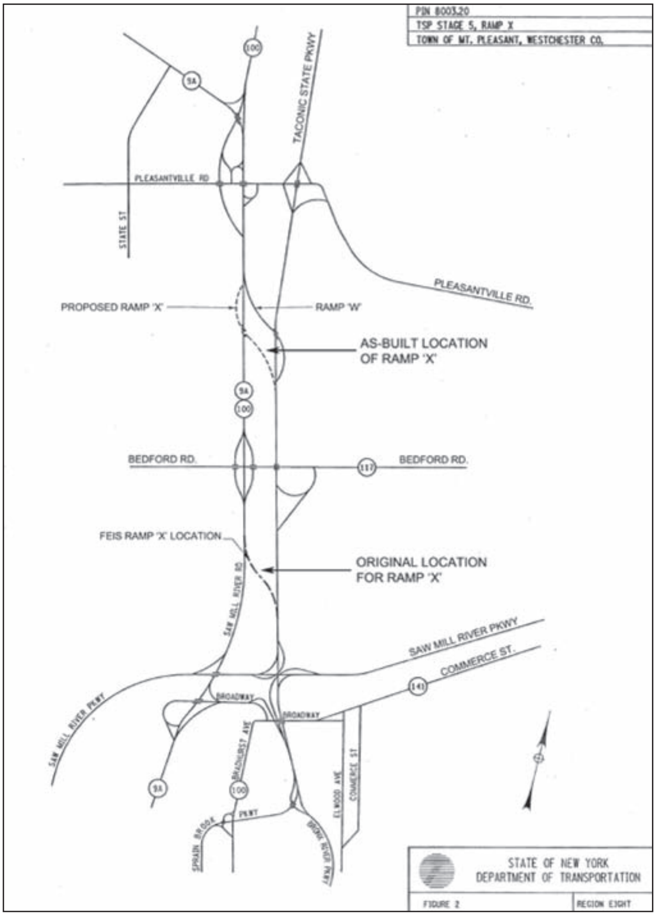


Figure 2. Original and as-built location of Ramp X.

2 DESIGN DETAILS

During the preliminary design phase, a 3-span continuous structure alternative was studied and compared with the single-span alternative. It was found that the 3-span continuous structure alternative would be much more costly, and it would require a pier located at the narrow median between the northbound and southbound Route 9A/100. The NYSDOT selected the single span alternative. The bridge deck crosses the 6-lane Route 9A/100 joining its southbound lanes with those of the Taconic State Parkway. As a result, the average span of the curved girders is about 51 meters (167 ft) as the span follows a pronounced vertical curve which further complicated the detailing and fabrication of superstructure.

The central feature of this project is a sharply skewed flyover bridge tightly aligned on compound horizontal alignment involving a circular arc segment joined by a tangent (Figures 3 and 4). The compound horizontal curves were a result of tight geometry constraints in the available space between Route 9A/100 and the Taconic State Parkway. The irregular curvatures of the bridge's steel framing required detailed analysis of stresses and deflections using the SAP2000 (Computer & Structures) general purpose finite element program. The irregular framing, including the girders and diaphragms, which are considered primary structural

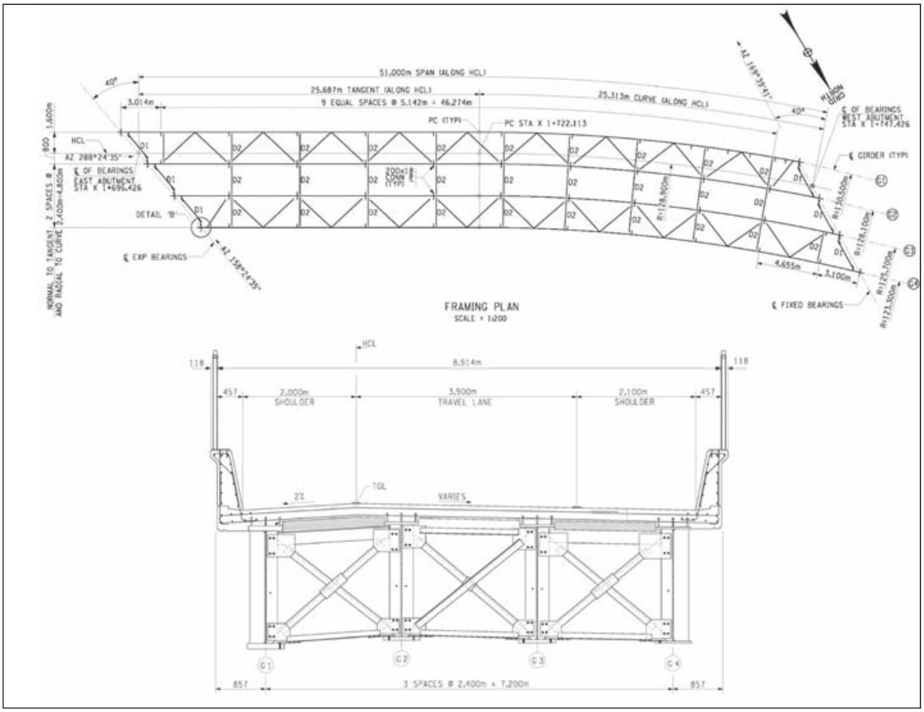


Figure 3. Ramp X Bridge framing plan and typical cross section.

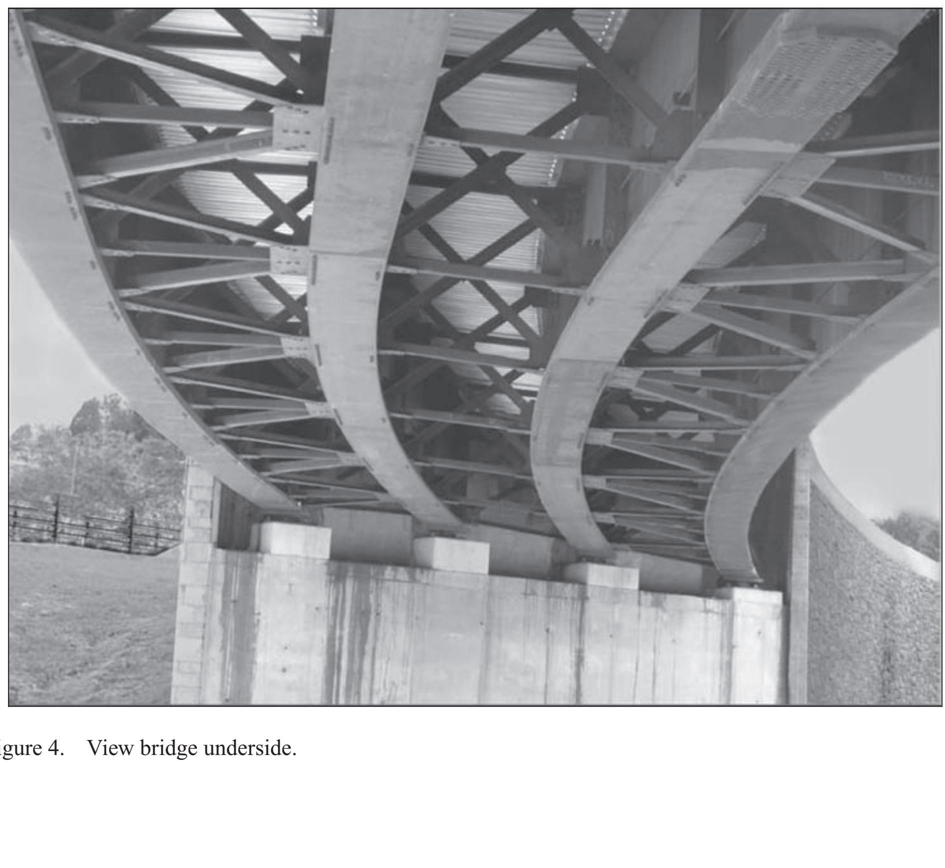


Figure 4. View bridge underside.

members in a horizontally curved structure, was imported into the SAP2000 program directly from the CAD drawings.

Environmental sensitivity of the project site is derived from the presence of eight NYSDEC-designated freshwater wetlands, and four US Army Corps of Engineers wetlands. One of the important aspects of this project was the creation of 18.2 acres of new wetlands in order to compensate the 15.6 acres of wetland lost as a result of the ramp construction. To minimize the adverse impacts on wetlands and the Beech Hill Road, a local access facility serving a residential enclave, the project required extensive use of tall retaining walls that secured the fills of bridge approach embankments, and the embankments of the relocated bikepath. Two types of retaining walls were used in the project including cast-in-place reinforced concrete retaining walls supported on steel H-piles for the bridge approaches, and a proprietary T-Wall system supported on fills for the relocated bikepath.

As mandated by NYSDOT, the bridge abutment wingwalls and retaining walls of the bridge approaches and those bordering the relocated bikepath were faced with granite veneers matching the color and pattern of those used in the adjacent Ramp W Bridge. This created an aesthetically pleasing and consistent design which blended in perfectly with the surrounding park-like setting of the Taconic State Parkway, without changing the landscape adjacent to the residential areas bordering Beech Hill Road. (Figures 5 and 6)

Another feature was the construction of a 2.40 m (8 ft) wide temporary bikepath during construction as part of the traffic control plans. This feature provided uninterrupted access to the North County Trailway (NCT) by the general public. Attesting the overall success of the project engineering and construction, NYSDOT received letter from members of the local citizens complimenting the agency on the temporary bikepath, which was a pleasant surprise to all participants in the project's creation at a time when the public's reaction to infrastructure jobs is often adverse.

Figure 5. View of west abutment wingwall and approach retaining wall.

Figure 6. View of relocated bikepath retaining wall.

3 CONSTRUCTION ASPECTS

The biggest challenge of this project was to maintain vehicular traffic on Route 9A/100 and the Taconic State Parkway with minimal disruption, because both facilities are multi-lane divided highways and carry approximately 71,000 and 84,000 AADT volumes, respectively, with a combined volume of 155,000. Much of this traffic load includes peak hour commuter flows destined for the New York City metropolitan area. Since no erection of any superstructure framing component is allowed by NYSDOT over live laneage, and the lane closure period was limited to ½ hour between midnight and 5 AM, the design drawing provided a suggested steel erection method involving field-splicing of pairs of oversize curved steel girders on the ground and installing fully connected diaphragms to allow for the speedy lifting of the package on the prepared bearing assemblies, all done within the ½ hour of lane closure period at night. During construction, the Contractor negotiated with the NYSDOT an alternative method based on using steel falsework in the very narrow median strip which allowed the erection of girder pairs in about half lengths for splicing in the air combined with lane closures during night hours (Figures 7 and 8). The selected construction method was executed skillfully by the Contractor without any mishaps or complaints from the public.

Another challenge was that due to the need for reverse superelevation as a result of the compound horizontal alignment, finishing of the bridge deck to the design profile and superelevations required careful setup and calibration of the screed machine.

The presence of existing Con Edison overhead electrical transmission lines within the project limits was another factor required careful planning during construction.

The Contract (D259968) was awarded to CCA Civil/Halmar Internation LLC joint venture. Construction began in April, 2006. As a result of the team work between the Region, the Design Consultant, the Resident Engineer and his staff and the Contractor, the project was successfully completed on time and within budget. The ramp opened to traffic in December, 2007.

The project team was awarded the 2008 "Tappan Zee Award" (Project of the Year) by the Lower Hudson Valley branch of the ASCE Met Section. In addition, the project was nominated by the NYSDOT to the 2008 America's Transportation Award co-sponsored by AASHTO, US Chamber of Commerce, and AAA for "On time, On budget and Innovative Management". Figure 9 shows the completed bridge.

Figure 7. Night time mobilization.

Figure 8. View of partially erected superstructure steel.

Figure 9. View of completed bridge.

Figure 10. View of relocated bikepath.

The project was also well received by the community. The relocated bikepath with improved access and standard bike railing enhanced the existing facility, and blended in nicely with its surrounding environment (Figure 10).

REFERENCES

Computer & Structures Inc., *SAP2000*, 1995 University Avenue, Berkeley, CA 94704.
NYSDOT/FHWA, *Final Environmental Impact/4(f) Statement*, January, 1987.
NYSDOT/FHWA, *Reevaluation Statement Taconic State Parkway Stage 5 – Ramp X*, June, 2004.

Chapter 5

Screening and analysis for the gusset plates of the Hawk Falls Bridge, Pennsylvania, USA

M.M. Myers
Arora and Associates, P.C., Lawrenceville, NJ, USA

ABSTRACT: In the aftermath of the I-35W Bridge collapse in Minnesota, the Federal Highway Administration issued a technical advisory to bridge owners to check the status of gusset plates on similar bridges. In response, the Pennsylvania Turnpike Commission initiated a screening program for the analysis of the Pennsylvania Turnpike's Hawk Falls Bridge. The 51-year old, 740 ft long bridge spans Mud Run and contains a three-span continuous, haunched Warren deck truss. An analysis of the bridge was performed using BAR7. The gusset plates were grouped into representative types and each type was analyzed considering worst-case loads. A finite element model was developed, which verified the results of the hand calculations, finding some stresses above allowable values, but below material yield strength. A monitoring program was subsequently implemented, and data will be collected through sensors and strain gages for one year to accurately evaluate the bridge and make decisions regarding future rehabilitation.

1 INTRODUCTION

At 6:05 P.M. EST on Wednesday, August 1, 2007, the bridge over the Mississippi River between University Avenue and Washington Avenue on highway I-35W in Minneapolis, MN, collapsed. Numerous vehicles were on the bridge at the time, and, as is well known, there was a tragic loss of life and a vital transportation link was severed. At the time, in light of the uncertainty surrounding the cause of the I-35W Bridge collapse, the Federal Highway Administration (FHWA) advised all State Transportation Agencies and other bridge owners to immediately re-inspect all steel deck truss bridges with fracture critical members or at a minimum to review inspection reports, including those for routine, in-depth, fracture critical, and underwater, to determine whether more detailed inspections were warranted.

Due to the similarity between the Hawk Falls Bridge, shown in Figure 1 below, and the I-35W Bridge, the Pennsylvania Turnpike Commission (PTC) initiated an investigation of the gusset plates on the Hawk Falls Bridge, and Arora and Associates, P.C. was hired to perform the structural analysis.

The Hawk Falls Bridge, located in the Poconos just south of Exit #35 on the Northeast Extension (I-476) of the Pennsylvania Turnpike, carries I-476 over Mud Run, which forms the Hawk Falls Gorge. This is the highest bridge crossing on the Pennsylvania Turnpike at 190 ft above the gorge. The five-span bridge consists of a continuous haunched Warren deck truss, which makes up the three main spans, and two deck girder side spans. The overall bridge span is approximately 740 ft, and its width is 61 ft 6 in. The longest of the five spans is the center span, which is 280 ft long.

2 METHODOLOGY

A review of the bridge inspection reports, design calculations, photograph logs, and plans provided by PTC was performed, and a detailed history of the bridge was compiled considering its structural configuration, modifications made through rehabilitation and maintenance

Figure 1. Elevation of the Hawk Falls Bridge.

contracts, and the condition of the primary members. This was helpful in determining what changes had been made to the structure since the original design and construction and the current state of stress in each of the primary members and, hence, the gusset plates. Using the information found in these documents, the most recent BAR7 run performed on this structure (2005) was reviewed and updated. BAR7 v. 7.11.0.6 was used to reanalyze the bridge and generate new loads, which represent the current state of stress on the truss. Based on the BAR7 results and the gusset plate geometry detailed in the original shop drawings, the gusset plates were grouped into categories for further analyses. The gusset plates were then investigated using the analysis techniques presented in the FHWA Turner-Fairbank Highway Research Center report on the I-35W Bridge dated January 11, 2008.

3 BAR7 RUN

The model used in the previous BAR7 run was reviewed to verify that the project information, bridge cross section, and truss geometry were accurate and that the run was performed correctly. The BAR7 model that was used to analyze the three-span, continuous deck truss unit is shown in Figure 2.

The member properties were carried over from the previous BAR7 run. However, it was necessary to adjust the model in some areas to account for the following:

– Panels 9–20 are physically all the same width. The previous BAR7 run listed some of these panels as 23.33 ft wide and the rest as 23.34 ft wide such that the sum of the widths would

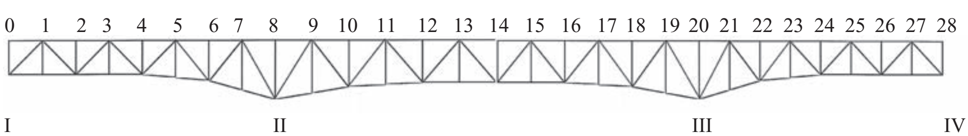


Figure 2. Truss geometry with node and support labels.

equal the center span length on the bridge. The width of panels 9–12 and 17–20 were changed to 23.34 to match panels 13–16 so that all panels would be the same width, as they are on the bridge. The change in panels 9–12 and 17–20 caused the overall length of the span to change slightly as well.
– Where necessary, the dead loads were adjusted to represent the current bridge configuration.
– A hinge was introduced into the model at L0 and one was removed from the model at U0 in order to avoid large asymmetries in the end vertical truss members.

There was an additional input change that was considered, but ultimately was not made. The width of panel no. 1 was entered as 21.37 ft in the previous run, which made it the same as its symmetrically similar panel, no. 28. In actuality, these panels are not the same width. Panel no. 1 is 20.63 ft wide instead of 21.37 ft as entered in the previous BAR7 run. This was probably done to simplify the calculations by having the whole bridge be symmetrical. By choosing 21.37 ft instead of 20.63 ft, a more conservative option is implemented (i.e. higher forces are generated), so this input was left unchanged.

With the revised input and updated model, BAR7 was run and new member forces were generated. Following guidance provided by PTC, the bridge owner, the live load cases considered were HS20 and P-82 loadings to account for regular and permit vehicles. For the P-82 live load run, it was assumed that there were three lanes loaded with 100%, 55%, and 55% of the truck load respectively. The two 55% lanes approximate the force of an HS20 live load, while the 100% represents the one lane of P-82 truck loading.

4 GUSSET PLATE GROUPS

There are 58 gusset plates in each of the deck trusses. Rather than analyze every gusset plate, a representative sample of gusset plates with worst-case scenario loads and geometry was selected for analysis. The first step was to split the plates into categories based on their thickness and whether they were located on the upper or lower chord of the truss. This resulted in 9 different categories. One of these categories consisted solely of the panel points at the pier supports. Since these panel points contain 4 gusset plates stacked together, they represent a unique situation. Within each of the other categories, the worst-case location was determined using the forces calculated in the new BAR7 run. Most of the gusset plates chosen were the ones whose connecting truss members had the largest overall forces. However, in some cases, gusset plates with the most extreme connecting member angles were selected, since this could cause large moment and shear forces in a plate.

5 GUSSET PLATE ANALYSIS

Calculations were performed to analyze the representative gusset plates to determine the forces and stresses acting on them. The calculations and methods used were based on the reconstructed design calculations for the I-35W over Mississippi River Bridge illustrated in the FHWA report. The procedure makes two cuts in the plate, one horizontal, just above or below the horizontal member (section A-A) and one vertical, just next to the vertical member (section B-B), as shown in Figure 3.

Based on the BAR7 member forces, equilibrating axial and shear forces and the moment about the midpoint of the cut plane are calculated to balance the truss member forces. The stresses along the section and the maximum tension and compression stresses either at the section neutral axis or at the edge of the plate are then calculated, taking into account any splice plates that contribute to the gusset plate section properties. Lastly, a Demand/Capacity (D/C) ratio is produced using AASHTO allowable stresses as the capacities.

The review of the available bridge plans and documents did not reveal which edition of AASHTO was used to perform the original bridge design or what grade of steel was used.

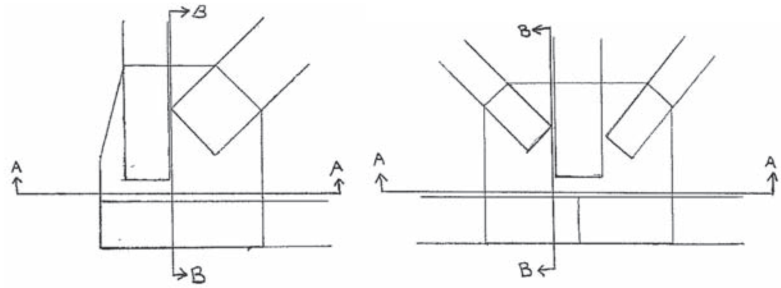


Figure 3. Typical section locations for end and interior gusset plates.

Therefore, the AASHTO Manual for Condition Evaluation of Bridges, Second Edition was used to determine the allowable stresses. In the Manual, there is a category for "unknown steel" constructed between 1936 and 1963, which lists Grade 33 steel. Since the bridge was designed in 1954 and constructed in 1957, Grade 33 steel was used to determine the allowable stresses for these calculations.

For Inventory Rating (IR) compression allowable stress, the AASHTO Manual for Condition Evaluation of Bridges formula for compression in concentrically loaded columns for bridges built between 1936 and 1963,

$$F_{comp} = 15{,}570 - 0.45(KL/r)^2 \tag{1}$$

was used, since the FHWA procedure models the edge of the gusset plate as a column. It is noted that the 1949 version of AASHTO uses

$$F_{comp} = 15{,}000 - \frac{1}{4}(L^2/r^2) \tag{2}$$

for compression in concentrically loaded columns having values of L/r not greater than 140 and with riveted ends. The differences between Equation 1 and Equation 2 would not have significantly changed the results of the analysis. Therefore, the AASHTO Manual equation (Eq. 1) was the one chosen, since it is a more general guideline. The Operational Rating (OR) equation for compression allowable stress that was used is

$$F_{comp} = 19{,}410 - 0.56(KL/r)^2 \tag{3}$$

The AASHTO Manual was also followed to determine the allowable stresses for tension (F_{ten} = 18,000 psi for IR and 24,500 psi for OR) and shear ($F_{v\text{-}avg}$ = 11,000 psi for IR and 15,000 psi for OR). The same allowable stresses used for tension were also used for bending allowable stress, F_b.

Based on the calculated stresses and D/C ratios, it was evident which gusset plates would require further investigation. There are 2 D/C ratios over 1.00 for the HS20 live load case, both for F_{comp}. These ratios occur in gusset plates U9 (1.00 at section A-A) and U13 (1.03 at section B-B). The gusset plates in the table that have NOT APPLICABLE written in the Section B-B columns are placed at the intersection of a vertical web truss member and a chord truss member. These are located at the centers of continuous chord members and are therefore not carrying much, if any, of the horizontal load. This is why the vertical cut was not considered applicable for the analysis. A summary of the analysis results is shown in Tables 1 and 2 below for HS20 and P-82 loading, respectively.

Table 1. Summary of D/C ratios calculated under HS20 live loading for all analyzed gusset plates.

	Section A-A				Section B-B			
HS20	f_b	$f_{v\text{-}avg}$	f_{ten}	f_{comp}	f_b	$f_{v\text{-}avg}$	f_{ten}	f_{comp}
U7	0.32	0.57	0.48	0.72	0.70	0.38	0.99	0.34
U8	0.00	0.00	0.00	0.68	NOT APPLICABLE			
U9	0.39	0.77	0.67	1.00*	0.57	0.69	0.75	0.66
U13	0.22	0.36	0.29	0.45	0.60	0.25	0.12	1.03*
U14	0.00	0.00	0.00	0.82	NOT APPLICABLE			
L0	0.26	0.39	0.22	0.77	0.08	0.40	0.34	0.48
L7	0.00	0.01	0.03	0.03	NOT APPLICABLE			
L8	0.04	0.05	0.01	0.36	0.05	0.20	0.14	0.32
L14	0.00	0.00	0.05	0.00	0.36	0.13	0.81	0.13
L15	0.00	0.00	0.06	0.00	NOT APPLICABLE			
L16	0.29	0.58	0.51	0.69	0.38	0.40	0.78	0.26
L28	0.30	0.39	0.21	0.84	0.10	0.40	0.34	0.48

* Cases where D/C ratio is greater than or equal to 1.0.

Table 2. Summary of D/C ratios calculated under P-82 live loading for all analyzed gusset plates.

	Section A-A				Section B-B			
P-82	f_b	$f_{v\text{-}avg}$	f_{ten}	f_{comp}	f_b	$f_{v\text{-}avg}$	f_{ten}	f_{comp}
U7	0.28	0.50	0.43	0.70	0.69	0.34	0.91	0.46
U8	0.00	0.00	0.00	0.66	NOT APPLICABLE			
U9	0.31	0.61	0.53	0.87	0.36	0.54	0.56	0.60
U13	0.19	0.31	0.26	0.42	0.50	0.21	0.10	0.94
U14	0.00	0.00	0.00	0.79	NOT APPLICABLE			
L0	0.24	0.36	0.20	0.77	0.08	0.37	0.32	0.48
L7	0.00	0.01	0.02	0.03	NOT APPLICABLE			
L8	0.03	0.03	0.00	0.32	0.04	0.16	0.11	0.28
L14	0.00	0.00	0.06	0.00	0.29	0.12	0.67	0.11
L15	0.00	0.00	0.04	0.00	NOT APPLICABLE			
L16	0.24	0.48	0.43	0.64	0.32	0.33	0.66	0.23
L28	0.27	0.36	0.20	0.84	0.10	0.37	0.32	0.48

6 ANALYSIS RESULTS

Unsupported edge length adequacy was also studied. Only three of the gusset plates investigated did not comply with the unsupported edge limit, expressed by

$$\text{Unsupported edge limit} = [11{,}000/\sqrt{F_y}] * (\text{plate thickness}) \tag{4}$$

This equation is for the unstiffened unsupported edge limit from the AASHTO Standard Specifications for Highway Bridges, Ninth Edition. Of the three plates not complying with the unsupported edge limit, one has an edge in tension, and the other two are effectively braced, as can be seen in Figure 4 below. Therefore, it was concluded that the gusset plates meet the criteria for unsupported edge length limit.

Figure 4. Typical lower, even numbered gusset plate, identifying unsupported edge. It can be seen that there are rivets near the edge for its entire length.

7 FINITE ELEMENT MODELING

As a verification of the hand calculations, a 2D finite element model using STAADPRO 2005 software was developed. Interior gusset plate L16 was chosen as a representative example for this model. The gusset plate dimensions, connection configuration and the limits of splice plates were established after a review of the original shop drawings. The geometry of gusset plate L16 can be seen in Figure 5.

Nodes (joints) were generated to define boundaries for the gusset plate, limits of splice plates, and the locations of the fasteners in each of the five connections. For simplicity, the connection holes in the gusset plate were not modeled. A 4″ × 4″ grid of nodes was placed over this set of "fixed" joint locations to fill in the areas between these key points.

The node geometry was reviewed and joints from the 4″ × 4″ grid that conflicted with either the connection, splice plate boundary, or gusset plate boundary nodes were deleted. Locations of conflict were determined by engineering judgment. Triangular plate elements were then generated between the remaining joints. Figure 6 shows the layout of the nodes and triangular plate elements.

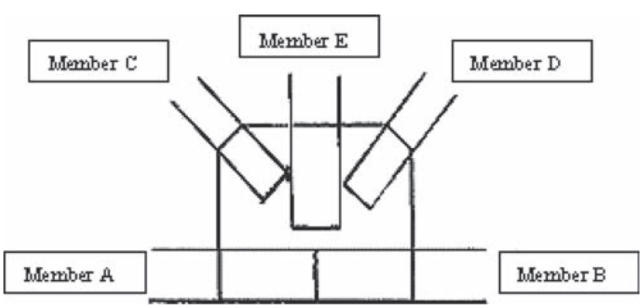


Figure 5. Geometry of gusset plate L16.

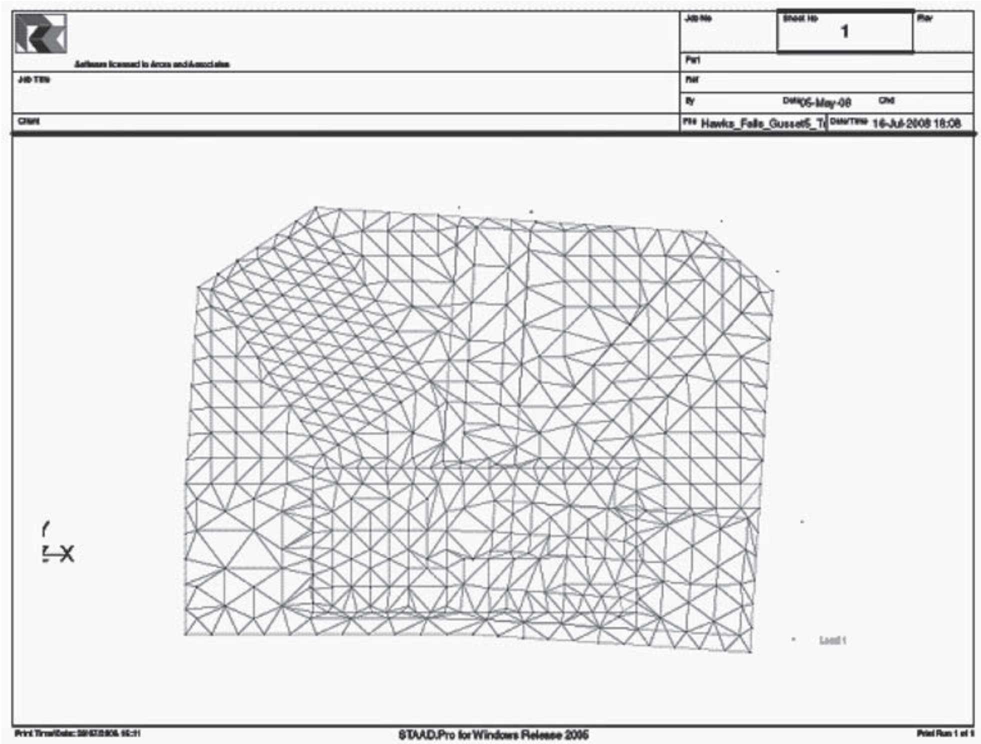


Figure 6. The finite element model depicting nodes and triangular plate elements.

The plate elements within the limits of the spice plate were assigned a thickness of the combined gusset and splice plates (2″), while plates outside this area were assigned a thickness of the gusset plate alone (5/8″).

The basic geometry model was analyzed for each of the sections evaluated with the hand calculations. For the horizontal section A-A, truss member forces from members B, C, and E were applied to the plate. For vertical section B-B, the member forces from members A and C were applied.

Support conditions for the finite element model were given careful consideration. Axial forces taken from the BAR7 output represent the maximum values in a given member, but these forces do not necessarily occur under the same loading conditions. As a result, the gusset plate as an individual free body is not in equilibrium when these maximum loads are applied simultaneously. Separate support conditions were established to develop equilibrating forces in the truss members on the side opposite of the section under consideration. This was accomplished by placing additional nodes in line with the fasteners of a given connection and just beyond the gusset plate boundary. These nodes were assigned the properties of a pinned support. From these supports, truss members were generated to each of the fastener nodes in the adjacent connection. These additional truss members were assigned a proportion of the area of the corresponding truss member. Loads were applied to the remaining connector nodes depending upon which section of the gusset plate was being evaluated.

For example: When evaluating vertical Section B-B, forces from truss members A and C were applied to the model and supports were generated at additional nodes located in line with the existing connections of truss members B, D and E.

Unit (1 kip) axial forces, resolved into components along the global X- (horizontal) and Y- (vertical) axes, were applied to each of the fastener nodes. Component values were calculated based upon the geometry of the individual connection. The forces from each truss member were considered as a separate load case. The live and dead load axial force values obtained from the BAR7 analysis were divided equally amongst the number of connectors in each member. This "per connector" value was then applied as a factor to the unit load cases in the Load Combination command.

Stresses were checked using the STAAD feature that allows the user to define a cutting plane. Using this STAAD command, horizontal and vertical cutting planes were established at the same locations in the model that were evaluated in the hand calculations. The stresses are summarized in Tables 3 and 4 below. Positive values represent tensile stresses; negative values represent compressive stresses.

Qualitatively, the STAAD model yielded results that matched those of the hand calculations. The difference in the stress values for the Section A-A model are attributed to local effects due to the support conditions. The model for evaluating Section B-B provides stresses that are in general agreement with the hand calculations. Also of note is the distribution of the stresses in the gusset plate. The larger stresses were found to occur at the edges of the cut section and reduced substantially towards the center of the plate. See Figure 7 below for stress contours on the gusset plate.

The model was then modified to account for section loss in the gusset plate that was documented in the latest available bridge inspection report. Inspection photos revealed that the typical

Table 3. Comparing stresses for horizontal section A-A of gusset plate L-16.

Section A-A	Left side of plate ksi	Right side of plate ksi
Hand calculations	–4.7	5.7
STAAD model	–2.8	0.4

Table 4. Comparing stresses for vertical section B-B of gusset plate L-16.

Section B-B	Left side of plate ksi	Right side of plate ksi
Hand calculations	–3.1	14.03
STAAD model	–4.9	14.0

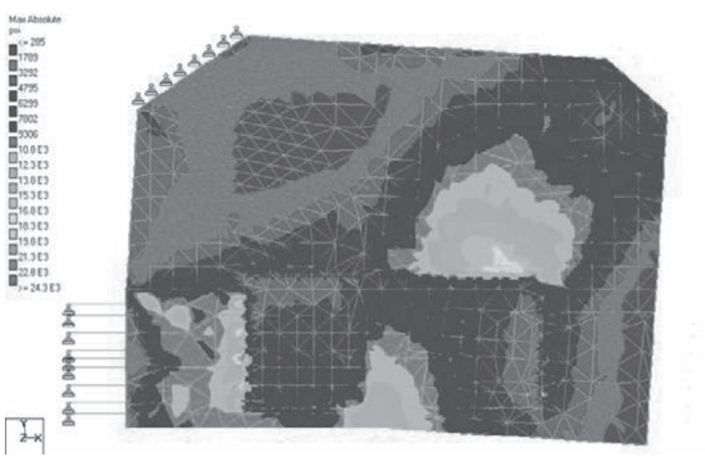


Figure 7. Stress contours on gusset plate L16 from finite element model.

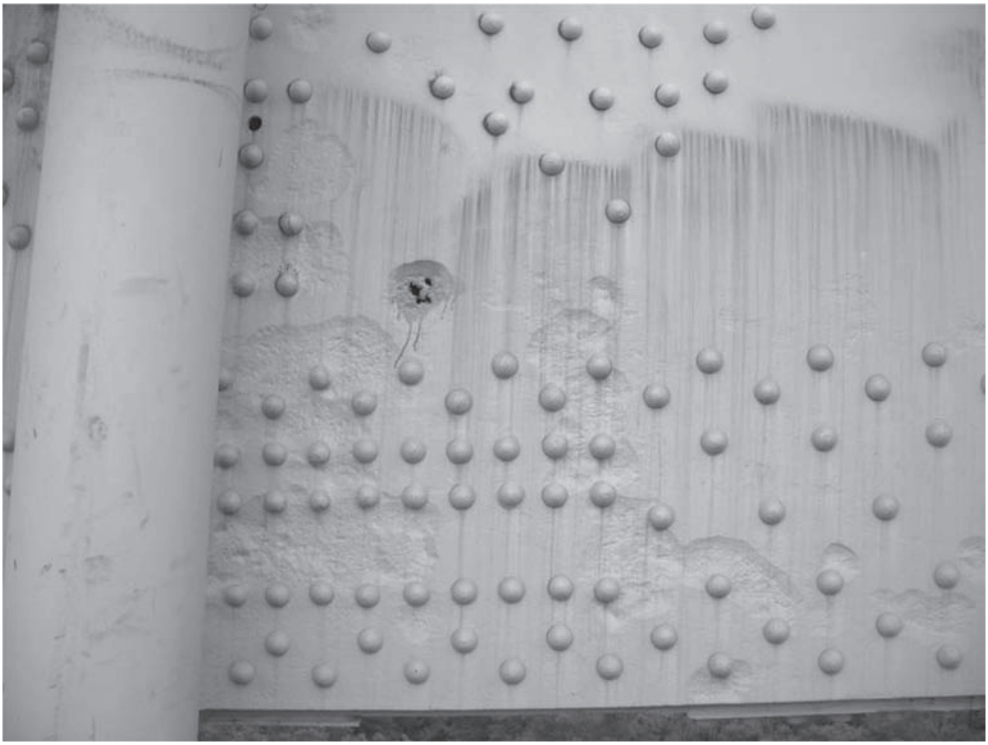


Figure 8. Typical section loss on gusset plate.

corrosion on the plate occurred just above the bottom chord members, as can be seen in Figure 8. Based upon a review of this information, the gusset plate elements immediately above the limit of the splice plates were reduced in thickness from ⅝″ to ½″. The model was re-analyzed and it was found that the section loss raised the stresses in the gusset plate by approximately 10% in the area of the section loss.

8 CONCLUSIONS

The analysis of the Hawk Falls Bridge gusset plates revealed only two locations with a Demand/Capacity Ratio equal to or greater than 1.00. These were at panel points U13, where the D/C Ratio was 1.03 and U9, where the D/C ratio was 1.00. A finite element analysis of the gusset plate at panel point L16 was performed, and compared to the hand computations made using the method of sections. The maximum stresses obtained from the finite element analysis were found to be of similar magnitude to those obtained from the hand computations. The finite element analysis of the L16 plate indicated that a 20% loss in plate thickness occurring just above the bottom chord, which is typical throughout the Hawk Falls structure, resulted in only a 10% increase in stress at the reduced area. The results of the analysis indicate that the gusset plates for the bridge have adequate capacity to support the current dead load and live loads carried by the bridge. Based on the analysis for dead and live load, no repair or rehabilitation actions were recommended beyond the continued inspection program and the PTC preventative maintenance program. The above analysis satisfied the initial needs of the PTC to screen the gusset plates and perform an initial analysis to verify the current, load-carrying capacity of this important, non-redundant deck truss bridge.

As with all bridges, the continued operation and maintenance of the Hawk Falls Bridge is an ongoing concern. Previous investigations have revealed that the frozen bearings on the structure contribute to increased forces in the truss members due to temperature variations. Evaluation of these additional forces was beyond the scope of this analysis, and it was recommended that a more detailed evaluation be performed that quantifies and assess the impacts of the thermal stresses on the plates. Since these analyses were performed, PTC has implemented a continuous monitoring program in April 2009 to gather additional data. A detailed analysis based on actual field readings will be performed following the data collection phase. This monitoring of the gusset plates will be accomplished through installation of various sensors and strain gauges. The data from these instruments will be consistently recorded and monitored for one year such that all seasonal variations and traffic cycles can be observed.

REFERENCES

Arora and Associates, P.C. *Pre-final report: gusset plate analysis of Hawk Falls Bridge prepared for Pennsylvania Turnpike Commission for project WBS Element EN-00035 Work Order No. 3*. August 4, 2008. Harrisburg, PA.

Henry, L.S. *Over the river and through Penn's Woods: a look at the Turnpike's river bridges.* http://www.paturnpike.com/newsletters/summer97/page2-3.htm. April 21, 2009. Pennsylvania Turnpike Commission.

Holt, R. & Hartmann, J. *Adequacy of the U10 & L11 gusset plate designs for the Minnesota bridge no. 9340 (I-35 over the Mississippi River) interim report*. January 11, 2008. Federal Highway Administration.

Rosenker, M.V. et al. Safety Recommendation H-08-1. January 15, 2008. National Transportation Safety Board, Washington, D.C.

Technical Advisory 5140.27—Immediate inspection of deck truss bridges containing fracture critical members (FCM). August 2, 2007. Federal Highway Administration, Washington, D.C.

Chapter 6

Selection and design of the high load multirotational bearings for the St. Anthony Falls Bridge in Minnesota

K.L. Western
Minnesota Department of Transportation, Oakdale, Minnesota, USA

T.A. DeHaven
Figg Engineering, Eagan, Minnesota, USA

R.J. Watson
R.J. Watson, Inc., Amherst, NY, USA

ABSTRACT: The construction of the New St. Anthony Falls Bridge over the Mississippi River in Minneapolis was no doubt a monumental achievement. The joint venture team of Flatiron Constructors and Manson Construction built this structure in 11 months which was 3 months ahead of an already aggressive schedule without any lost time injuries. The designer of this impressive \$234 million 378 m (1233 ft.) long segmental concrete twin span structure (Figg Engineering Group) worked closely with the Minnesota Department of Transportation and the local community in coming up with a highly functional and aesthetically pleasing bridge. One of the important aspects of this project that has not commanded a great deal of attention is the design and selection of bearings to accommodate the loads, movements and rotations of this signature bridge. The bearings selected for this project were high load multi-rotational devices that ranged in vertical load resistance from 5330 kN (1200) to 25,780 kN (5800) kips. Four types of bearings were utilized to support the segmental boxes. One type required uplift restraint during a factored load case however a positive reaction was noted for all service cases. The bearing also featured details that accommodated replacement and were corrosion resistance. Under such as aggressive schedule the process of selection, detailing, testing, and shop assembly required streamlining and parallel processes to ensure on-time delivery without reducing quality. This paper will go into detail on the design and selection process for these important devices. In addition the manufacturing process and testing scheme will be discussed in an effort to display the versatility of these critical mechanisms.

1 INTRODUCTION

August 1, 2007 is a day that most engineers will not forget for a long time to come. The collapse of the I-35W steel truss bridge sent shock waves through the engineering community (Figure 1). Remarkably just 3 days after the collapse the Minnesota Department of Transportation issued a request for qualifications from design/build teams interested in constructing a replacement bridge. Four days after this a short list of teams was issued. By October 8, 2007 the team of Flatiron Constructors, Manson Construction and Figg Bridge Engineers had received their notice to proceed with the replacement structure (Van Hampton, 2007).

2 NEW BRIDGE DETAILS

The new bridge, constructed in 11 months (3 months ahead of an already aggressive schedule) is 377 m (1223 ft.) long with a 154 m (504 ft.) long main span (Figure 2). In addition to the contract price of \$234 million the team earned an additional incentive of \$27 million for completing the structure

Figure 1. Collapse of the I-35W Bridge in Minneapolis.

Figure 2. The completed St. Anthony Falls Bridge (photo courtesy of Figg Bridge).

by mid September. The bridge being out of service was costing the State of Minnesota in excess of \$400,000 per day and early completion was rewarded with an incentive of \$200,000 per day.

The bridge which is designed for a 100 year life has four 21 m (70 ft.) tall piers supported on drilled shafts into bedrock. The bridge is comprised of twin structures with a combined width of 55 m (180 ft.) which allow for 10 lanes of traffic and future potential for bus or light rail transit.

The main span is constructed of precast concrete segments while the approach spans of 97 m (319 ft.), 76 m (248 ft.) and 45 m (148 ft.) are cast-in-place concrete constructed on falsework. Each of the precast concrete segments are 13 m (42 ft.) wide, 4.0 (13) or 4.9 m (16 ft.) long and up to 7.6 m (25 ft.) deep. There are 120 segments which weigh from 1330 kN (150 tons) to 1780 kN (200 tons) each (Cho & Thilmany, 2008).

Supporting these segments and transmitting the load down to the substructure is the task of the little discussed but vitally important bridge bearings. In addition to accommodating the vertical loads these bearings must also transmit live load rotations and displacements due to thermal changes and creep shrink.

3 DISK BEARING CONCEPT

After much debate and deliberation the joint venture team selected high load multirotational disk bearings (Figure 3). Disk bearings were originally developed in the early 1970's as a cost effective superior performance alternative to pot or spherical bearings.

The key to the functionality of the disk bearing is the use of a polyether urethane load and rotational element. Polyether urethanes have tremendous compressive strength and have outstanding weathering properties. In addition they have a material stability range of –70° to +120°C (–94° to +248°F.) (Watson, 1986). The outstanding physical properties of polyurethanes result in their having many consumer uses such as garden hoses, bowling balls and in-line skates. In addition many paints and protective coatings contain urethane which give them good flexibility, durability and toughness.

Under the design load the polyurethane pad will deflect in the range of 10–15%. This differential deflection gives the bearing the ability to distribute rotations regardless of the direction of orientation. The maximum allowable pressure in the rotational element is set by AASHTO at 35 MPa (5 ksi). However the ultimate load the polyurethane is capable of is on the order of 20 times that load. Therefore the vertical load safety factor is very high. This is important because during the superstructure installation process bridge bearings are frequently overloaded. The rotation of these bearings transmits very little moment into the substructure, therefore they allow for an efficient foundation design where the superstructure loads are primarily vertical loads on the columns. On the I-35W project there was very little room for large foundations so the disk bearings were the designer's preferred bearing choice.

Figure 3. Unidirectional disk bearing.

In order to accommodate translation a PTFE disk is bonded on top of the upper bearing plate. As a means of preventing the PTFE from migrating over time it is also recessed into the upper bearing plate which mechanically locks it into place. Bearing on top of the PTFE disk is an upper slide plate which is faced with a highly polished mirror finish stainless steel sheet. Sliding friction testing at design pressures of 24 MPa (35 ksi) typically yield coefficient of friction values of 2% or less.

One of the first major structures to utilize disk bearings was the Pasco Kennewick Intercity Bridge over the Columbia River in the State of Washington. The Pasco Kennewick Bridge was the first cable stay bridge built in the 48 contiguous states. It is 782 m (2500 ft.) long with a 296 m (970 ft.) main span. The disk bearings on this bridge range from 2670 kN (600) to 12400 kN (2800 kips) in vertical capacity. Since the deck was designed to be continuous all of the movement of the structure was designed to be taken at one joint at the expansion abutment. A large modular expansion joint system was utilized designed for 660 mm (26 in.) of movement. Likewise the disk bearings at the expansion joint also had to accommodate this magnitude of displacement. This was simply done by designing the upper slide plate long enough to provide this amount of movement. From the PTFE down the disk bearing design would be the same regardless of the movement.

Over 30 years later the disk bearings on the Pasco Kennewick Bridge are performing well and are in excellent condition (Figure 4).

Other notable disk bearing installations are on the Sunshine Skyway Bridge near Tampa Florida, (Figure 5) the H-3 Windward Viaduct in Hawaii, the Penobscot Narrows Cable Stay Bridge (Figure 6) in Maine and the Hoover Dam Arch Bridge on the Arizona/Nevada border.

4 DISK BEARING DESIGN

The fabrication and erection of the multi-rotational disk bearings for this I-35W Bridge was a task in itself due to the staggering size and capacity of the bearings that were chosen for this project.

Disk bearings were chosen because of their excellent history of low maintenance, easy inspection, and simple design, in addition to the fact that supplier was able to meet the expedited schedule. The fabricator's engineering staff were in almost constant contact with the design-build team to design, manufacture, test, and ship the bearings to the site in order to meet the proposed schedule.

Figure 4. 30 year old disk bearing on the Pasco Kennewick Bridge.

Figure 5. Sunshine Skyway Bridge, Florida (photo courtesy of Figg Bridge).

Figure 6. Penobscot Narrows Bridge, Maine (photo courtesy of Figg Bridge).

At the south abutment there are four guided and four non-guided expansion disk bearings which are designed for vertical service loads of 9780 kN (2200 kips) with a horizontal strength load capacity of 3640 kN (820 kips) a maximum longitudinal displacement of 508 mm (20 in.) and a maximum rotation of ±0.02 radians.

At the piers two and three, the 24 fixed disk bearings are designed for vertical service loads of 25,780 kN (5800 kips) and a maximum rotation of ±0.02 radians (Figure 7). Another design feature for the bridge is at span three of pier four, which required eight guided expansion disk bearings to accommodate 890 kN (200 kips) of uplift capacity in addition to 6220 kN (1400 kips) of vertical load (Figure 8). The reason for uplift restraint at this span is related to a particular strength load-case, in which live load is positioned on one side of the box girder in combination with a wind load. Conventional disk bearings can be easily modified to resist uplift forces. First the shear resisting mechanism (SRM) at the center of the bearings is reconfigured so that it acts much like a trailer hitch on an automobile. By connecting the upper and lower bearing plates with this modified SRM the bearing then has the ability to resist uplift forces. If displacement capacity is also required of the uplift bearing the upper slide plate is then outfitted with guide bars that tuck underneath the upper bearing plate. Due to this innovative design, these bearings are capable of accommodating uplift, displacement and rotation simultaneously. The bearings also contain stop bars to limit longitudinal service movements. The remaining 14 fixed disk bearings at pier four, span four, carry 5330 kN (1200 kips) of vertical load.

4.1 *Replaceability details*

One of the other features that attracted the design team to disk bearings is the ease of replacement details (Figure 9). Most transportation design authorities require that bearings be designed so that they can be replaced with a minimal amount of jacking. This feature was utilized on this project on span 1 where the contractor needed to proceed with construction while the bearings for this location were being tested. The span was set on temporary supports. Once the production bearings were approved they were installed after the precast segments were already placed. The replaceability design utilizes a bolted in containment plate which keeps the bearing fixed in place. To

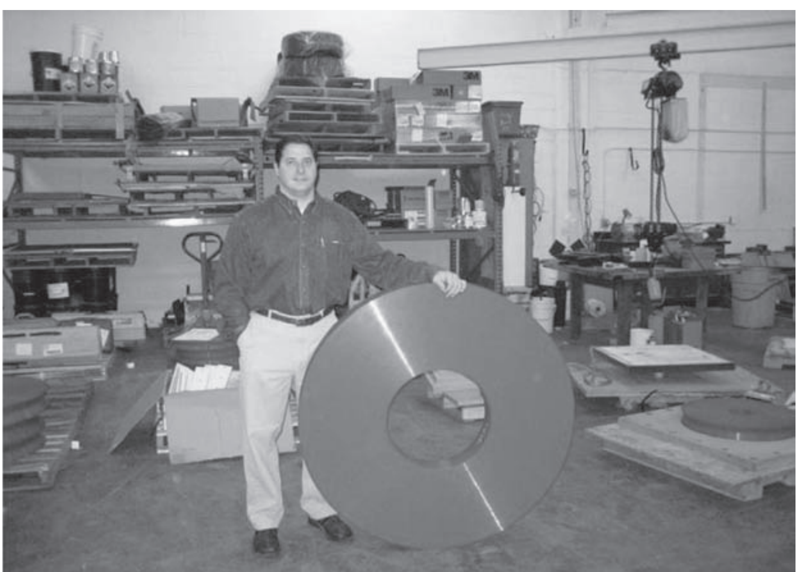


Figure 7. Polyurethane rotational element for 25,780 kN (5800 kip) disk bearing.

facilitate removal the containment plate is unbolted and removed allowing the bearings assembly to be pulled out in one piece.

5 TESTING

Very few facilities in North America have the capacity to test bearings at such high loads as those required for the bearings at piers two and three of this bridge. The University of California at San Diego SRMD Facility (Figure 10) was chosen for full-size testing of two disk bearings according to Section 18 of AASHTO LRFD bridge construction specifications, which included simultaneous horizontal and vertical proof load testing (Figure 11). The remainder of the testing required was completed by the fabricator at its own testing facilities, and all bearings met or exceeded the AASHTO LRFD testing requirements.

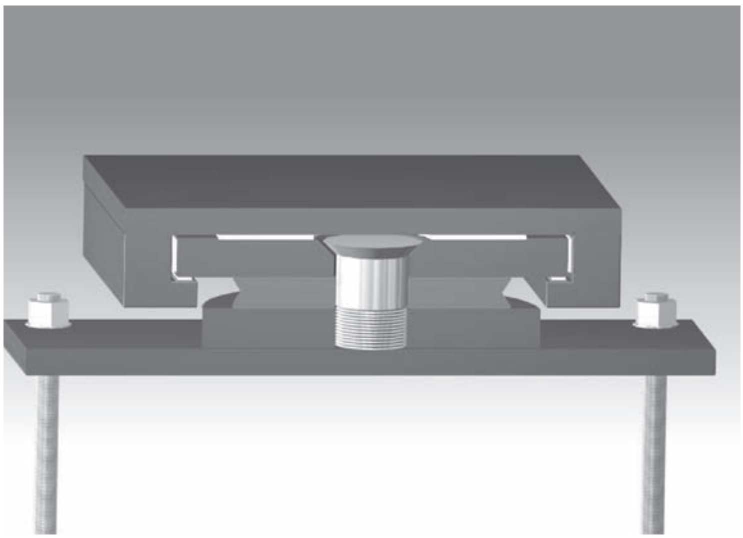


Figure 8. Uplift restraint disk bearing.

Figure 9. Disk bearing installed on I-35W Bridge showing containment plate.

Figure 10. UCSD/CALTRANS SRMD test facility.

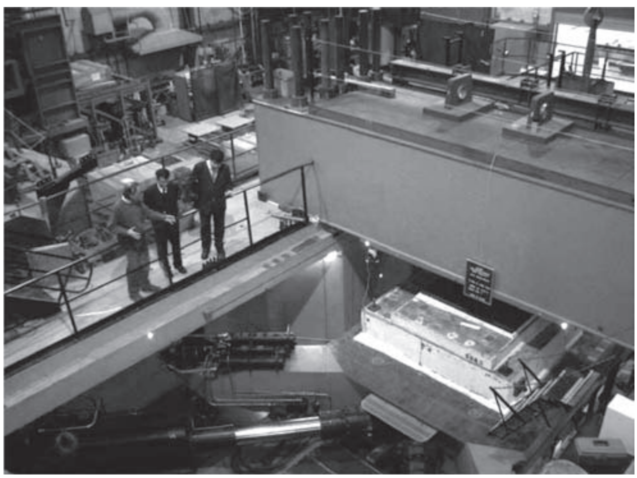


Figure 11. Disk bearing undergoing testing at SRMD test facility.

6 CONCLUSIONS

In conclusion, the bearing requirements for a fast track, high profile project such as the St. Anthony Falls Bridge in Minneapolis were numerous. Rotational capacity, high vertical and horizontal loads, uplift restraint, ease of replacement and a history of long term performance were just some of the features required for this project. Disk bearings met all of these requirements and provided the Minnesota DOT a cost effective solution.

REFERENCES

Cho, A.T. & Thilmany, J. 2008. Monumental Milestone, *ENR*.

Van Hampton, T. 2007. Minneapolis Bridge Rebuild Draws Fire, *ENR*.

Watson, R.J. 1986. Some Considerations Involved at Points of Bearing in Contemporary Bridges and Structures, *ACI* SP-94-18.

2 Cable-supported bridges

Chapter 7

Potential of multi main-spans suspension bridge

M. Inoue
IHI Corporation, Tokyo, Japan

ABSTRACT: In this paper, some optimal and possible solutions of significant problems for a multi main-spans suspension bridge have been studied using a suspension bridge model with triple main-spans, each of 800 meter length. At first, advantages and disadvantages of multi main-spans suspension bridge are investigated by comparison with a single main-span suspension bridge of the same total length. After an overview of the basic characteristics, the effect of articulations, such as vertical supports of suspended deck at the towers and longitudinal connections, cable slippage resistance at the tower saddles and the deformation of suspended deck are presented followed by a study of constructability of cables and deck erections. Finally, issues of concern regarding safety and quality of construction are discussed and solutions are proposed.

1 INTRODUCTION

In the early 20th century, multi main suspension spans were constructed in the USA, and are still in service today. Though some of these are for vehicle crossing and others are for pedestrian, the length in main span is less than some 60 m for each bridge.

A suspension bridge with multi main-spans attracts a bridge engineer's attention again because of its potential of cost savings, structural efficiency and aesthetic features. The continuous multi main-spans suspension bridges with each main span of more than 500 meters are planned in some countries, and 4 span continuous multi main-spans suspension bridge is now under construction in China with each main span of some 1000 meters in length.

In the past, series of studies for this type of bridge have been carried out by many researchers and engineers. Through their studies, it is well known that the behavior of inside tower plays so important role, especially in the points of the deflection of suspended deck and the cable slippage resistance at the tower top. Also the effect of tower imperfections for the durability as a whole bridge has been investigated and some notes for how to handle the tower imperfections in the design and in the construction were already shown by the author.

However, to take a step forward for the realization, i.e. much competitive to alternative candidates such as a cable stayed bridge, it is of importance to grasp the behavior and the concerning issues to be solved in the design more clearly.

In this paper, some optimal and possible solutions of significant problems for this type of bridge have been studied using the suspension bridge model with triple main spans of each 800 meter length. At the beginning, basic characteristics of multi main-spans suspension bridge have been studied mainly in the points of "Design". The governing factors of towers were investigated through the parametric analyses, and the characteristics of inside and outside towers were shown in comparison with the past single main-span suspension bridges. Then advantageous and disadvantageous of multi main-spans suspension bridge were investigated by comparison with a single main-span suspension bridge with same total length in the points of quantities, deformability and so on. After overseeing the basic characteristics, the effect of articulations, such as vertical supports of suspended deck at the towers and longitudinal connections, to the cable slippage resistance at tower saddles and the deformation of suspended deck have been investigated. Also, the workability in cable and deck erections has been studied. Finally, as a conclusion of this study, the concerning issues and possible solutions are summarized in the points of safety and quality.

2 BASIC CHARACTERISTICS OF MULTI-MAIN SPAN SUSPENSION BRIDGE

2.1 *Bridge model*

The five span continuous multi main-spans bridge was used as a basic analysis model. The bridge configuration is shown in Figure 1. The total bridge length is 250 + 3 @ 800 + 250 = 2900 m, and the sag to span ratio of 1/11 is adopted. The two normal vehicle lanes are assumed in each direction and the deck width is 22.2 m. To compare the results easily with other experiences, the material of tower was assumed to be steel with the yield strength of 355 MPa for all plates.

As a live load, the uniformly distributed load of H20 loading according to AASHTO was burdened on the deck with sidewalk loadings. The total load with vehicle and sidewalk is 35.6 kN/m/bridge. The burdened loads were applied to give the critical conditions for the main cable and the towers, respectively. In this study, only H20 loading is a live load to determine the required section properties of the main cable and the towers. For the load combination, the load factors are 1.00(Dead load), 1.75(Superimposed load) and 1.20(H20) for SLS(Serviceability limit state), and 1.05(Dead load), 1.75(Superimposed load) and 1.50(H20) for ULS(Ultimate limit state), respectively.

2.2 *Governing factors of tower in design*

The governing factors for the tower design were investigated through the parametric analyses by changing the moment inertia of inside and outside towers. As illustrated in Figure 2, the equivalent spring coefficients K_I for the inside tower and K_O for the outside tower are adopted as parameter to see the maximum vertical deflection of suspended deck in the downward direction (denoted as "δ") and the cable slippage resistance at the top of inside tower (denoted as "FOS").

The equivalent spring coefficients K_I and K_O are calculated for the freestanding tower under a longitudinal force at the tower top. The geometrical nonlinearity was taken into consideration for parametric analyses of the whole bridge but not for calculation of these coefficients.

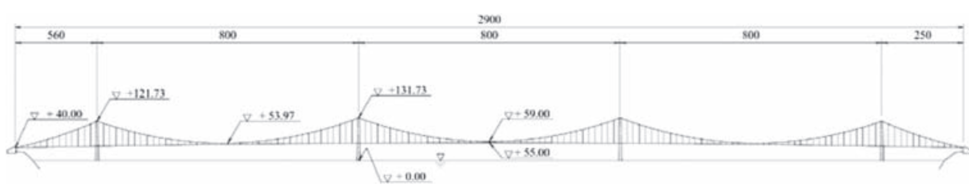


Figure 1. Bridge configuration of multi main-span suspension bridge.

Figure 2. Image of tower springs.

For the cable slippage resistance, FOS can be obtained by using Equation 1.

$$FOS = \frac{\mu \cdot \theta}{\ln(T_1 / T_2)} \quad (1)$$

where "μ" is the friction coefficient between the main cable and the surface of saddle; "θ" is the sum of both cable angles from the horizontal line in radian; and T_i is the tensile force of main cable between the tower.

The analysis results are shown in Figures 3 and 4. Here, the friction coefficient "μ" is assumed as 0.2. In the figures, the horizontal axis stands for the equivalent spring coefficients of tower and the vertical one is for "FOS" in Figure 3 and for "δ" in Figure 4, respectively. The maximum vertical deflection of suspended deck appears in the main span between the inside towers.

Through these results, two things can be noted. First, FOS for the inside tower seems not to have an appropriate value under the live load since more than some 2.0 is most likely required

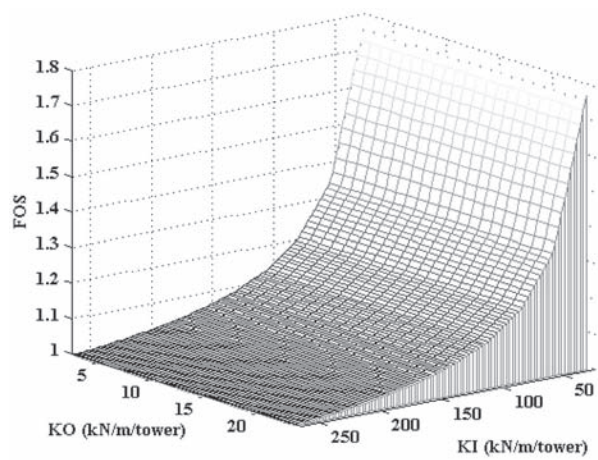


Figure 3. Relationship among the equivalent spring coefficients and FOS.

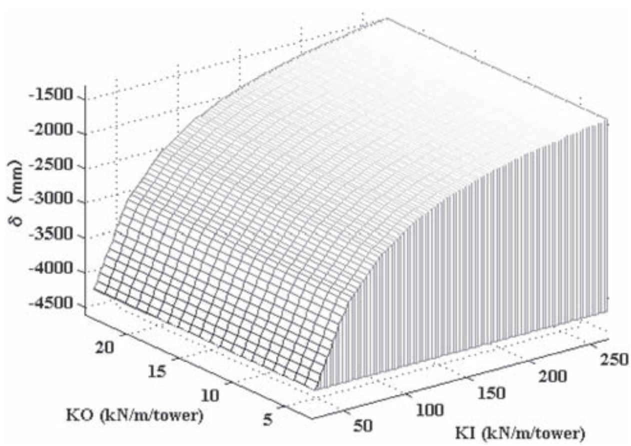


Figure 4. Relationship among the equivalent spring coefficients and the maximum deflection of deck.

for FOS in design. Also, FOS and δ are contrary to each other. More exactly, FOS becomes worse if less δ is required. Thus, the installation of additional system at the tower top saddles is needed, which can increase the friction coefficient between the main cable and the saddle. For example, the system of tower top saddles with a friction plates adopted in Rainbow bridge in Tokyo is effective to improve FOS without changing δ. Of course, the combination with other possible solutions, such as the increment of sag, the installation of stay hangers and so on, may be effective and required.

Second, the inside tower governs the characteristics of FOS and δ, and the outside tower has almost no effect on these properties. Also, the maximum deflection δ is larger than that for the single main-span suspension bridge with the same main span length. To keep the vehicle running stability, the allowable maximum deflection under the live load is generally specified in the technical requirement, e.g. the maximum vertical deflection, the maximum slopes in the longitudinal and the horizontal axes and so on. These requirements are not usually critical for the tower design in the single main-span suspension bridge, but critical for that in the multi main-spans suspension bridge. In this study, the maximum allowable vertical deflection is defined as L/250, where L is the length of main span. Since the main span is 800 m in length in the bridge model, the maximum vertical deflection should be less than 800/250 = 3.2 m. This requires the inside tower with the spring coefficient of some 50 MN/m/tower. In this condition, the required section of tower is basically governed by the required stiffness but not by the stress as same as single main span suspension bridges.

To grasp the characteristics of inside tower more clearly, the result is compared with those for the single main-span suspension bridges, i.e. the bridge model with the same total length as shown in Figure 5 and the past experiences in Japan. The single main-span suspension bridge was designed to have the same ratio of main-span length to side-span one as that for the multi main-spans suspension bridge.

If the tower is considered as a beam fixed at the bottom and simply supported at the top, the behavior of tower including the effect of main cable can be approximately calculated by Equation 2.

$$\frac{Fh^3}{EI \cdot \delta_0} \approx \frac{(\alpha h)^3}{\tan(\alpha h) - \alpha h} \tag{2}$$

where F is the support reaction (see Figure 6); δ_0 is the forced displacement at the tower top; α is the dominant factor defined as $\alpha^2 = V/EI$; and h is the tower height.

In this equation, F < 0 means that the tower is unstable for the condition with the vertical load at the tower top (vertical component of main cable tensile forces) if the main cable does not exist. In Figure 6, the relationship between the non dimensional support reaction and the dominant ratio is shown. The dominant ratio can be expressed by using the dominant factor as follows (Equation 3).

$$\frac{\alpha h}{\pi} = \sqrt{\frac{V}{EI}} \cdot \frac{h}{\pi} = \sqrt{\frac{V}{EI \cdot \pi^2/h^2}} = \sqrt{\frac{V}{V_E}} \tag{3}$$

where V_E is Euler buckling load with hinge connection at both ends.

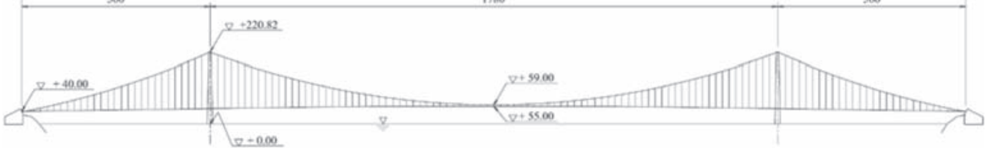


Figure 5. Bridge configuration of single main-span suspension bridge.

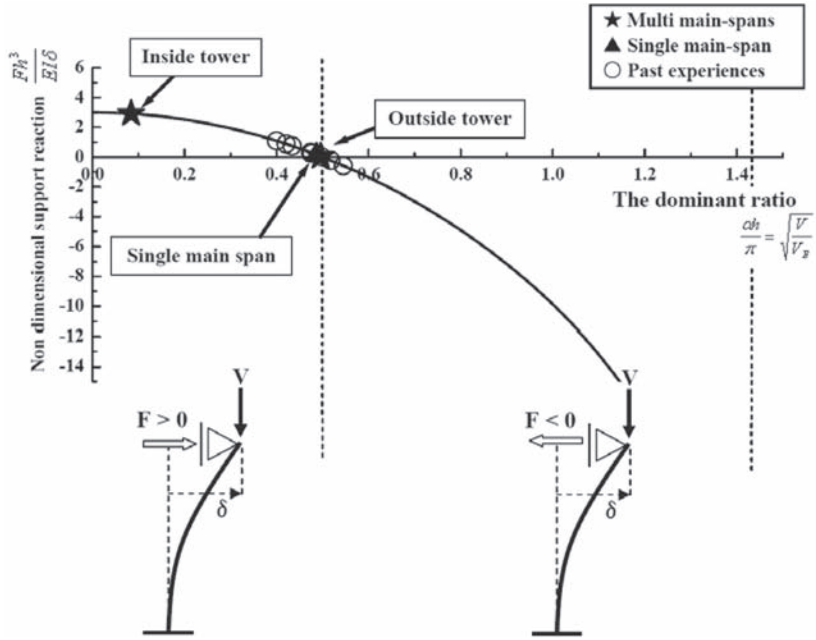


Figure 6. Change of characteristics of tower.

The characteristics of tower can be mainly classified into the following five states;

a. The dominant ratio is less than 0.5
   In this range, the support reaction F is always positive. Thus, the tower acts to prevent the cable horizontal displacement. The maximum bending moment of tower appears at the bottom of tower.
b. The dominant ratio is equal to 0.5
   If the dominant ratio is equal to 0.5, the support reaction F is zero. Thus, the system is in the equilibrium state.
c. The dominant ratio is in the range from 0.5 to 1.0
   If the dominant ratio is greater than 0.5, the support reaction F is always negative. Thus, the tower acts to increase the cable longitudinal displacement. And the location of maximum bending moment moves up with the increment of the dominant ratio, but there is no inflection point in the tower.
d. The dominant ratio is equal to 1.0
   If the dominant ratio is equal to 1.0, the bending moment at the bottom of tower is zero. And the maximum bending moment appears at the middle of tower.
e. The dominant ratio is greater than 1.0
   If the dominant ratio is greater than 1.0, the location of zero bending moment moves up with the increment of the dominant ratio, and the inflection point appears in the tower. Also the vertical force V reaches its critical value V_{CR} when the dominant ratio is 1.430.

The dominant ratios under the own weight and under the maximum vertical load are summarized in Table 1 for a multi main-spans suspension bridge, a single main-span suspension bridge and past experiences. Also those under the own weight are plotted in Figure 6. As is obvious in these results, the dominant ratio for the inside tower of multi main-span suspension bridge is quite low compared with others (thus so rigid), and the singularity of inside tower can be clearly understood.

Table 1. Comparison of tower characteristics.

Bridges (Tower height)	Spring coeff. (kN/m)	Inertia (m^4)	Vertical forces (MN) V_D	V_{max}	Dominat ratio (–) V_D	V_{max}
Multi main-span suspension bridge						
Outside tower (121.7 m)	1,705	5.1	168.3	197.9	0.496	0.538
Inside tower (131.7 m)	50,000	190.5	154.5	183.5	0.084	0.092
Single main-span suspension bridge						
Model (220.8 m)	2,660	47.7	457.1	524.4	0.487	0.521
Past experiences						
Akashi (286.7 m)	3,268	128.4	914.9	992.1	0.545	0.567
Minami Bisan-Seto (184.3 m)	3,453	36.0	281.2	334.0	0.518	0.565
Kita Bisan-Seto (173.8 m)	3,407	29.8	239.3	288.0	0.496	0.544
Shimotsui-Seto (141.1 m)	4,059	19.0	210.4	256.0	0.477	0.526
Ohnaruto (128.3 m)	4,542	16.0	172.6	204.0	0.421	0.458
Kanmon (136.0 m)	1,754	7.4	104.6	126.0	0.518	0.572
Innoshima (138.2 m)	1,682	7.4	85.3	108.0	0.474	0.533
Ohshima (89.8 m)	1,805	2.2	50.3	65.5	0.434	0.496

2.3 *Comparison with single main-span suspension bridge*

In this section, the object is focused on more practical issues, i.e. those related to the initial cost. Through the comparison with a single main-span suspension bridge with a same total length, which is already mentioned in the previous section, the advantageous and the disadvantageous for a multi main-spans suspension bridge were investigated in the required quantities and reactions. Though the total project cost highly depends on its own conditions such as country, navigation, ground and so on, it is so useful to know the tendencies for those which are relatively common among the projects. In the comparison, the main cable and the towers were designed only under the vehicle live load (H20 uniform distributed load) with the material safety factors of 2.20(SLS) and 1.67(ULS) for the main cable and of 1.20(SLS) and 1.05(ULS) for the tower, respectively. The difference of deck and hanger between two models in weight was assumed to be only caused by its length.

The comparison in quantities is shown in Table 2. For the main cable and the hanger, the required quantities of multi main-spans suspension bridge is less than those of single main-span one since the main span length (governing parameter) is 800 m and 1,780 m, respectively. The required quantity of the suspended deck is almost same between two. On the other hand, the towers show a significant feature. In a common sense for a single main-span suspension bridge, the required

Table 2. Comparison in quantities.

Main component		Multi main-spans t/bridge (ratio)	Single main-span t/bridge (ratio)
Main cable		10,375t (100%)	27,415t (264%)
Hanger		269t (100%)	608t (226%)
Tower	Outside	13,877t	
	Inside	67,887t	
	Total	81,764t (100%)	73,147t (89%)
Deck		26,680t (100%)	26,880t (100%)

Table 3. Comparison in maximum reactions at towers and anchorage.

			Multi main-spans MN/bridge (ratio)	Single main-span MN/bridge (ratio)
Tower	Axial force	Inside	208 (100%)	602 (289%)
		Outside	233 (112%)	
	Bending moment	Inside	2840 (6300%)	229 (509%)
		Outside	45 (100%)	
Anchorage (Cable tensile force)			249 (100%)	692 (278%)

quantity of tower is correlated with its main-span length, and this sense can be applicable between the outside tower of multi main-spans suspension bridge and the tower of single main-span one. However, the inside tower of multi main-spans suspension bridge requires almost same quantity as for a single main-span one even though the tower height of multi main-spans suspension bridge is some 0.60 times as that of single main-span one. This is because the inside tower of multi main-spans suspension bridge is designed based on the allowable deflection of deck but not on the stress.

In Table 3, the reactions at the towers and the anchorage are summarized. Though the maximum axial reactions at the towers are quite natural compared among those, the maximum bending moment especially for the inside tower shows remarkable value because the governing factor of inside tower in the design is to control the deflection of suspended deck within the reasonable value. This fact may be a potential of an uneconomical factor for multi main-spans suspension bridge. The effect caused by the larger bending moment at the basement of inside tower seems to be not so critical to the design of tower basement though it surely depends on the ground conditions. In the anchorage, the maximum tensile force of main cable for the multi main-spans suspension bridge is lower than those for single main-span one corresponded to the difference of main span length, and more economical design can be expected.

3 EFFECT OF ARTICULATION

Through the analysis results in the previous sections, the importance of inside tower can be clear especially for the deformability of suspended deck and the cable slippage resistance at the tower saddles. Here, the possible solution with an appropriate balance among the vertical deflection of deck, the cable slippage resistance, durability, the initial and maintenance cost, running comfortable and so on was studied through the analyses with different articulation conditions since these two factors are contrary to each other.

The base model is a 5-span continuous suspension bridge with the vertical supports at both girder ends, which was used for the study in the previous chapters. The following four models were used as alternatives;

Model-1: The continuous deck is fixed in the vertical direction at both ends and at the towers.
Model-2: The deck consists of five simple supported beams.
Model-3: The continuous deck is fixed vertically at both ends as same as the base model and connected in the longitudinal direction to the inside towers by the buffers which act as almost rigid under the vehicle live load.
Model-4: The continuous deck is fixed vertically at both ends as same as the base model and connected in the longitudinal direction to the outside towers by the buffers which act as almost rigid under the vehicle live load.

The minimum cable slippage resistance of saddle at the top of inside tower and the maximum (downward) vertical deflection of suspended deck under the vehicle live load are summarized in

Table 4. Comparison among the bridges with different articulation conditions.

Model type	Minimum cable slippage resistance	Maximum vertical deflection
Base model	1.47 (100%)	–4208 mm (100%)
Model-1	1.22 (83%)	–3200 mm (76%)
Model-2	1.17 (80%)	–3275 mm (78%)
Model-3	1.24 (84%)	–2909 mm (69%)
Model-4	1.29 (88%)	–3221 mm (77%)

Table 4 for five models. Though any alternative improves the deformability of suspended deck but change the cable slippage resistance worse, Model-3 where the buffer is installed at the location of inside tower is much effective among those alternatives. However, the base model seems to be more feasible because the cable slippage resistance is much critical for this type of bridge and less articulation can make the minimization of maintenance cost easily.

4 BEHAVIOUR DURING THE CONSTRUCTION

4.1 *Cable erection*

In the cable erection, to keep a sufficient cable slippage resistance is one of the most important issues to achieve the main cable with high degrees of accuracy. For this purpose, frequently the tower is set back by using the temporary rope, or the tower top saddle is shifted relative to the tower before starting the cable erection. In case that the saddle is set back, the saddle needs to be resumed back toward the centre of the tower. Since the required horizontal force to resume the saddle back to its design position increases with the progress of bridge construction, the resume back operation in the earlier construction stage is desirable. In a multi main-spans suspension bridge, the behavior and the characteristics of main cable at the outside tower are so similar to those for a single main-span suspension bridge, thus the behavior of main cable at the inside tower was especially studied here.

In this model, it was assumed that the outside tower is set back by the temporary rope and the saddle of inside tower is shifted relative to the tower during the cable erection. There is another solution such as set-back for both inside and outside towers, but the adopted solution seems to be better since the inside tower is so rigid to be set back and can be expected to locate in the navigation sensitive area (the additional equipment with a potential of accident such as stay rope shall be avoided as much as possible). If the shift condition is determined as the tensile forces of main cable between the inside tower are in equilibrium condition in the longitudinal component, 195 mm of longitudinal shift toward the anchorage is required for the saddle at the top of inside tower and 595 mm of set-back in longitudinal direction toward the anchorage is required for the outside tower.

This condition leads FOS of 33.4 for the cable slippage resistance at the top of inside tower during the cable erection (the friction coefficient between the main cable and the tower saddle was assumed as 0.2), and the stable cable erection can be carried out. The cable slippage resistance changes 7.6 if the tower saddle at the inside tower is resumed back to its design position after the cable completion. Since FOS is sufficiently high in this model, the resume-back operation can be done in this stage and less equipment to move the tower saddle is needed compared with that in the later stage. Of course, much cable slippage resistance is obtained if the additional device to increase the friction coefficient is installed before the resume-back operation.

4.2 *Deck erection*

During the deck erection, the cable slippage resistance is more critical than that both during the cable erection and in service even if the additional device to improve the friction coefficient between the main cable and the saddle is installed.

Now the situation of lifting the first deck unit is considered in the model. If the tower saddles are set in the design position, FOS at the saddle of inside tower are 1.0 for lifting the deck unit at mid centre and 0.8 for lifting at the centre between the inside and the outside towers, respectively (the friction coefficient was 0.2). On the other hand, if the saddle at the top of inside tower is shifted to keep an appropriate cable slippage resistance, more than 250 mm shift is required to lift the first deck unit at the mid centre and more than 1100 mm for lifting at the centre between the inside and the outside towers, respectively. Though lifting two symmetrical deck units between the inside tower simultaneously is one of possible solutions, practically the followings seem to be a better solution;

- As the first lifting unit, the deck unit which requires less amount of the tower saddle shift to keep an appropriate cable slippage resistance shall be selected. If the required shift can be same between for the cable erection and for the first lifting unit, it is desirable and much advantageous in the point of workability. In this model, the first unit shall be in the main span between the inside towers.
- Subsequent lifting shall be done symmetrically with respect to the inside tower to keep an appropriate balance in weight of erected unit among the main spans.
- If the cable slippage resistance can keep a sufficient value by increasing the weight of erected units during the following deck unit lifting, the tower saddle shall be resumed back to its design position.

5 CONCLUSIONS

In this study, a potential of multi main-spans suspension bridge has been investigated for further applications as one of attractive solutions for a long span bridge with safety, reasonable cost and elegant feature. It was shown that the inside tower is a key component and plays so important role not only to the bridge performance but also to the project cost. The required characteristics of the inside tower is most likely governed by the deformability of deck or the vehicle running stability on the deck. In this situation, the equivalent spring coefficient of tower in the longitudinal direction is a major factor in design and the critical deformability can be almost controlled by this factor. Thus, the minimum requirement for the inside tower is to have an appropriate equivalent spring coefficient and the designers can bring their ability into full play within the requirement based on the project concept. It is also noted that the additional equipment or alternatives needs to be installed to the saddle at the top of inside tower to keep a sufficient cable slippage resistance.

In the construction, the special attention should be paid to keep an appropriate cable slippage resistance especially during the deck erection. Of course there are many minor concerning issues to be solved in addition to the cable slippage resistance, however those are not so serious and seems to be relatively easy to find the optimal solution for each project.

REFERENCES

Honshu-Shikoku Bridge Authority. 1989. Design instruction of tower for suspension bridges.

K. Kloppel et al. 1965. Die Berechnung eingespannter und fest mit Kabel verbundener Hangebrucken pylonen bei Beanspruchung in Bruckenlangsrichtung. Stahau H.

M. Inoue et al. 2008. Durability of suspension Bridge with multi main spans. In Hyun-Moo Koh & Dan M. Frangopol (eds), *Bridge Maintenance, Safety, Management, Health Monitoring and Informatics*, proceedings of the fourth international conference on bridge maintenance, safety and management, Seoul, Korea, 13–17 July 2008.

K. Takena et al. 1992. Slip behavior of cable against saddle in suspension bridges. J. Struct. Engrg. ASCE, pp. 377–391.

Y.J. Ge & H.F. Xiang. 2008. Bridging capacity innovations on cable-supported bridges. In Hyun-Moo Koh & Dan M. Frangopol (eds), *Bridge Maintenance, Safety, Management, Health Monitoring and Informatics*, proceedings of the fourth international conference on bridge maintenance, safety and management, Seoul, Korea, 13–17 July 2008.

Chapter 8

Kanchanaphisek Bridge: A cable-stayed bridge designed for a low maintenance budget

R. Hsu & P. Vineyard
Parsons Brinckerhoff, New York, USA

ABSTRACT: The Kanchanaphisek Bridge, with a 500 m main span, is the bridge with the longest span in Thailand. The bridge was designed to be long lasting, owner friendly, require minimal maintenance and provide access to all critical structural components for future inspection and maintenance. Simple low cost ladders and platforms allow ready use access to the stay cable top and bottom anchors for inspection and maintenance. Concrete counterweights, instead of mechanical tie-downs, were used to eliminate uplift at the anchor piers of the cable-stayed bridge. The bridge's integral anchor piers and innovative "floating" connection at the towers eliminated the need for bearings on the entire bridge. Therefore the need for costly bearing inspection and replacement has been eliminated.

1 INTRODUCTION

On November 15, 2007, Kanchanaphisek Bridge crossing the Chao Phraya River in south Bangkok, as shown in Figure 1, opened to traffic. This bridge is a major part of the Southern Outer Bangkok Ring Road (S-OBRR) Project, which is a 21 km long viaduct to complete the outer ring road around Bangkok.

Figure 1. The Kanchanaphisek Bridge crossing the Chao Phraya River in Bangkok, Thailand.

The Department of Highways (DOH) of Thailand retained a consultant team to provide the feasibility study and design services for the S-OBRR Project. PB Asia, the local office in Thailand of Parsons Brinckerhoff, Inc. (PB), was responsible for the Chao Phraya River crossing. PB's New York office provided technical support to PB Asia.

A feasibility study comparing bridge and tunnel for the Chao Phraya River Crossing was performed in 1996. The study selected a cable-stayed bridge with 500 m main span over the tunnel alternative.

The detail design of the S-OBRR started in 1999. The Chao Phraya River Bridge design team was led by PB, with support from other Thai consulting companies. The design was successfully completed in 2000.

At the start of the design, three goals for this cable-stayed bridge were established:

- Design a long lasting owner friendly bridge that has minimal maintenance
- Provide access to all critical structural components for future inspection and maintenance
- Design a signature bridge for Bangkok, the capital city of Thailand.

Construction of the bridge began in 2004. The general contractor was Ch. Karnchang (CKC), which is one of the biggest contractors in Thailand. Jean Muller International (JMI) was the construction engineer supporting CKC. Freyssinet supplied and installed all stay cables. PB provided technical support during the construction. The construction of the bridge was completed in November, 2007. King of Thailand named this project to be Kanchanaphisek.

2 GENERAL DESCRIPTION OF THE BRIDGE

The Kanchanaphisek Bridge crosses the Chao Phraya River with a 500 m main span, which is the longest span in Thailand and the 18th longest cable-stayed bridge in the world. (Figure 2) Its 36.7-m-wide superstructure carries three traffic lanes in each direction. The bridge provides 50.5 m vertical underclearance for the busy marine traffic to and from the Khlong Toey Harbor upstream of the bridge.

This bridge was designed in accordance with the AASHTO Standard Specifications for Highway Bridges (US), Sixteenth Edition, 1996. The design live load is 1.3 × AASHTO HS20 or HS26 truck and lane loads.

2.1 *Tower*

The Kanchanaphisek Bridge includes two 187-m-high A-shaped towers. (Figure 3) The tower legs are hollow reinforced concrete structures. A horizontal strut, just below the deck level, provides lateral support to the slender legs. The two legs join at the top, forming a chamber to house the

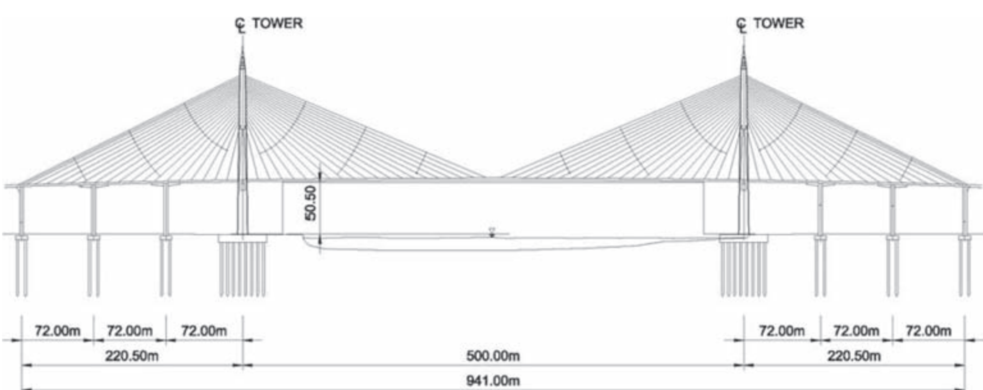


Figure 2. Bridge elevation.

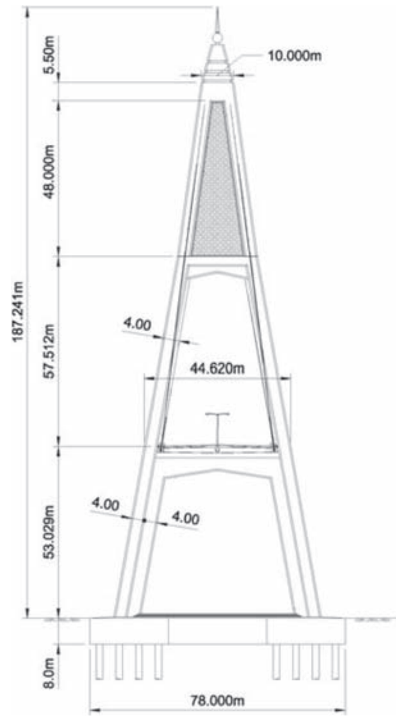


Figure 3. Tower elevation.

cable anchors. The A-shaped towers combined with three anchor piers in each back span provide maximum vertical, lateral and torsional stiffness of the bridge superstructure. The high stiffness enables the bridge to resist higher wind speeds.

A decorative 3 m diameter stainless steel sphere and an 8 m tall spire are on top of each tower. They are specially designed in traditional Thai style and are painted in gold color. Convenient access to the interior spaces of the sphere and spire has been provided for inspection and these elements are expected to last as long as the tower without any rehabilitation or replacement. The two towers are located on land to provide an uninterrupted shipping lane and avoid hazards of vessel impacts to the tower legs. The west tower is located behind the existing wharf to keep 100 m long wharf for harbor operation. Twenty five 2 m diameter drilled shafts support each tower leg. A reinforced concrete beam ties the two individual tower leg footings together.

2.2 *Superstructure (Deck)*

The superstructure is comprised of a pair of steel box shaped edge girders, built-up I-shaped steel floorbeams spaced at 4 m intervals and a reinforced concrete deck slab. (Figure 4) The deck is made composite to the steel frame. The thickness of the concrete deck slab varies from 260 to 310 mm. Thicker slab segments extend 98.5 m on both sides from the tower to resist higher compression induced by the stay cables.

The superstructure is supported by stay cables at 12 m spacing along the edge girders. Conventional cable-stayed bridges use bearings to support the superstructure at towers. However, the Kanchanaphisek Bridge “floats” the superstructure using a pair of cables, instead of bearings, to support the superstructure at tower. (Figure 5) In addition to eliminating the need for bearings, this design greatly reduces negative bending moment in the edge girders at the towers.

Figure 6 illustrates a typical box shaped steel edge girder. A typical edge girder is 1.5 m high to allow people to work comfortably inside. The flanges and webs designed so that neither transverse

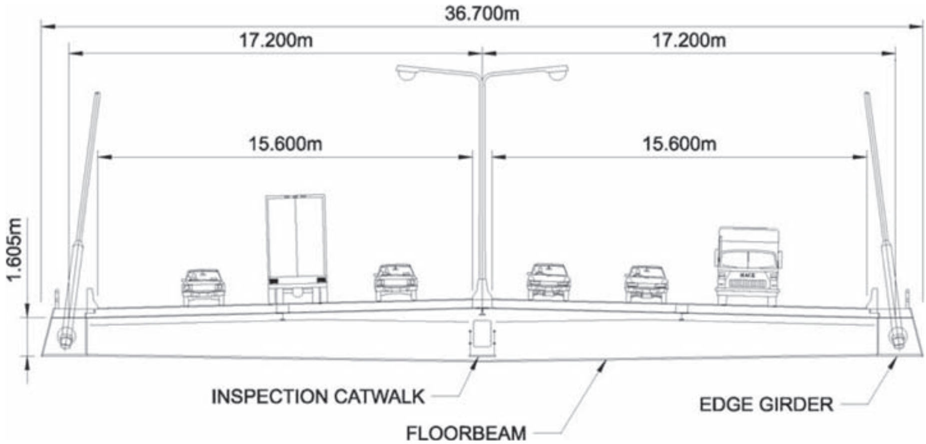


Figure 4. Typical superstructure cross-section.

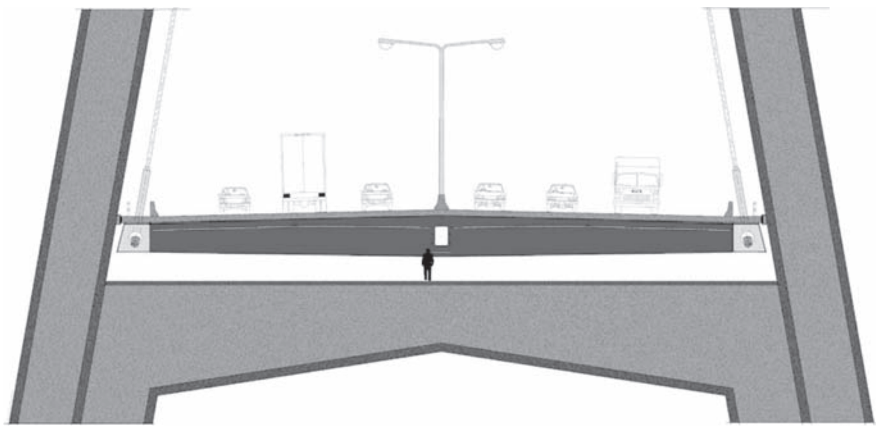


Figure 5. Superstructure "floating" at tower.

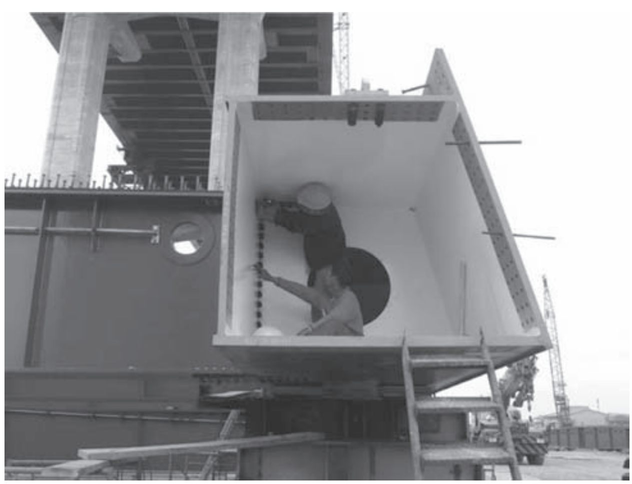


Figure 6. The edge girder box was designed for people to work comfortably inside.

nor longitudinal stiffeners are required. The edge girder was fabricated in 12 m segments. Cable anchors are located inside the edge girder to protect them from rain.

2.3 *Stay cables*

The superstructure is supported by 168 stay cables. Cables comprise of strands, which are 15-mm-diameter, low relaxation, seven wire weldless strands conforming to ASTM Specification A416-90a Grade 270. (Figure 7)

The strands are galvanized, coated with wax, individually sheathed within a layer of polyethylene (PE). Parallel strands in a bundle are then placed in a high-density polyethylene (HDPE) pipe for aerodynamic purposes. White color HDPE pipes are used to lower temperature in the strands due to solar heating. Spiral bars are welded on the exterior surface of the HDPE pipes to control cable vibration caused by wind and rain interaction. The 500 m span requires long cables that are prone to large amplitude vibrations. Cross ties, which are the most economical and effective method to control cable vibration, were included in the original design. (Figure 2) However, hydraulic dampers were installed on the longer cables due to construction access issues for installing the cable cross ties. (Figure 8)

2.4 *Anchor pier*

The bridge is supported by three anchor piers in each back span. Each of the anchor piers consist of double columns. The pier columns are 4-m × 4-m hollow reinforced concrete structures supported by 2 m diameter bored piles. The two columns are tied on top to provide lateral stability. (Figure 9) Cantilever arms extend longitudinally from the top of each column. These arms carry the weight of the dead load counterweights before the anchor cables are installed and the weight of the live load counterweight. The anchor piers are made integral with the superstructure through the counterweight concrete. This design increases superstructure stiffness, reduces bending moments in the edge girders and eliminates the need for bearings.

Figure 7. Stay cable strand is pulled into the PE pipe.

Figure 8. Hydraulic dampers.

Figure 9. Underside of the superstructure at anchor piers showing, catwalk, platforms, and floorbeams. Counterweights are located at the tip of the cantilever arms.

3 DESIGN CONSIDERATIONS TO REDUCE MAINTENANCE

3.1 *Reinforced concrete only*

Most cable-stayed bridges use post-tensioning in the deck and tower concrete elements. However, it is difficult to repair or replace post-tensioning in a concrete structure. The Kanchanaphisek

Bridge is a major structure that will be a vital transportation artery of Bangkok for at least the next 100 years. Only the time proven reinforced concrete was used to construct this bridge, with one exception. During construction, post-tensioning was added to the tower foundation to keep the tie beam in compression. The sole purpose of this post-tensioning is to prolong the life span of the tie beam, which is permanently submerged.

3.2 *Concrete counterweight*

To keep the anchor cables in tension at all times, most cable stayed bridges are designed to have main span heavier than the back span. A consequence of this design is uplift at the anchor piers. To resist the uplift, a tie-down device is connected between the superstructure and the anchor pier. The weight of the anchor pier is then mobilized to resist the uplift force. The tie-down devices are usually a steel rod, a cable, a pin, etc. Because the tie-down device is non redundant, failure to a single tie-down can cause the bridge to collapse. Reliance on non-redundant critical components of a bridge should always be avoided whenever possible. Another common problem with a mechanical tie-down device is its proximity to the expansion joint. Since the bridge expansion joints are also commonly at the anchor pier, numerous cable-stayed bridges place the tie-downs directly beneath the expansion joint. Water will eventually leak from the deck expansion joint onto the tie-down device and will ultimately accelerate corrosion of this critical element. Inspection and maintenance of the tie-down devices are usually difficult due to limited and difficult access. If a tie-down device is found to be deficient, replacement of the device will be a major effort if even possible at all.

The Kanchanaphisek Bridge does not have a mechanical tie-down system. Instead, concrete counterweights, as shown in Figure 9, were made integral with the superstructure and sized so that the bridge has no uplift during the worst loading case. This design is very simple and reliable. The hidden concrete counterweights of the Kanchanaphisek Bridge do not require inspection or maintenance and they are redundant. A significant amount of time and resources will be saved over the lifespan of the bridge by using the concrete counterweight system instead of a tie-down device.

3.3 *Easily replaceable horizontal bumper*

Four horizontal bumpers at the towers, as shown in Figure 10, transfer wind and seismic loads from the superstructure to the towers. The bumpers are made of a very simple elastomeric pad. They are

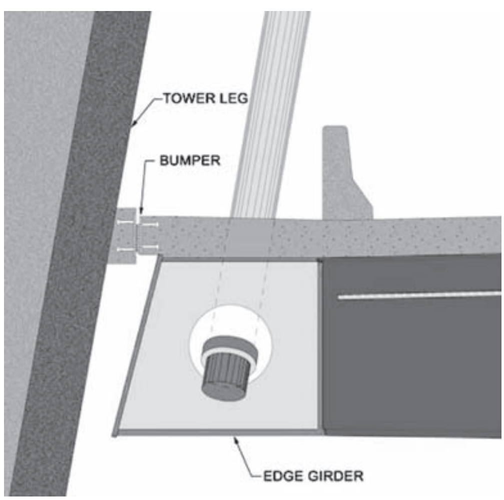


Figure 10. Horizontal bumper.

designed to be light weight so that each bumper can be removed by a single person with only hand tools. The bumpers can be accessed directly by standing on the deck behind the barrier.

3.4 *Integral anchor pier design*

Expansion bearings and wind locks require frequent inspection and maintenance. The anchor piers of the Kanchanaphisek Bridge are made integral with the superstructure, which eliminates commonly used expansion bearings and wind locks. The multiple anchor pier design also greatly reduces typical high stress ranges in the anchor cables. This will lessen the possibility of cable damage due to fatigue, which will ultimately reduce the requirement for future cable replacement.

3.5 *No bearing*

Bridges normally need bearings to transfer vertical and horizontal loads from the superstructure to the substructure. High capacity pot bearings are usually required for a long span cable-stayed bridges. The pot bearings require regular inspections, maintenance, and they will eventually wear out and need replacement. Removing and replacing a large and heavy bearing at over 50 m in height is a major task. With the goal of reducing maintenance in mind, the Kanchanaphisek Bridge's integral anchor piers and "floating" connection at the towers require no bearings at all. This design eliminates costly replacements and time consuming inspections of the bearings.

3.6 *Deck replacement*

A bridge deck is continuously abused by traffic for the entire life span of the bridge. In spite of all precautions, the deck slab will deteriorate. The only unknown is when the deck will deteriorate to a point that a replacement becomes required. Since the deck slab in a cable-stayed bridge is in compression, failure of the deck will cause failure of the bridge. Compression transfer and the bridge stability must be maintained when any deck area is removed. The deck replacement technology has not been fully developed and there is no cable-stayed bridge in the world has its deck replaced so far.

The deck slab of most cable-stayed bridges is post-tensioned. Tendons are usually very long, which will require total or a large area of deck to be removed before the new deck can be constructed. Providing stability of the bridge with no deck is very difficult, if not impossible. A bridge deck without post-tensioning provides opportunity of removing and replacing a small deck area at a time. This will keep the bridge stable all the time. Since the deck of the Kanchanaphisek Bridge has no post-tensioning, it is possible for the deck to be replaced in the future.

3.7 *Deck overlay*

The original design specified the use of high performance concrete as the deck overlay. During construction, the client elected to use an asphalt concrete overlay. The high performance concrete overlay is a more durable and longer lasting material, but requires a high degree of workmanship when compared to the asphalt concrete overlay. Since the client was more comfortable with the installation of the asphalt material, the overlay was changed.

3.8 *Superstructure painting*

The design of the steel floorbeams specified the use of weathering steel to eliminate painting in the future. The steel plate used for the webs and flanges followed the specifications, but due to unfamiliarity with weathering steel in Thailand, the fabricator was found using regular steel consumables for welding at the early stage of the fabrication. Substantial lead time was required to acquire the correct weathering steel weld consumable, which would have drastically affected the construction schedule. As an alternative to replacing the weld material, the contractor elected to paint all the floorbeams. Had the original design been implemented, the substantial effort of repainting the numerous floorbeams would have been eliminated.

4 PROVIDE ACCESS TO ALL CRITICAL STRUCTURAL COMPONENTS

4.1 *Easy access to cable lower anchors*

Designing access to the box girder under the deck slab is always very difficult. A unique access system of the Kanchanaphisek Bridge has solved this problem.

All of the cable lower anchors can be reached from ground level by taking an elevator or climbing a ladder inside the tower leg to the tower horizontal strut, which is directly below the superstructure. A catwalk has been installed along the full length of the bridge and passes through manholes at the midspan of each floorbeam. (Figure 9) The catwalk can be accessed from the tower strut. From the catwalk, the lower cable anchors can be accessed by walking along the widened bottom flange of the floorbeam to a platform with a safety railing installed at the edge girder. (Figure 11) The steel handrail attached to the web of the floorbeam is available for holding and connecting a safety harness attachment. (Figure 12) The cable anchorage can then be accessed through a manhole in the web of the box shaped edge girder. The manholes have be located in a manner to provide access to the font and back side of every cable anchor. This access design helped erection of the edge girders, the installation of the stay-cables during construction and it will benefit any future cable replacement efforts. In addition, the manholes provide sufficient ventilation while working inside the box edge girder.

The cable lower anchor access system was designed to be ready for use, permanently. In case of emergency, inspection and repair can be done immediately instead of awaiting for special equipment and/or falsework to be installed.

4.2 *Easy access to cable top anchors*

Easy access to the cable anchors on top of the tower has been neglected for decades. Architectural driven design has made it more difficult. However, the architect for the Kanchanaphisek Bridge had worked together with the structural engineer to create a design of an architecturally elegant bridge with easy access to the cable top anchors.

A large chamber on top of the tower was designed to house cable anchors. The chamber is 8.8 m × 3.4 m at top and 23 m × 5 m at bottom. The spacious chamber provides ample space for ladders and platforms to directly access all cable top anchors as shown in Figure 13.

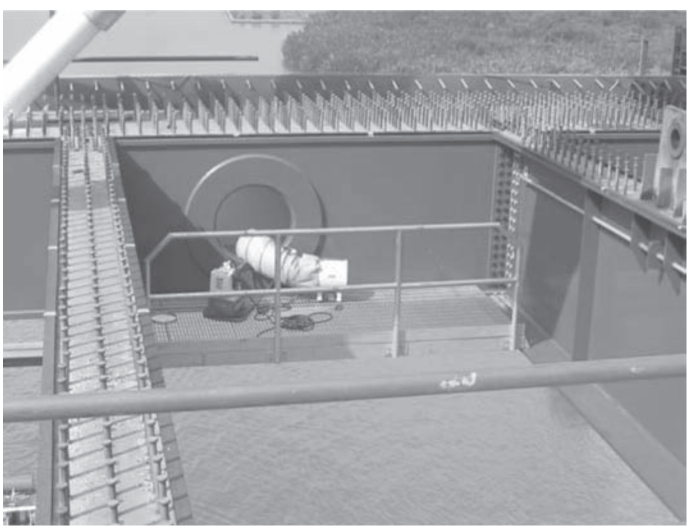


Figure 11. Edge girder can be accessed by walking on the floorbeam bottom flange to the platform.

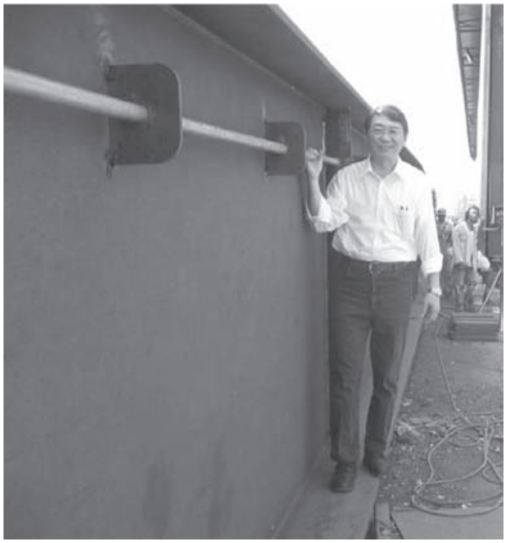


Figure 12. Walking on top of the floorbeam bottom flange is very comfortable and safe.

Figure 13. Large chamber on tower top allows cable anchors to be readily accessed by ladder and platform.

Figure 14. Cable connection frame inside the tower provides a convenient work platform for cable installation.

Steel cable connection frames were used for cable installation without using falsework as shown in Figure 14. This can also be used for cable replacement.

A 10 ton capacity lifting lug ring was installed under the top slab of the chamber and a 1.5 m × 1.5 m opening is located in the bottom slab. The lifting lug system was designed to assist in future maintenance and cable replacement. During the future cable replacement operation, a winch could be attached to the ring and would be used to lower the old cable anchors through the bottom slab opening directly to a truck parked next to the median barrier at the centerline of the tower.

New cable anchors can be lifted up directly from a truck to their final positions. This design also enables heavy equipment or materials to be lifted directly from the deck to the cable anchors for future inspection and maintenance. This unique design will simplify the inspection, maintenance and cable replacement operations.

4.3 *Safe box interior space*

The edge girder is box-shaped. The interior of a traditional box is cluttered with transverse and longitudinal stiffeners. The stiffeners on the webs and flanges are hazards to the inspectors working

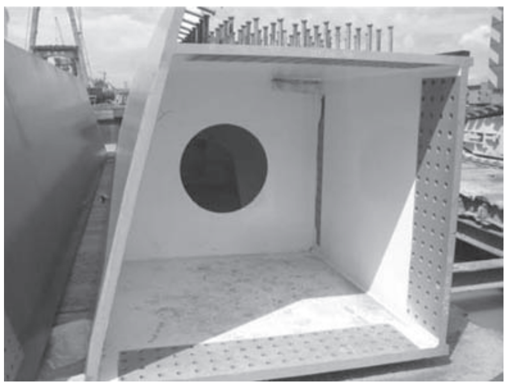


Figure 15. Clean space without any stiffeners on the webs or flanges is safe for people working inside the edge girder.

Figure 16. Elevation of the Competed Bridge.

inside the box. The edge girders of the Kanchanaphisek Bridge have no stiffeners inside the box and large manholes have been installed in the diaphragms. (Figure 15) This design provides a clean and safe working space for construction, inspection and maintenance. In addition, the more robust box girders will last longer.

5 CONCLUSIONS

The Kanchanaphisek Bridge, Figure 16, will be easy to maintain due to the elimination of many frequently used but troublesome elements, such as tie-downs, expansion bearings, and wind locks. All major elements will be easily accessible without the need for special equipment. Easy access for inspection and the low maintenance design of the bridge will keep the inspection and maintenance budget of the bridge owner to a minimum.

REFERENCES

Hsu, R. Design and Construction of the Chao Phraya River Cable-Stayed Bridge. International Bridge Conference, Pittsburg 2007.

Hsu, R. Innovative Design of A Cable-Stayed Bridge to Minimize Maintenance. National Steel Bridge Alliance Conference, New Orleans 2007.

Chapter 9

Design-build of the Indian River Inlet cable stayed bridge

K.V. Butler
AECOM, Glen Allen, Virginia, USA

D. Robb
Delaware Department of Transportation, Delaware, USA

P. Halvarsson
Skanska USA Civil Southeast, USA

ABSTRACT: This paper presents the design aspects for the Indian River Inlet Bridge Replacement design-build project in Delaware, USA. The new bridge will carry the SR1 Coastal Highway across the Indian River Inlet. The roadway includes four lanes of traffic with shoulders, a 12 ft sidewalk and a sand bypass system. The main span unit consists of a concrete cable-stayed bridge with a 950 ft main span and 400 ft back spans. The superstructure components include edge girders, floor beams and a concrete slab. It is supported by two vertical planes of stay cables anchored in the edge girders. The stay cables are anchored in two vertical reinforced concrete pylons using structural steel anchorage boxes to resist longitudinal tensions across the pylon section. The floor beams and edge girders are post-tensioned, as well as the top slab in the longitudinal direction in the vicinity of the transition piers and center portion of the main span. The foundations for the main span unit consist of 36 inch × 36 inch prestressed concrete piles. The deck is fixed for longitudinal movements at the North pylon and free to move at the South pylon. The deck is guided laterally at both pylons. Expansion joints are located at the transition piers and abutments. The cable-stayed spans are built on falsework over land and in cantilever with a traveling form for the portion of the main span located over the Inlet. The paper will discuss several innovative design-build features. Some of the innovative features that will be presented include: large (36") driven piles, which proved to be the least cost and most effective foundation for the challenging soils; innovative edge girder transitions through the pylons minimized the overall footprint of the structure; single mast pylons with no cross struts; and use of cast-in-place and precast components to optimize design and construction efficiencies.

1 SITE DESCRIPTION AND GENERAL BACKGROUND

The site lies on a barrier island bounded by the Atlantic Ocean on the east and Rehoboth and Indian River Bays on the west as shown in Figure 1. This barrier island is part of the 2,825-acre Delaware Seashore State Park. State Route 1 (SR1) travels down the barrier island, connecting the towns of Rehoboth Beach to the north and Bethany Beach to the south of the Indian River Inlet. Dunes and beaches dominate the landscape to the east of SR1, while tidal marshes and wetlands are located to the west.

Until 1928, the Inlet functioned as a natural inlet, shifting up and down the coast over a 2-mile range. Between 1928 and 1937 the Inlet was kept open by dredging, and in 1938, the U.S. Army Corps of Engineers constructed parallel jetties to create a stable 500-foot-wide inlet that provided navigation for recreational boats. In 1941, depths throughout the inlet were less than 20 feet below mean low water (MLW). The inlet scoured both its bed and unprotected banks, increasing the average depth by about 0.5 foot per year. By 1974, some holes were deeper than 40 feet. From about 1975 on, the scour rate increased to about 1 foot per year, and in 1991 nearly the entire inlet was deeper than 40 feet with some holes over 100 feet deep. As can be seen in Figure 2, current scour

Figure 1. Site location of Indian River Inlet and vicinity.

Figure 2. Inlet water depth contour, based on 2007 hydrographic survey.

holes are approaching 105 feet deep in some locations. Essentially, scour depths of 20 to 80 feet have occurred since 1941.

Scour was not the only negative impact that resulted from constructing a stable inlet. Severe beach erosion between the north and south side of the inlet is also occurring. As can be seen in Figure 2, the beach on the north side of the inlet is eroding as a result of the inlet jetties that were constructed. In 1990, beach sand bypassing was begun, passing on average about 100,000 cu yd per year from the south beach to the north. Figure 3 illustrates the sand bypass system which consists of a pump house on the south side of the inlet and a pipeline that runs across the existing bridge discharging on the north side of the inlet.

The first bridge over the Inlet was a timber bridge constructed in 1934, followed by a concrete and steel movable swing bridge built in 1938. This bridge lasted until 1948 when it was destroyed by

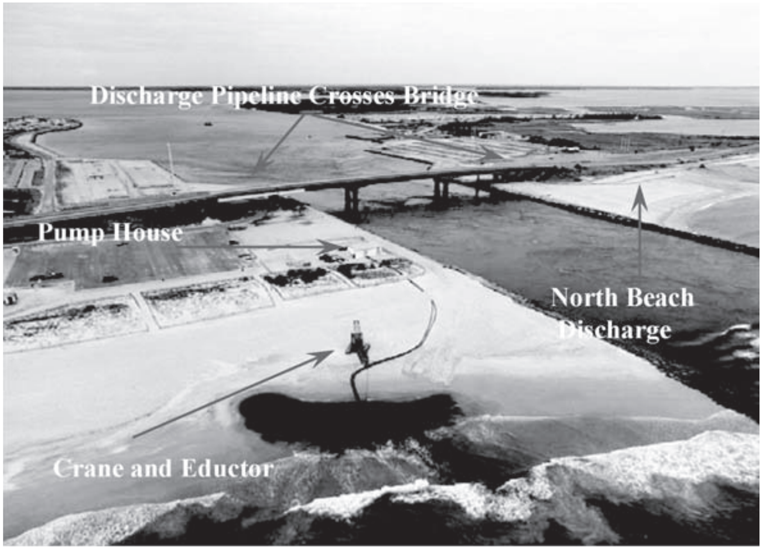


Figure 3. Sand bypass system.

Figure 4. Rendering of new cable-stayed bridge over the Indian River Inlet.

ice flow and extreme tides. Another concrete and steel swing span bridge built in 1952 lasted until the current steel girder bridge was built in 1965. The current 860-foot long bridge was widened in 1976. Scour around the existing SR 1 bridge piers constructed in 1976 (two piers located at about the one third points across the channel) led to 1989 placement of a riprap blanket 3 to 6 feet thick and 300 feet wide from bank to bank around the piers. Design velocity for the riprap was 15 fps.

The new cable-stayed bridge will be the fifth bridge over the inlet in just over 60 years. The history of scour of the previous bridges over the inlet led the Delaware Department of Transportation (DelDOT) to restrict placement of any piers in the inlet. Additionally, DelDOT imposed a 900-foot horizontal clearance requirement to accommodate the potential future widening of the inlet from 500-foot to 800-foot. These two criterions led the design-build team of

Skanska USA Civil Southeast and AECOM USA to develop an economical and aesthetically pleasing bridge solution that consists of a 950-foot long cable-stayed main span over the inlet. Figure 4 illustrates a rendering of the new bridge that is currently being designed and constructed. This paper presents the basic geometrics of the new bridge and some of the key design-build elements.

2 BRIDGE TYPE, SIZE & LOCATION

The primary objective during the bridge tender phase was to evaluate the most economical, constructible and aesthetic bridge alternatives for the Indian River Inlet Bridge Replacement project. The bridge type, size and location was developed based on site specific geometric requirements, efficient and cost effective span lengths, structural performance, constructability, maintainability and inspectability considerations. As a first step, optimum span configurations were determined based on site constraints, design criteria, design codes, bridge horizontal alignment, vertical profile geometry, and deck cross section requirements as provided by DelDOT in the Scope of Services Package. The end result of the design-build teams' evaluation was that a 2,600-foot long structure consisting of a 3-span cable stay bridge (400'–950'–400') with flanking 425-foot approach spans on both sides would provide the best solution. The cable stay superstructure consists of cast-in-place (cip) concrete edge girders with both precast and cip concrete transverse floor beams, and a cip concrete deck. The four approach spans, on each side of the cable stayed unit, are all 106-ft long and consist of 70" deep prestressed concrete Bulb T girders. There are four expansion joints on the structure. Armored strip seal joints are used at each abutment and galvanized steel tooth joints are used at the transition piers between the cable stayed spans and the flanking Bulb T girder approach spans. The General, Plan & Elevation of the bridge is shown in Figure 5.

Figure 6 illustrates the roadway cross section which includes four lanes of traffic with shoulders, a 12 ft sidewalk and a sand bypass system. The cable-stayed superstructure is supported by two vertical planes of stay cables anchored in the edge girders. The overall depth of the superstructure is 6-ft, which compliments the relatively low-level structure profile over the Inlet.

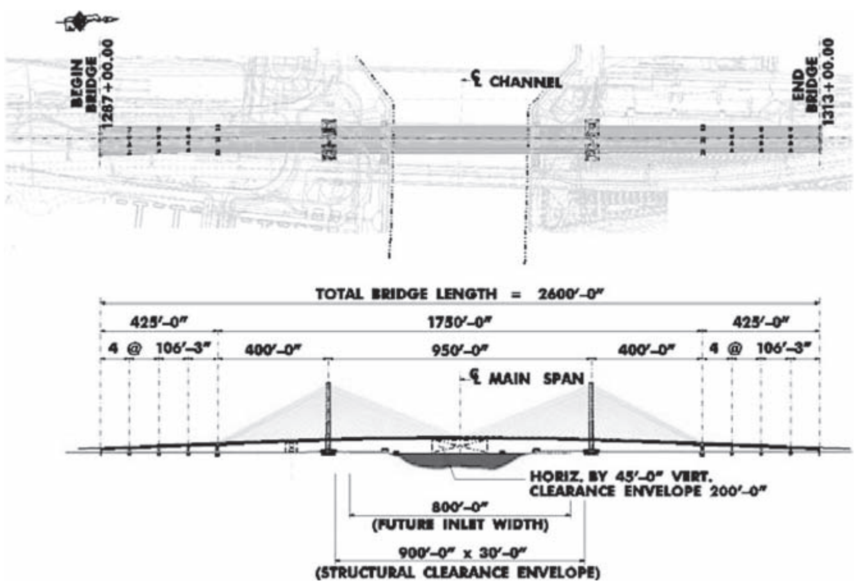


Figure 5. General, plan & elevation.

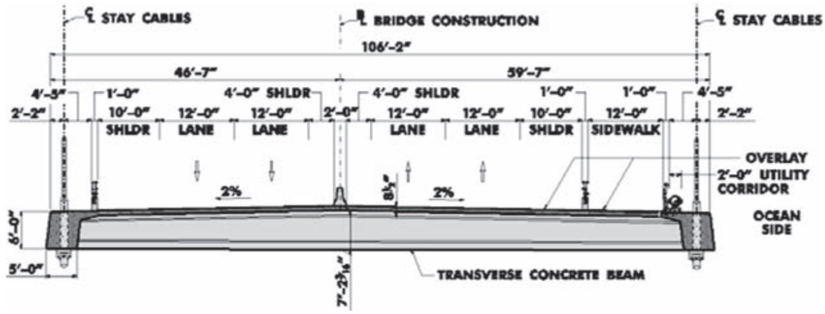


Figure 6. Typical cross section, cable stay main span.

3 KEY DESIGN ELEMENTS

3.1 *Design criteria*

In order to achieve a 100 year design service life DelDOT specified: high performance and low permeability concrete in the superstructure and substructure (maximum permeability, 1500 coulombs); 1 5/8" Latex Modified Concrete deck overlay; 2-inch concrete cover in the deck; epoxy coated reinforcing steel for entire structure; and no tension at the Service Limit State after losses for effective prestress and permanent loads. The superstructure must satisfy the allowable stresses for "fully prestressed" components included in the *AASHTO LRFD Bridge Design Specification* for both longitudinal and transverse directions. In addition to the allowable stresses for the AASHTO Service Limit States, the superstructure tensile stress limit after losses is "no tension" for an additional Service Limit State Combination consisting of the sum of effective prestress, permanent loads and the long term effects from creep, shrinkage and relaxation. The "no tension" tensile stress limit applies in both the longitudinal and transverse directions of each component. Finally, a detailed Corrosion Control Plan is required by the specifications to ensure that the specified service lives for each structural component are achieved and that the 100-year service life is realized.

3.2 *Scour*

The technical approach for the coastal hydraulic and scour study includes: determining design storm characteristics (both Hurricanes and Northeasters); developing and coupling storm surge (ADCIRC) and wave (SWAN) models; modeling storm events; analyzing storm event frequency-of-occurrence relationships with empirical simulation technique; and performing bridge scour analyses. The design flood for scour is based on the 100-year return period event and the check flood for scour is based on the 500-year return period event. Minimum design scour depths for the foundations for the 100-year return period event are 30.0 feet and for the 500-year return period event, 35.0 feet.

3.3 *Wind*

The minimum design wind for the completed permanent structure is based on the site specific wind profile corresponding with the 100-year return period. The bridge is designed to not have any structural instability, such as buckling or brittle failure for an Extreme Event limit state load combination including wind loads with a 2,000-year return period. Additionally the bridge cannot have aerodynamic instability (flutter, torsional divergence, etc.) for a wind event having a probability consistent with a return period of 10,000 years. According to the wind climate analysis

Report BLWT-SS31-2004, the 100-year, mean hourly speed at deck level is 91 mph; 121 mph as a 2000 year return period speed; and a 10-min mean speed of 140 mph for the 10,000 year speed. The wind tunnel testing program consist of Sectional Model testing, Small Scale Sectional Model Testing, Aeroelastic Model Testing (Completed Bridge and During Construction); and Cable Vibration Assessment. Figures 7 and 8 illustrate the Sectional and Aeroelastic Models tested. The section with traffic is considered to be aerodynamically stable.

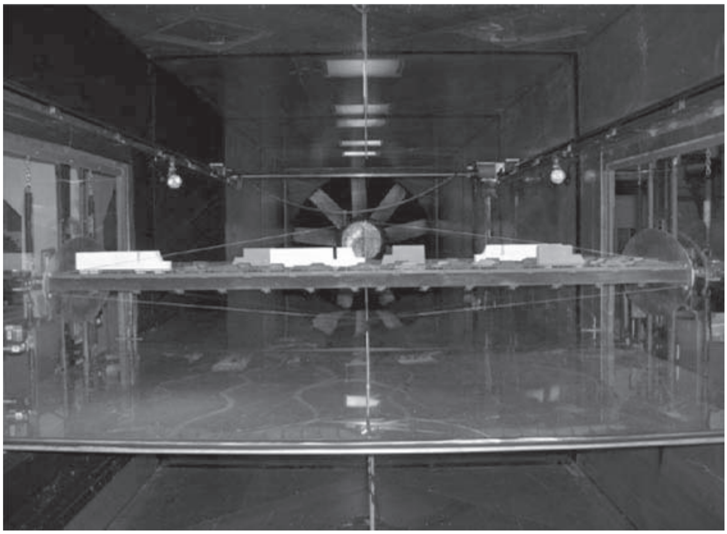


Figure 7. Sectional model.

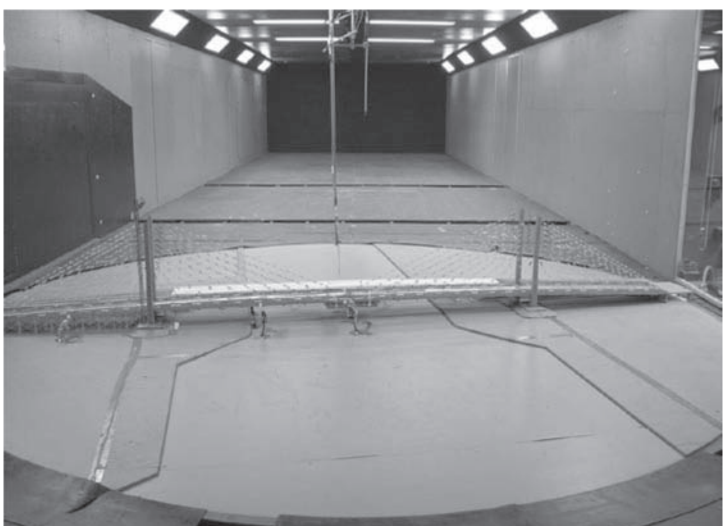


Figure 8. Aeroelastic model.

3.4 *Geotechnical*

Explorations conducted at the site have encountered an upper zone (Stratum I) consisting of barrier island medium to fine sands that extend down to about −35 feet. Below the sands are lagoonal clays and silts (Stratum II) that extend to a depth of about −50 to −100 feet. This zone is underlain by medium dense to very dense Pleistocene sands and gravels that extend to approximately elevation—135 feet (Stratum III), underlain by somewhat over consolidated clays and silts (Stratum IV) inter-layered with sands to termination of drilling depths ranging from 150 to 250 feet in the deepest borings. Figure 9 depicts the soil cross section of the bridge site. Bedrock in this vicinity is likely more than 1,000 feet below ground surface.

The loading conditions imposed by the bridge require use of deep foundations. Several deep foundation types were considered including 6-foot to 10-foot-diameter drilled shafts and 48-inch, open-ended steel pipe piles. These foundation elements would bear in the dense sands of Stratum III. In addition to the above pile types, the team also considered use of 36-inch-square, precast, prestressed concrete ("PSC") piles due to previous successful application on a similar project and availability of pile manufacturing capacity and suitable pile driving equipment within the team organization. Ultimately it was determined that the 36-inch-square piles provided the most benefits from a capacity, schedule and cost considerations. The primary foundation load demand is generated by the pylons that support the cable stayed bridge. The pylons have groups of 42 piles each arranged in 6 rows of 7 piles. Pile spacing is 2.9-diameters center-to-center. The footings are 50-ft by 58.75-ft. The inside spacing of the pylon footings and foundations maintains the 900-foot clear distance required in the *Specifications.* The significant benefit of using these piles is that they minimize the overall footing imprint. These piles allow the main span to be 950-ft based on the 900-ft horizontal clearance requirement. The large bending capacity of these piles allows them to be designed as vertical piles without having to batter them to resist horizontal loads. Three static compression load tests were performed and two tension load tests. The 36-inch-square piles were successfully loaded to 1800 tons in compression and 600 tons in tension.

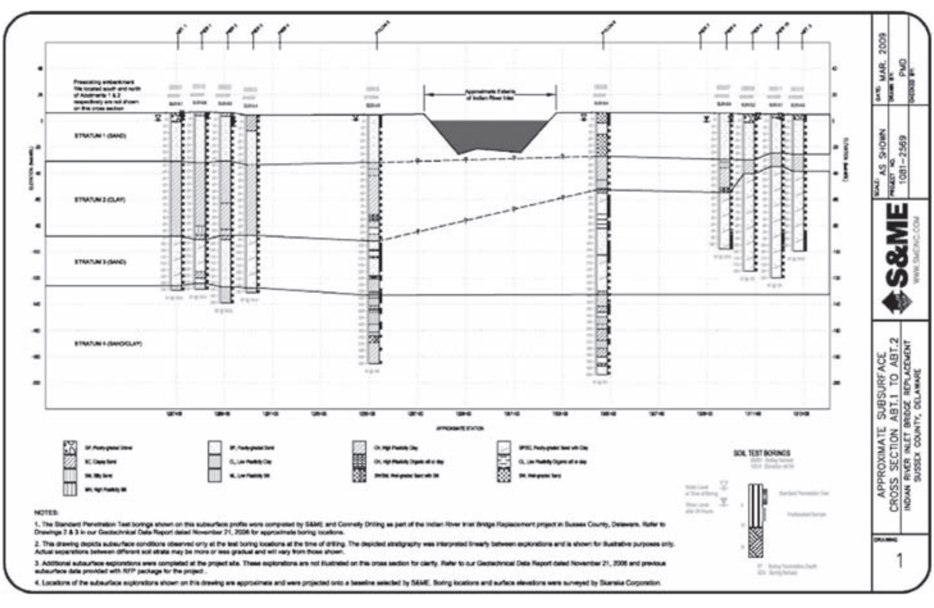


Figure 9. Subsurface profile.

3.5 Structural components

3.5.1 Cable stay superstructure

The design uses a semi-harped stay configuration. As shown in Figure 10, cable stays are spaced at 24-ft centers with cip superstructure segments also in 24-ft sections. Four planes of cable stays make up the structure with 19 stays for the back span and 19 stays for the main span at each of the four pylons. As is typical for this type of structure, the back spans are meant to balance the main span loads. Experience dictates that the most efficient back span to main span ratio falls between 40% and 45%. The 400':950' span ratio yields approximately 42%. This span configuration results in the last 5 stays being more closely spaced at approximately 8-ft centers. This type of configuration also results in reduced bending in the back span, but requires ballast or counterweighting to offset the remainder of the main span load.

Figure 11 illustrates that two transverse floor beams spaced at 12-ft centers are located within a 24-ft long typical cip segment. The reason for using the closer 12-ft spacing rather than a more conventional spacing of 25-ft to 30-ft, which is normally used for this type of cable stayed bridge, is two fold. First, the deck thickness is minimized to 8½" with a 12-ft beam spacing rather than 12" that is required for the larger 25-ft to 30-ft spacing. Secondly, the 12-ft beam spacing facilitates easier deck form placement and removal, which speeds up the erection process. Perhaps the greatest benefit of using closer beam spacing is the overall savings in dead load. There is approximately 11% savings in the concrete weight and about 2% reduction in stay weight. These two reductions also result in lesser demands on the foundations and form traveler.

A combination of precast and cip concrete transverse floor beams are being used. Precast concrete floor beams are being used in the back spans and the portion of the main span that is accessible by land. The precast concrete floor beams take advantage of a standard prestressed concrete girder shape, but modify the web thickness and truncate the top flange. The portion of the main span over the Inlet uses cip concrete floor beams. Since much of the bridge is land accessible, it makes sense to precast the floor beams as much as possible to remove this operation from the critical path of construction. It also results in one less concreting operation to be done on-site, which saves both time and money.

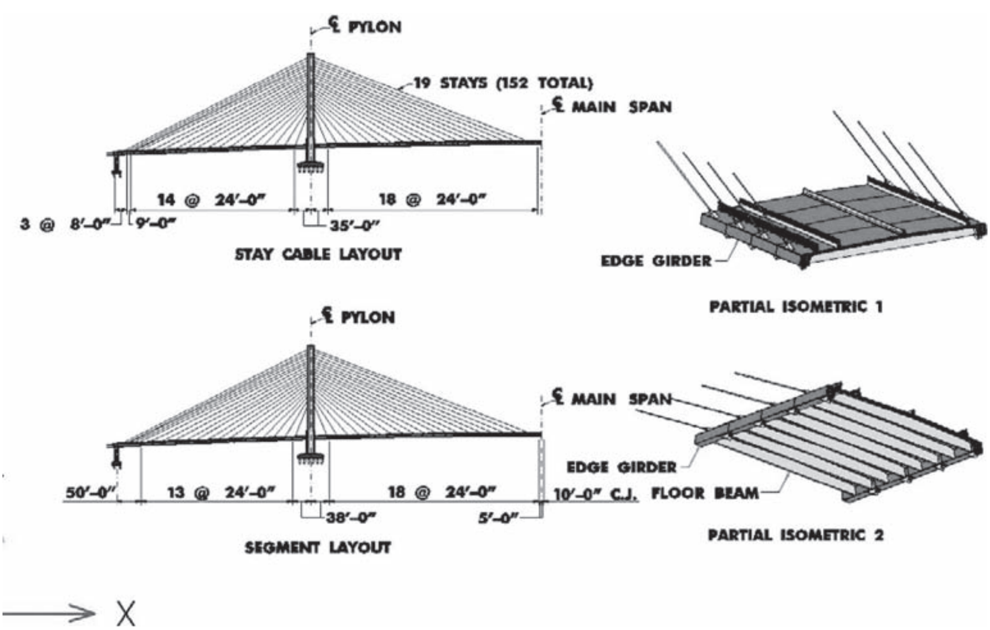


Figure 10. Superstructure segment layout & cable stay configuration.

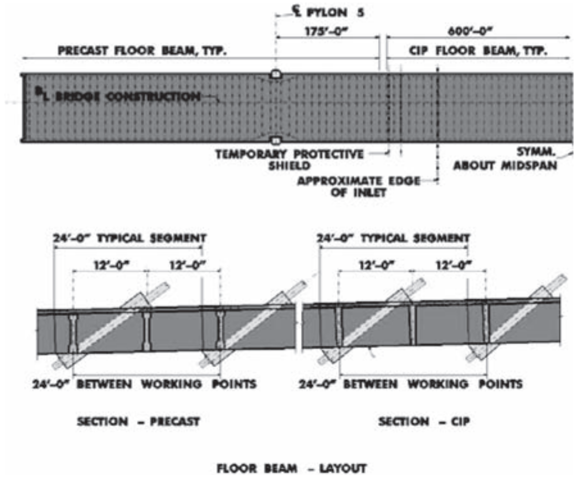


Figure 11. Transverse floor beams.

3.5.2 *Pylons*

As illustrated in Figure 12, the stay cables are anchored in two vertical reinforced concrete pylons using structural steel anchorage boxes to resist longitudinal tensions across the pylon section. The deck is fixed for longitudinal movements at the North pylon and free to move at the South pylon. The deck is guided laterally at both pylons. There are many benefits associated with using single mast pylons, not the least of which is their inherent aesthetic appeal. Some of the major benefits include:

1. Ease of construction with jump forms and tower cranes.
2. Ease of access to the interior void and straight shaft allows for a simple platform, ladder and material lift configuration.
3. Structural steel boxes are used to anchor the stays. The anchorage boxes take advantage of the high tensile capacity of structural steel. They are a very efficient and economical way to resist the high horizontal tension resulting from the cable stays, while the vertical compression from the cable stays is handled by the concrete.
4. One of the innovations made for the Indian River Inlet Bridge is that the inside wall thickness of the pylon is 2'–6" while the outside wall thickness is 1'–6". This results in the center of gravity of the section falling within 3-inches of the centerline of the stay cables, thereby minimizing the p-delta effects of eccentrically loading the pylon.
5. The slender shape and rounded corners of the pylon cross section provide excellent aerodynamic properties. The improved shape allows for wind drag reduction, which in turn reduces the moments in the pylons and the demands on the foundations.
6. The need for a cross strut that would normally be used to connect the twin pylons for stability, has been eliminated. The lack of the strut significantly expedites the speed of construction, is less costly to build, and reduces the need for inspecting and maintaining another bridge component. The elimination of the strut resulted from a combination of judicious design (minimizing the p-delta effect resulting from the centerline of stay cables being eccentric to the center of gravity of the pylon), and improving the aerodynamic characteristics of the cross section. This enabled the pylons to be designed with conventional reinforcing steel.
7. Not using vertical post-tensioning tendons reduces the overall cost, construction time, and most importantly future maintenance and inspection related to grouted post-tensioning tendons.

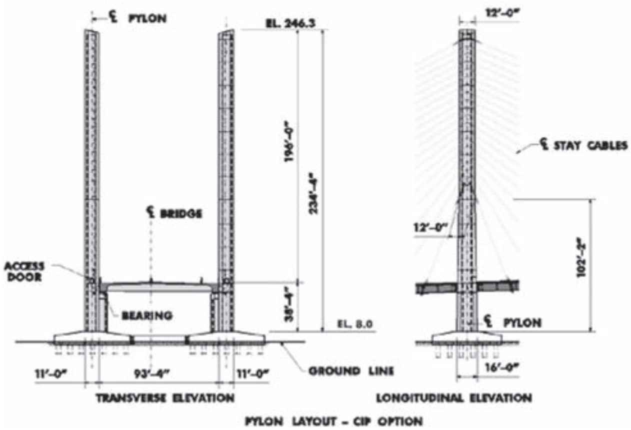


Figure 12. Pylon configuration.

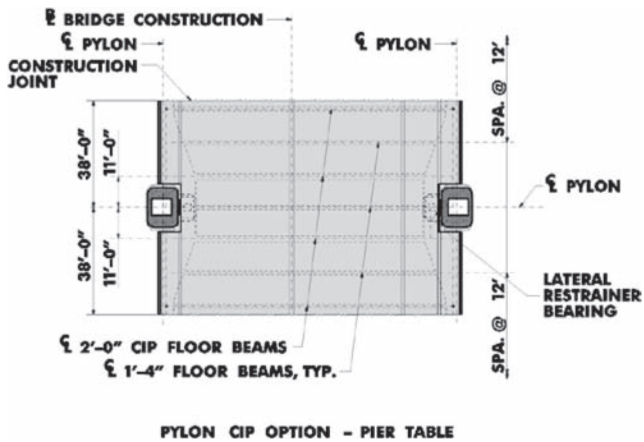


Figure 13. Deviated edge girder.

8. Another unique and innovative aspect of the single mast pylon is that the superstructure edge girders are designed to deviate around the pylon (Figure 13). This accomplishes several key design features:

   - Allows the cable stays to remain in a vertical plane because the centerline of the edge girder remains in line with the center of gravity of the pylon
   - Improves aesthetics and also eliminates any out of plane forces on the pylon
   - Allows the pylon to act as a primary compression member under dead load

3.5.3 *Stay cable details*

Freyssinet's patented state-of-the-art cable stay system is being used for this project. Freyssinet's system as illustrated in Figure 14 will meet the Department's required 75-year service life for the stay cables and the 25 year service life for the stay cable vibration suppression system. The corrosion protection system consists of an outer HDPE sheathing and then each strand is individually protected with HDPE sheathing. Each strand is coextruded with HDPE sheathing and all voids within the strand are filled with a corrosion resistant material (typically wax or grease). The outer sheathing has a double helical rib (extruded during duct manufacture) to reduce wind and rain vibrations. The outer sheathing is also weather and ultra-violet resistant. The color of the outside sheathing will be based on public input. The compact stay system is the latest innovation developed by the industry and it will be employed for this project. The compact system results in about a 17% reduction in stay pipe diameter. The smaller pipe diameter reduces the dynamic wind effects and wind loadings. The compact stay system optimizes the pylon, pier and foundation designs.

The cable stays will be mono-strand stressed, which facilitates ease of installation and stressing (i.e. no pulling large bundles of stays through sheathing and large hydraulic rams are not needed to stress the stays as a bundle). The stays are designed to be replaced completely, or one strand at a time while maintaining traffic. The stays are also designed for the ultimate cable stay loss load case. The cable stay system will accommodate DelDOT's requirement for fifteen (15) reference strands to be distributed throughout the bridge for corrosion and service life monitoring purposes. The stay cable sizes range from 19 to 60 strands (0.62-inch diameter) for stay cable lengths ranging from 95-ft to 505-ft. There are a total of 152 cable stays that anchor in the pylons (dead end) and edge girders (live end). In accordance with the Department's design requirements, the live end will have two (2) caps. The first cap will be transparent to allow visual inspection of the strands. The second cap will be stainless steel and will be designed to protect the transparent cap. It will be designed to be removed to expose the transparent cap during inspection.

3.5.4 *Transition piers*

Transition Piers offer another unique innovation by using a combination of counterbalance weight from the approach span prestressed concrete Bulb T girders, transition pier cap and ballast concrete in the cable stay superstructure. Counterweight is required to control uplift that is induced by the cable stay support system. By using the approach span girders and transition pier cap, the need for ballast concrete is reduced. Figure 15 illustrates the cap being supported by disc bearings. This bearing support system allows the superstructure to move without transferring large longitudinal forces into the columns and pile foundations. These piers are relatively short and the resulting stiffness characteristics would result in unsightly ring cracking of the piers if they were designed with a fixed connection between the transition pier cap and the columns. The Bulb T girders are supported by sliding bearings designed to provide longitudinal movement of 17".

Figure 14. Freyssinet stay cables.

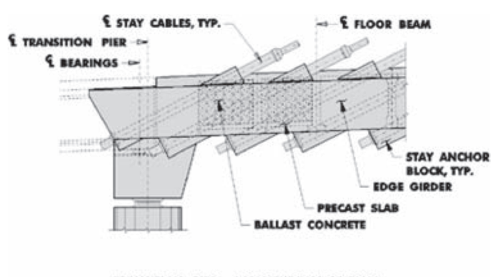


Figure 15. Transition pier.

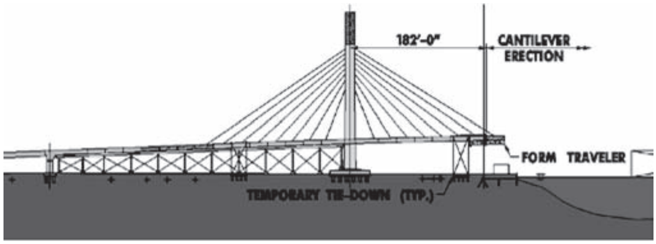


Figure 16. Typical erection sequence.

3.5.5 *Erection sequence*

The cable stayed spans will be a combination of cantilever erection over the inlet using a form traveler and cast-in-place construction on falsework for the portions of spans over land. Figure 16 illustrates the basic construction sequence. Skanska is constructing the bridge on the north and south sides of the inlet simultaneously. Two sets of equipment including pile driving equipment, form travelers, pylon forms, tower cranes, temporary falsework, materials, manpower, etc. are being used to open the bridge 7 months ahead of DelDOT's schedule.

4 SCHEDULE

Design:	2008–2009
Cable Stay Spans:	2009–2010
Approach Spans:	2010
Bridge Open to Traffic:	Summer 2011

ACKNOWLEDGEMENTS

Owner: Delaware Department of Transportation
Design Build Team: Skanska USA Civil Southeast
AECOM USA (Lead Design)
International Bridge Technologies (Cable Stay Design)
S&ME (Geotechnical Engineering)
RWDI (Wind)
Freyssinet (Cable Stay Supplier)
PB Americas (Design QC & CEI)
Maunsell | AECOM (Erection Engineering)
Finley Engineering Group (Falsework)

REFERENCES

American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications, Fourth Edition, 2007 and subsequent interim specifications through 2008.

Comite Euro-International De Beton—Federation Internationale De La Preconstrainte (CEB-FIP) Model Code for Concrete Structures, 1990, Chapter 2 Material Properties (reference for concrete creep and shrinkage effects only).

Delaware Department of Transportation Bridge Design Manual, May 2005.

Delaware Department of Transportation, Standard Specifications for Road and Bridge Construction, 2001.

Post-Tensioning Institute Recommendations for Stay Cable Design, Testing and Installation, Fifth Edition, 2007.

Scope of Services Package—Contract Documents Part 3—Design-Build Performance Specifications.

Scope of Services Package—Contract Documents Part 4—Design-Build Special Provisions.

Service d'Etudes Techniques des Routes et Autoroutes (SETRA) Cable Stays—Recommendations of French Interministerial Commission on Prestressing (CIP), June 2002 (reference for stay cable bending stresses only).

Chapter 10

A self anchored suspension pedestrian bridge over Harbor Drive in San Diego, California, USA

D. Fitzwilliam & J. Tognoli
T.Y. Lin International, San Diego, California, USA

ABSTRACT: San Diego has commissioned a landmark pedestrian bridge from its new ballpark—over several sets of train tracks, and over a busy downtown thoroughfare—to the convention center and the San Diego Bay. The bridge design is a self-anchored suspension bridge with a single inclined pylon. The main span of the bridge is 108 m and the pylon is 40 m tall. The pylon is inclined at a 60 degree angle from the horizontal and leans over the deck to support the single pair of suspension cables. 34 individual suspenders attached to the main cable support the 6 meter wide deck from the top of the railing at only one edge of the deck. The bridge is horizontally curved and a tendon is stressed at the top of the railing. The radial force generated by the tendon above the deck elevation generates a restoring moment which balances the forces in the bridge deck.

Keywords: footbridge; self-anchored suspension; aesthetic; structural concepts; stainless steel.

1 INTRODUCTION

The downtown area of the City of San Diego is situated on the San Diego Bay. For many years it has been the goal of the city to complete a pedestrian and bicycle link between the historic Balboa Park area of the city through downtown all the way to the Bay and Harbor area. The last step in the link from Park to Bay is blocked by the local trolley tracks, several sets of freight train tracks, and a busy downtown thoroughfare. In 2004, the city commissioned the Centre City Development Corporation (CCDC) to design and build a bridge to complete the approximately 2 mile route linking the Park to the Bay. The site chosen for the final bridge link is adjacent to the recently constructed Petco Park, Home of Major League Baseball's San Diego Padres and the nearly 50,000 sq.m. San Diego Convention Center on the San Diego Bay.

CCDC recognized that the high profile project location needed a landmark structure to act as the gateway to the city and as an icon of the revitalized downtown area of San Diego. They hired a design team lead by bridge designers, T.Y. Lin International, to develop plans, Figure 1, for the bridge and surrounding plazas. This paper describes the design process for the bridge from the type selection phase through final design and highlights some of the more interesting features of the bridge.

2 TYPE SELECTION PHASE

The first step in the process of designing the bridge was the type selection phase. Four structural concepts were developed and sketched to be presented to the various entities involved as well as to the public. The combination of public input and agency discussions led to the selection of the design concept that was taken to final design. The four bridge alternates presented for consideration were:

1. Steel space truss bridge
2. Cable stayed bridge
3. Self anchored suspension bridge
4. Arched steel shell bridge

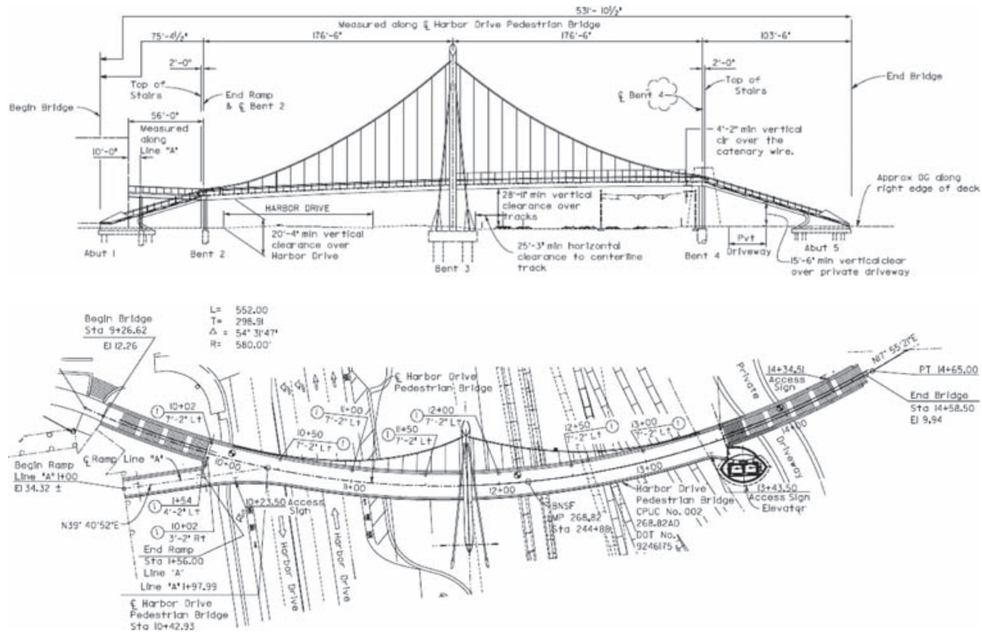


Figure 1. Plan and elevation of Harbor Drive Pedestrian Bridge (Fukuhara, 2006).

Each of the design concepts were evaluated for aesthetics, cost, and suitability for the site; among other criteria.

2.1 *Steel space truss*

The steel space truss alternate for the site used tubular steel members to form a triangular space truss to support the concrete bridge deck. The design used an undulating alignment with spiral ramps at the approaches. Columns were also proposed to be constructed from tubular steel elements. Figure 2 shows the architects rendering of this alternate.

2.2 *Cable stayed bridge*

The cable stayed bridge alternate is a fairly conventional single pylon cable stayed bridge design. The bridge deck was proposed on a curved alignment to provide a more dynamic feeling. The bridge concept had two planes of stays connected to a single pylon situated in the center of the walkway width. Renderings of how the bridge would look from various vantage points are shown in Figure 3.

2.3 *Self anchored suspension bridge*

The third alternate presented was a self-anchored suspension bridge. This is a relatively unique concept which borrows some of its design characteristics from conventional two pylon suspension bridges but changes the anchorage points from the usual large and costly ground anchors to an anchorage which is attached to the structure itself. More drama was added to the bridge layout by inclining the single pylon on its axis and using a single plane of suspender cables to suspend the deck from only one edge. The overall effect of these modifications to a conventional suspension bridge can be seen in Figure 4.

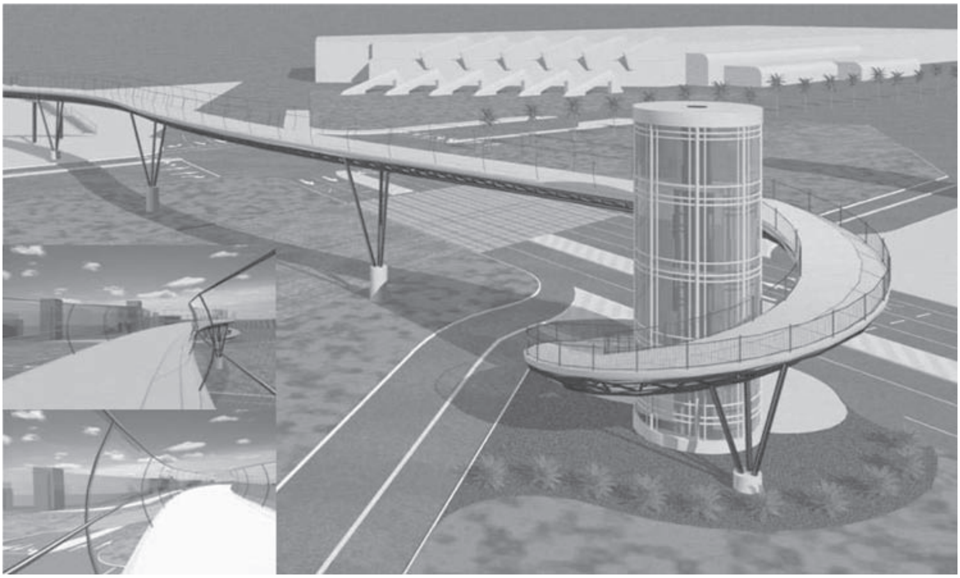


Figure 2. Steel space truss alternate (Rabines, 2005).

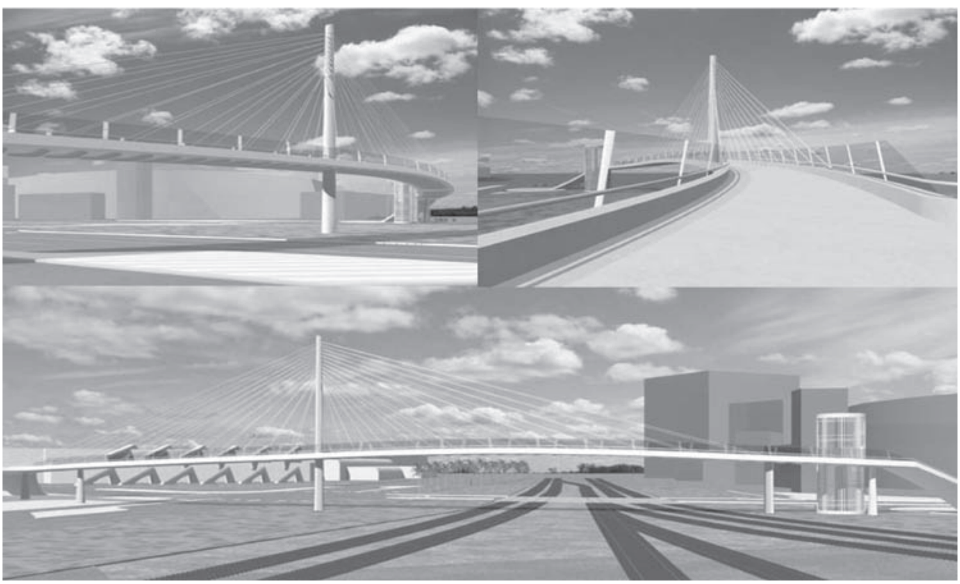


Figure 3. Cable stayed alternate (Rabines, 2005).

2.4 *Arched steel shell*

The final design concept is probably the most unique of them all. The concept uses an arched steel shell structure to support the thin concrete bridge deck. Design and fabrication of the steel shell would likely be more complex than the previous three concepts, but it could be made to produce a very striking and memorable bridge as shown in Figure 5.

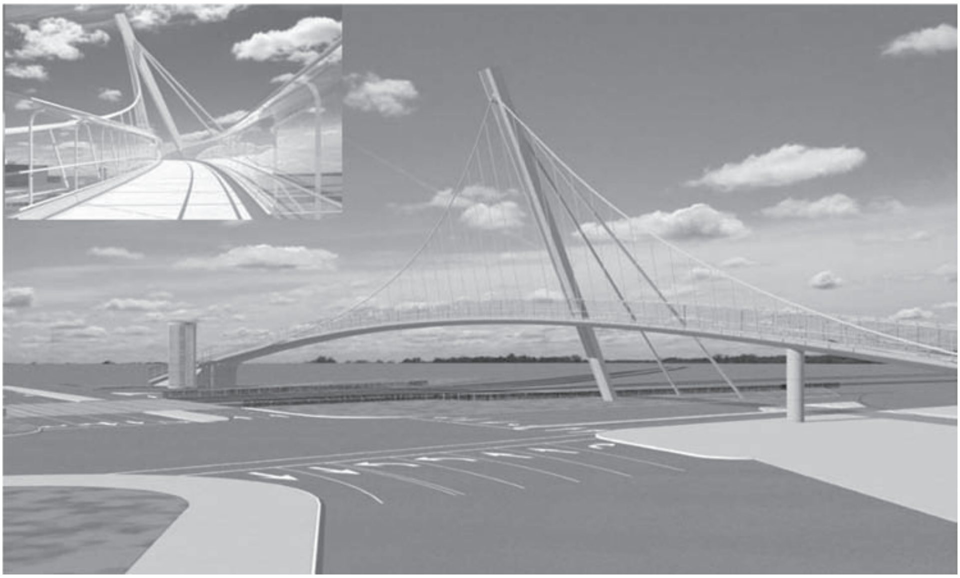


Figure 4. Self anchored suspension alternate (Rabines, 2005).

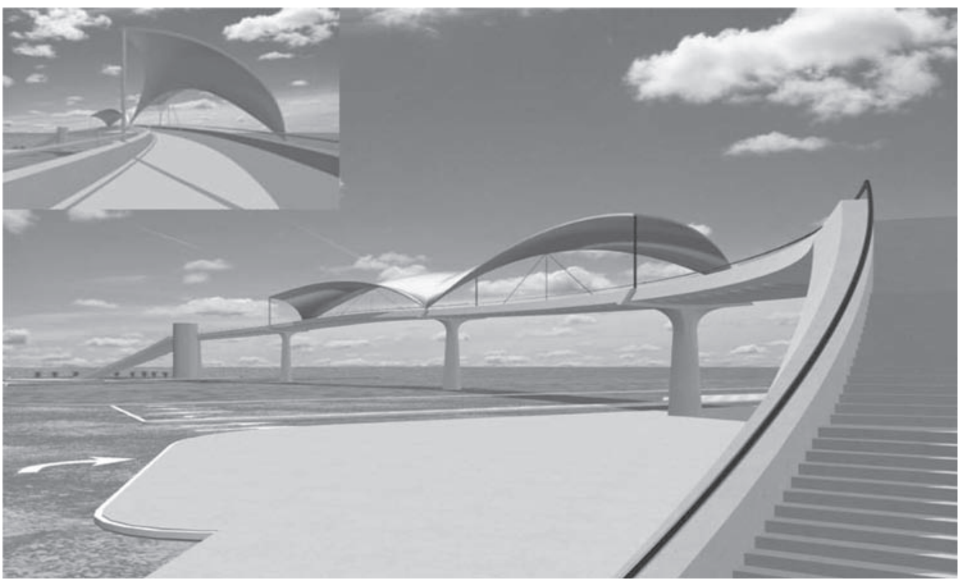


Figure 5. Arched steel shell alternate (Rabines, 2005).

2.5 *Selected alternate*

After several open house meetings with the public and subsequent stakeholder meetings, the self-anchored suspension bridge alternate was chosen. This alternate was a clear public favorite and allowed a relatively transparent feeling. As can be seen in Figure 6, the curved alignment allows the public to view the bay, ballpark, and the bridge itself from different perspectives. The bridge also fits in well aesthetically with both the ball park and the convention center.

Figure 6. Rendering of final design of selected alternate (Rabines, 2007).

3 BRIDGE STATICS

Once the selection of the alternate for final design was made, many challenges still remained. The most obvious feature that needed to be addressed was the self-anchored suspension bridge mechanics. In conventional suspension bridges, the main cable element is usually draped between two main pylons. At the ends of the main cable the tension induced by supporting the weight of the bridge is resisted by massive cable anchorages firmly attached to the ground. In a self-anchored suspension bridge the tension in the main cable is resisted through compression in the deck of the structure.

Figure 7 demonstrates the fundamental difference in the relationship between the bridge and cables in a conventional and a self-anchored suspension bridge. The bridge is further complicated by the presence of stairs in the approach spans at each end of the bridge. The solution in this case was to continue the main cables all the way to the base of the stairs and anchor them in the abutments at the bottom of each set of stairs. In order to handle the angle change from the deck to the stairs, the cable passes through a steel deviator device placed at the top of each stair span and embedded into the concrete bend cap at these locations. This solution means that some of the tension in the main cables will be resisted by the abutment foundation, but the majority of the force from the main cable is still resisted by the much stiffer stair and deck section.

The method of supporting the bridge deck with cables attached to only one side of the deck was another design feature that provided a challenge in designing the bridge. From the typical section of the bridge shown in Figure 8, the unbalanced nature of the deck becomes very apparent. It seems as if the deck wants to rotate clockwise around the suspender support point at the left edge of the section. The torsion generated by the unbalanced support location must be compensated in some way. The best solution would be to get the line of action of the supporting force to pass through the center of gravity of the bridge section. An unsymmetrical cross-section was developed to try to move the center of gravity of the section as far to the left as possible. Unfortunately, the center of gravity could not be shifted far enough to the left to achieve a balanced design. The support point was then moved to the top of the railing in an effort to move the line of action farther to the right in the section.

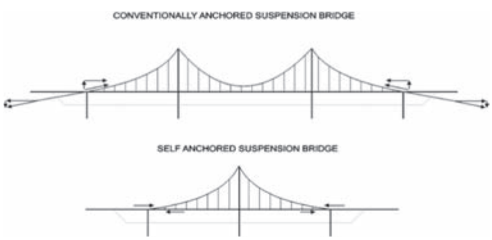


Figure 7. Suspension bridges (Fukuhara, 2005).

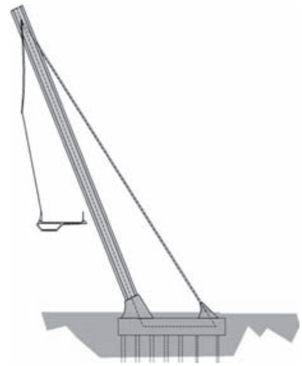


Figure 8. Typical section (Fukuhara, 2005).

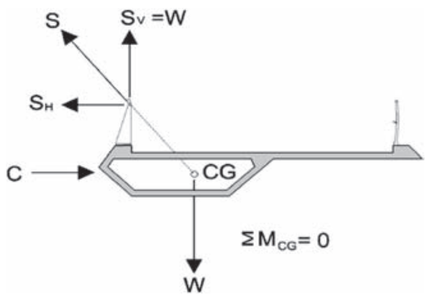


Figure 9. Force diagram (Fukuhara, 2005).

Figure 9 also shows that in order to achieve equilibrium, an additional horizontal force is needed to balance the horizontal component of the suspender force. In order to provide this necessary horizontal balancing force, the deck of the bridge was designed with a horizontal curvature. In this way the axial compression in the deck generates a radial force directed outwards that can be made to balance the horizontal reaction from the suspenders. Due to the over six meter wide deck, the final geometry of the structure fell short of the balanced condition shown in the figure. Still more balancing torsion was needed to reach equilibrium in the section. Since the deck had already been constructed on a horizontal curve, an additional radial inward force could be added to the equation at a distance above the center of gravity in order to provide the balancing torsion that was needed. The additional radial force was provided in this case by the addition of a longitudinal post-tensioned cable at the location of the top of the railing post.

4 MAIN CABLE SYSTEM

Figure 10 shows the final resulting diagram of forces acting on the section resulting in a balanced system. Another non-traditional design feature of the bridge is the treatment of the connection of the suspenders to the main suspension cable. Typically, the cables are connected to specially designed cable bands which are tightly connected to the main cable. The cable bands are fastened so that they transmit the complete suspender reaction into the main cable. Often the presence of the cable band at the location of each suspender creates a bulky and rather aesthetically unpleasing discontinuity in the otherwise smooth line of the main cable. In large scale vehicular bridges, this is typically viewed only from a long distance or when passing in a car at high speeds.

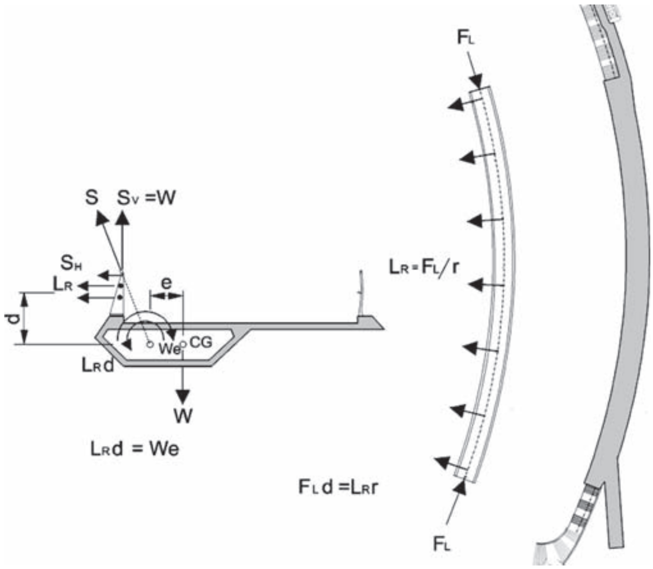


Figure 10. Radial load and arch effect provide equilibrium (Fukuhara, 2005).

For a pedestrian bridge the user is much closer to the details of the bridge and is travelling at much lower speeds. The details of the connections and hardware on the bridge take on a much more important role in these types of bridges. For this bridge, the main cable is completely enclosed inside a continuous stainless steel guide pipe. This guide pipe provides a smooth line along the length of the main cable without any discontinuities at the cable bands. The suspenders are in turn attached to gusset plates on the pipe rather than directly fastened to the main cable via a cable band. This means that the force from each suspender is partially resisted by the angle change in the main cable and partially by the axial force in the guide pipe. More specifically, the force perpendicular to the direction of the main cable is resisted through an angle change in the direction of the main cable, and the force tangential to the main cable is resisted primarily by a change in axial force in the guide pipe and partially by the friction between the main cable and the guide pipe.

Modelling the unique connection method requires modelling of both the main cable as a cable element as well as modelling the guide pipe as a series of truss elements with a friction connection to the main cable. Detailed hand calculations of the predicted final shape of the main cable were required to achieve an analysis model which was close enough to allow the finite element software to converge on the correct solution.

5 CONSTRUCTION CONSIDERATIONS

The bridge is designed to be constructed as a cast-in-place box girder on conventional falsework built over the existing Harbor Drive and Railroad tracks. Once the entire bridge is built and the stays are attached, the main suspension cable will be stressed simultaneously from both ends of the bridge. The stressing of the main cable will take up all the slack in the individual suspenders and lift the deck off the falsework. The suspenders will then be tested to determine if they are within tolerance of the expected load. They can be adjusted as needed to achieve the final design force and geometry.

The pylon is cast-in-place in four lifts. After each lift, internal post-tensioning tendons are stressed to provide stability and to control stresses in the pylon. At the final pour of the top

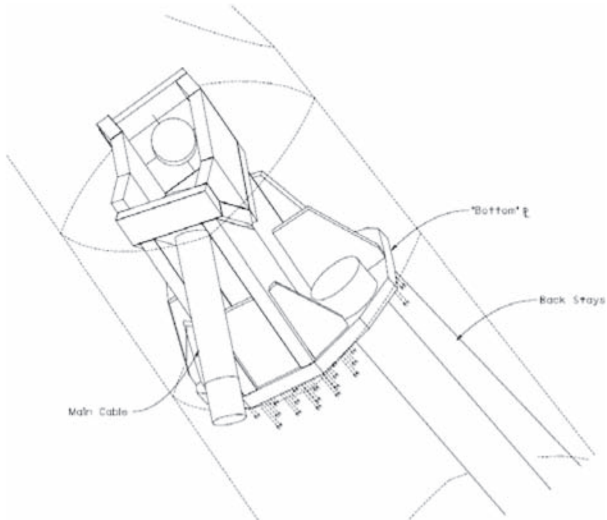


Figure 11. Pylon head anchorage (Fukuhara, 2006).

of the pylon, a large steel weldment, shown in Figure 11, is cast into the concrete and acts as the anchorage frame for the main cables as well as for two back stays which provide the final equilibrium to balance the horizontal component of the dead load generated by the main cable. The overturning moment generated on the pylon footing is resisted by using twelve soil anchors cast into the pile cap of the pylon foundation. The compression forces on the foundation are taken by four 2.1 meter diameter cast-in-drilled-hole (CIDH) piles.

The angle of the main cable as it anchors into the pylon generates a net horizontal reaction on the pylon. The horizontal force is balanced by a net horizontal reaction on the abutment foundations. The force is transmitted to the deck through the suspender reactions along the entire length of the main span. These radial forces acting on the bridge deck generate an inward deflection of the bridge deck. This inward force must be accounted for in a horizontal camber of the bridge structure. In order to provide for this camber, the falsework used to construct the cast-in-place deck must be flexible enough to allow for a horizontal displacement of the deck when the main cable is stressed.

In addition to the vertical and horizontal camber, there is also a need to allow for some rotational camber of the bridge deck. As shown in Figure 10, the radial force generated by the post-tensioned tendon at the top of the railing post generates the torsion necessary to balance the dead load torsion on the deck. This means that the deck is not balanced for torsion until both the main cable and the “radial” cable have both been fully stressed. The result is that the deck must be constructed with a slight torsional camber to achieve the final super elevation needed.

6 STAINLESS STEEL

Another aspect of the bridge that was greatly influenced by the nature of pedestrian bridges was the use of stainless steel. Since the bridge is adjacent to the San Diego Bay it is classified as a marine environment. The materials used in such an environment must take this into consideration. In order to limit the maintenance requirements on the bridge, and to mitigate the effects of neglected maintenance, the steel on the bridge was designed using stainless steel rather than painted steel. Stainless steel has the added benefit of a clean and architecturally pleasing quality.

The two most common stainless steels, Type 304 and 316, are susceptible to crevice corrosion in chloride containing environments. Based on the results of chloride ion tests performed at the site, it was determined that lower grade varieties of stainless steel would likely rust in the salt air around the bay. More highly alloyed stainless steels are not susceptible to crevice corrosion.

As discussed in the previous section of this paper, the deck is suspended from the top of the steel railing elements at the edge of the walkway. Furthermore, there is a longitudinal tendon at the top of the railing generating a considerable radial force which must be transferred through the railing posts down into the concrete deck. With the primary load path passing through the steel railing elements, there was a need to have high strength reliable steel for the design of the railing posts.

The combination of the need for corrosion resistance and high strength lead to the selection of high strength duplex stainless steel for the project. Type 2205 was the preferred grade specified. As an added benefit, to the properties described above, Type 2205 stainless steel was less expensive than Type 316 stainless steel at the time of writing of the specifications.

The selection of duplex stainless steel for the railing posts also posed a challenge for the connection of the railing posts to the other steel elements of the bridge. The suspenders selected for use on the bridge also needed to be made of stainless steel in order to avoid galvanic corrosion problems that could limit the bridge life. The suspender anchors were also specified to be made from Type 2205 duplex stainless steel. For similar reasons the bolts which anchor the railing posts down to the bridge deck were chosen from stainless steel materials. 17-4 precipitation hardened stainless steel was specified for all the bolts which would come in contact with the stainless steel posts.

7 CONCLUSIONS

The high profile location of the final link in the pedestrian route from the historic Balboa Park to the downtown and San Diego Bay area led to the selection of a truly unique and exciting bridge. The design and detailing of this project required some innovative solutions that both challenge conventional bridge methods and provide new architectural models for future bridge designers. Innovation is demonstrated in the project from the basic selection of the bridge type to the smallest details of the design. Construction of the bridge is scheduled for completion in Summer 2010. When complete, the bridge will stand as an icon for the City of San Diego.

ACKNOWLEDGEMENTS

Client/Owner:	Centre City Development Corporation
Architect:	Safdie Rabines Architects
Conceptual Collaboration and Independent Checker:	Strasky and Anatech (Jiri Strasky, Tomas Kompfner)
Design Team:	T.Y. Lin International (Joe Tognoli, Brett Makley)
Stainless Steel Consultant:	TMR Stainless

REFERENCES

Fukuhara, T. 2005. Sketches of bridge equilibrium.
Fukuhara, T. 2006. Plans for construction of Harbor Drive Pedestrian Bridge.
Rabines, R. 2005. Renderings for type selection of Harbor Drive Pedestrian Bridge.
Rabines, R. 2007. Final Rendering of Harbor Drive Pedestrian Bridge.

Chapter 11

Stay cable replacement, high level engineering for an extended serviceability

E. Mellier, S. Joye & V. Maillet
Freyssinet International, France

ABSTRACT: This paper illustrates through Freyssinet experience that stay cable replacement works are highly technical works requiring specialised equipment, techniques and engineering at all stages of the operation. Structural assessment and design, construction methods, work schedule and organisation are closely interconnected and need to be studied in a very coordinated manner by teams having experience in all of these fields. This is necessary to end with a technical solution that is durable according to modern standards, compatible with the structural capacity of the bridge, acceptable for the users in term of disturbance on the bridge operability.

Regarding these concerns, parallel strand cables offer significant advantages as they are highly durable, modular, and they require only light equipment and small space for being installed.

By extending the service life of bridges without traffic interruption, stay cable replacement operations respond perfectly to the rising demand for sustainable solutions permitting material and cost savings.

Keywords: Repair works, stay cable replacement, parallel strand stay cables

1 INTRODUCTION

Development of stay cable bridges really started in the 70s. At that time and until the early 90s, the stay cable specific durability requirements were not assessed properly. Many bridges built at that time now evidence early corrosion and fatigue deterioration of the cables. As the design life of these bridges is far from being reached, the cables have to be replaced.

As a leading actor on the stay cable market, Freyssinet have acquired a unique experience. General Belgrano and Zarate bridges in Argentina, Mezcala bridge in Mexico, Lanaye bridge in Belgium, Second Severn bridge in UK, Penang bridge in Malaysia are among the major cable replacement projects Freyssinet have completed over the past ten years.

This paper evidences the common problematic that was faced on these six projects, outlining the specificity and the complexity of this kind of works, but also how they are challenging for engineers.

2 EVALUATION OF THE STRUCTURE

Like any repair work, the first step is the evaluation of the real state of the structure and particularly the cables. This is to be done in several stages, beginning with a visual inspection of the structure. As the visual inspection does not automatically delivers all the information, further investigation needs to be performed, through local openings, or monitoring, and completed with a computation of structural strength in order to evaluate the actual safety in the damaged members.

2.1 *Visual inspection*

Visual inspection of the cable is of course the first action to be undertaken when starting with cable health assessment on a bridge. This can reveal obvious damages as visible on Figure 1 for example:

– Unusual sag of the cable, revealing the loss of tensile force in the cable
– Heavy rust on external wire layers
– Wire failures
– Even cable rupture

This is of course the first, cheapest and easiest stage of the inspection. But this is not sufficient as some cables, especially parallel wire cables can experience gangrene effect (Figure 2). By this phenomena is called the fact that moisture penetrate into the cable through a local damage in the outer protection somewhere along the cable, travels by capillarity along the cable through the inner voids that exist in-between the wires and is trapped especially at the bottom end of the cable. Due to grout injection and the interwire friction, broken wires are reanchored by their neighbours which are overloaded and the load in the cable is not released. Hence the cable can be in very serious distress without exhibiting any visible sign of it.

A typical example is a bridge in South America where Freyssinet was called to repair one cable that had experience sudden failure and discovered that 9 other cables where very heavily corroded (up to 60% of the wires broken) without any visible evidence of that.

Hence the necessity to go beyond outer visual inspection of the cable and to open and inspect the anchorages, open locally the outer cable sheath, especially at the bottom end, next to the anchorage.

Another way to evaluate a cable health is the monitoring of the cable.

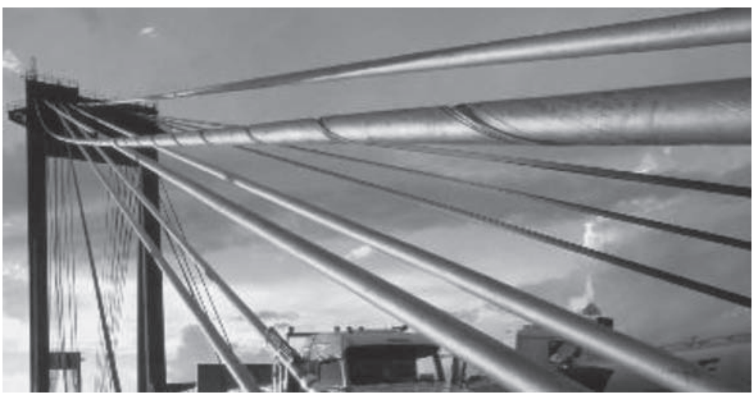


Figure 1. Obvious damage on a locked coil cable, Gal Belgrano Bridge, Argentina.

Figure 2. Example of gangrene effect on a PWS cable, Zarate Bridge, Argentina.

2.2 *Monitoring*

Monitoring the cables can be very useful when assessing a structure as it gives indications on both the current situation and the evolution of the state of the cable.

Forces can be measured by several means:

- A lift-off operation with a jack gives the value at the time of the measure with a very good precision.
- A method through vibration analysis has the advantage of requiring much lighter equipments but with an accuracy that is lower.
- When one wants to evaluate the force variations under different weather conditions and traffic configurations through a continuous force measurement, a load cell needs to be installed.
- An acoustic monitoring is also very useful when dealing with fragile cables, either due to corrosion or bad fatigue behaviour. By automatically detecting the noise emission when a wire breaks, one can know the number and the frequence of wire breaks, their evolution with time. Through specific analysis methods developed by companies like Advitam with their Soundprint® system (Figure 3), it is possible to come back to the residual strength of the cables and use it as a warning system in order to act correctly and with the adequate reactivity in case of a sudden acceleration of the cable degradation process.

2.3 *Computation*

The analysis of the real state of the cable need to be completed with a proper and detailed structural analysis of the bridge in order to quantify the residual bearing capacity of the structure and any restriction on live load that might be required in most severe cases so that an adequate level of safety remains.

This was done for example for Mezcala bridge (Figure 4) where two cables were damaged in 2007 following an accident involving a truck and a bus that burnt.

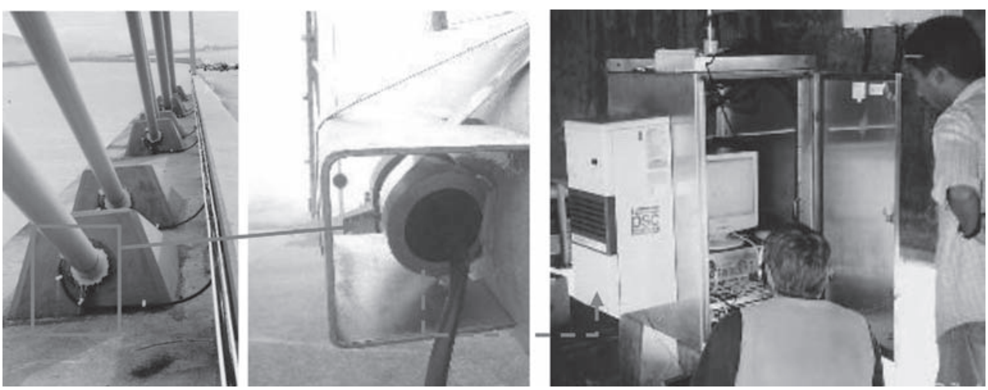


Figure 3. Soundprint® system on Penang Bridge, Malaysia.

Figure 4. Structural model for the assessment of the residual bearing capacity of Mezcala Bridge (Mexico) after 2007 major road accident.

As part of the repair works, Freyssinet performed a computer analysis of the bridge behaviour within a few days after the accident and proved that two out of the four lanes on the bridge could remain in operation without any risk for the structure and for the users.

3 3D COMPUTER MODEL

Whenever the assessment reveals extensive loss of capacity or shortening of service life of the cable, the replacement needs to be foreseen. This is a heavy repair operation that needs to be prepared adequately.

3.1 *Computer model*

Replacing cables a temporary and sometimes permanent major modification of the structural behaviour of stay cable bridge. Hence it requires a detailed evaluation of the forces and stresses in the structure at all stages:

– Initial stage
– During each stage of the replacement works
– At the final stage

Hence it is necessary to build a complete 3D model of the structure in order t assess properly all consequences of the works on the existing structural members.

3.2 *Permanent state*

First verification should focus on the structure checking in the future configuration with new stays. As the cable technology may be changed during the replacement, especially if the original one has evidenced unexpectedly rapid ageing, the stiffness of the cable may not be exactly the same. Also, as materials are stronger nowadays than they were a few decades ago, the engineer can be tempted to benefit from these progresses and install cables with stronger material and hence smaller cross section.

One has to be careful with the fact that this implies a change in the rigidity of the cables, often from stiffer to softer cables. Hence deformations under live loads are bigger and hence flexural stresses in the deck are increased compared with the original state. Three cases then:

– In the best case these overstresses are acceptable and the new cables can be optimised to modern standards, provided it is demonstrated through calculation.
– In the worst case, the structural design is already optimised and the structure cannot accept any overstress. Hence the new cables have to be designed in order to provide the same rigidity as the old ones, even if they don't work at their full strength in service.
– In most cases, the reality is in-between and an optimum has to be found through iterative computer analysis in order find the minimal cable cross sections that induce the maximal overstresses that are acceptable for the existing structure.

This optimisation has been necessary for the cable replacement of Penang bridge in Malaysia where original cables were very stiff as they were composed of prestressing bars encased in a thick steel tube. The Freyssinet parallel strand cables that have been installed to replace them have of course a much higher working stress (50% of 1860 MPa). Hence cables designed for the same ultimate strength would have been too slender and have induced bending in the concrete deck that would have lead to unacceptable crack openings.

3.3 *Seismic assessment*

For the same reason, a complete reassessment of the seismic behaviour of the bridge is necessary if the new cables do not have the same rigidity as the old ones (Figure 5). A cable replacement

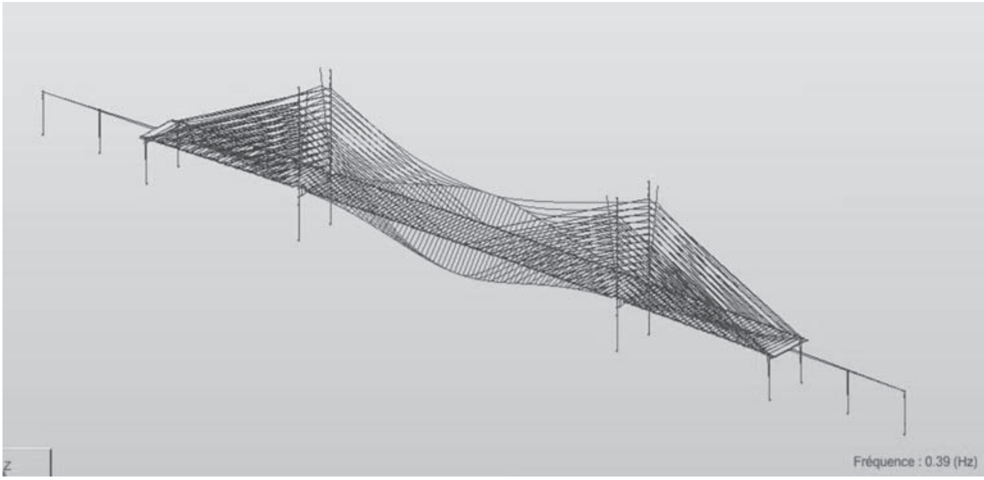


Figure 5. Reassessment of Penang Bridge behaviour under seismic load.

operation may also be an opportunity to perform a seismic retrofitting of the bridge, following a seismic code revision for example.

3.4 *Temporary phases*

Temporary phases need to be fully justified as well. The engineering work to be done is very specific to this kind of repair works, and much heavier than for a classic construction engineering.

The main reason for this is that the bridge remains in operation during the replacement works. Hence any construction stage which is a different structural model as cables are missing and some temporary ones are added, is actually a service state and needs to be combined with traffic loads, wind loads and temperature variations.

An assumption has to be made on the stresses that are acceptable for the old cables whose capacity is known with a variable degree of reliability. A criteria that can be adopted and that seems of common sense is that the old cables should not experience tensile stresses higher than the one they have effectively experienced in a recent past. This choice of this criteria is crucial in term of the bridge structural safety during the works. It is the result of a risk analysis and a compromise: if the criteria is too low, the required structural safety is not obtained. If too high, it will have serious consequences on the studies and work complexity as it will impose the use of temporary cables stressed prior to the replacement works in order to release part of the tension in the existing cables and multiply the number of tensioning and detensionning phases in order to continuously adapt the cable tensions. Hence it has to be chosen carefully in close coordination between the owner, the engineer and the contractor.

Another aspect to be careful with is linked to the stiffness variation that was already mentioned. At the transition between old and new cables, there is a sudden variation in the stiffness of the deck support as illustrated on Figure 6. Hence the stiffer cables, usually the old ones, attract more load than usually, which can lead to unacceptable overstresses. Once again, this can make necessary the use of temporary cables or length adjustments on existing and new cables in order to compensate this effect.

4 TEMPORARY CABLES

Temporary cables are often required when replacing cables. Different configurations happen actually.

On old cable stay bridges cable severance and cable replacement have not been considered as a load case at the design stage. Hence the deck and the adjacent cables are not capable of holding

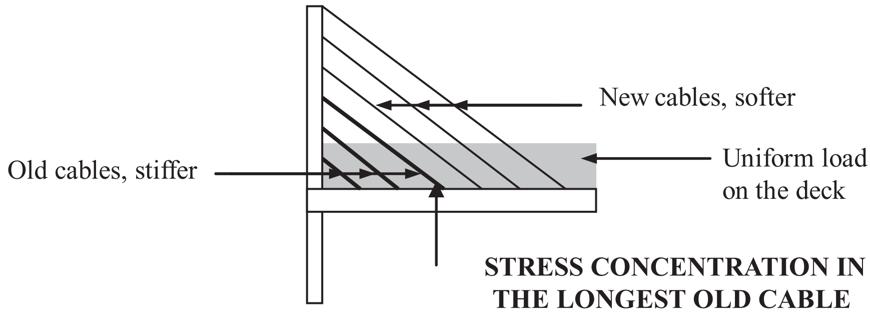


Figure 6. Mechanism of stress concentration in the longest old cable due to cable stiffness difference.

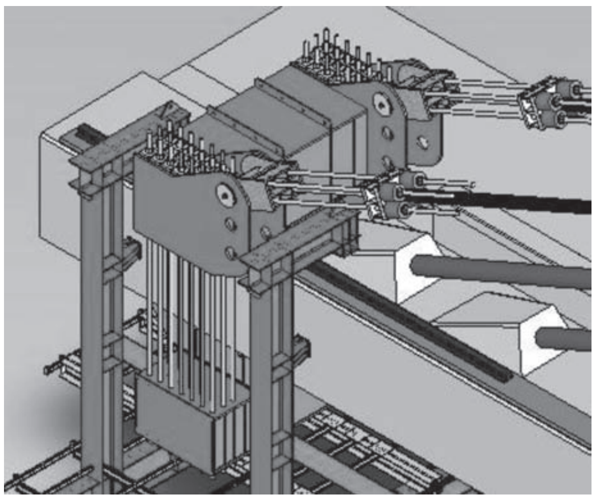


Figure 7. Clamping beam anchoring temporary cables.

the loads when a cable is missing. In that case, temporary cables are necessary and need to be anchored close to the anchoring points of the dismantled cable.

The bridge is designed for cable replacement but additional cables are required even though because the actual state of the existing cables is not known with certainty and extra cables are used to limit overstresses in adjacent cables. In this case, there is more flexibility with the choice of the location of the temporary anchors.

Of course, anchoring new cables even temporarily in a structure that has not been designed for this requires engineering expertise and experience in order to minimize remaining damage on the structure.

Steel anchorages clamped on the structure with prestressing bars are often chosen as the damage on the structure is minimal (Figure 7). Especially coring is minimized.

5 CABLE DETENSIONNING AND DISMANTLING

The detensionning of the existing cable is certainly the most critical operation as a huge amount of energy is released during the operation. Hence this needs to happen smoothly in order to:

Figure 8. Controlled cable distressing in progress.

- Limit dynamic effects that could damage the structure
- Prevent any unexpected movement of the cable that could endanger the working personnel
- Prevent any unsafe feeling from the bridge users

The progressive energy release can be obtained by two procedures:

- Either the cable is composed of multiple tensile elements (i.e. wires) and hence each time you cut a wire you only release a small part of the total cable force. This is only applicable when the cable wires are totally independent and do not re-anchor one on the other, otherwise the energy is not released but transferred to other wires until the remaining ones fail suddenly.
- Either it is preferred that the entire cross section of the cable is cut at the same time and in this case it is necessary to distress the entire cable with heavy means (jacks) either before or after having cut the cable (Figure 8).

In any case, cutting the cable with tools that are usually used for external prestressing cables inside concrete box girders is not possible as this would induce excessive hazards for bridges users and working personnel. A picture of a cut prestressing tendon is self explanatory.

6 WORK UNDER BRIDGE OPERATION AND CONSEQUENCES

Another particularity of these works is that they always occur on bridges that are long span structures and vital links for the economy and the social life of the neighbouring communities. Hence it is not an option to close the bridge during the replacement works that last several months.

Only the emergency lane and in most favourable cases one traffic lane can be closed. It means that these operations have to be accomplished with very limited space, practically a 3 meter wide strip along the bridge edge girder.

This has crucial consequences on the tools that can be used, the handling capacity, the sequencing of the works and finally the schedule.

Many specific tools need to be designed in order to work in such a specific environment for:

- Distressing the cables
- Lowering the cables to the deck
- Evacuating old cables
- Installing new cables.

On this last item, parallel strand cables offer a significant advantage as the strand by strand installation process only requires light equipment and small space on the deck (Figure 9).

Figure 9. Parallel strand stay cable installation on a bridge under traffic.

7 CONCLUSIONS

This paper has demonstrated through various examples that stay cable replacement works are highly technical works requiring specialised equipment and techniques with engineering needed at all steps of the operation. Structural assessment, construction methods and work schedule and organisation are closely interconnected and need to be studied in a very coordinated manner by teams having experience in all of these fields. This is the only way for ending with a technical solution that is durable according to modern standards, compatible with the structural capacity of the bridge, acceptable for the users in term of disturbance on the trafficability.

Regarding these concerns, parallel strand cables offer significant advantages as they are modular, require only light equipment and small space for being installed.

By extending the service life of bridges without interruption of traffic, stay cable replacement operations respond perfectly to the rising demand for sustainable solutions permitting material and cost savings.

REFERENCES

BS 5400, Part 4. 1990. CP for Design of concrete bridges.
EN 1993-1-11. 2007. Design of steel structures. Design of structures with tension components.
Freyssinet. 2007. Freyssinet HD Stay Cables.

3 Movable bridges

Chapter 12

Rehabilitation of Bridge Street Bridge preserving a nineteenth century historic bascule lift span

P. van Hagen & H. Protin
HDR Engineering, Newark, New Jersey, USA

ABSTRACT: The Bridge Street Bridge was constructed in 1880 by the King Iron Bridge Company of Cleveland, Ohio. It survives as a unique example of a nineteenth century hand operated mechanical drawbridge. The lifting system is based on a French design from the early 19th Century. This French design had a unique solution to dealing with the variation of bridge balance required while opening a bascule span. This system allowed the bridge to be opened and closed by hand with a relatively constant effort.

This paper will discuss some of the unique characteristic of the basic bridge design. The paper will also discuss the efforts that were made during the rehabilitation of the Bridge Street Bridge to maintain an important link to Rockland County's historic heritage as well as our nation's heritage of creative bridge engineering.

1 INTRODUCTION

The Bridge Street Bridge was constructed in 1880 by the King Iron Bridge Company of Cleveland, Ohio. It survives as a unique example of a nineteenth century hand operated mechanical drawbridge. The lifting system is based on a French design from the early 19th Century.

The Bridge Street Bridge has long since been replaced as a vehicular crossing across the Sparkill Creek. The Ferdon Avenue Fixed Bridge crossing is located just down stream of Bridge Street. The Bridge Street Bridge has deteriorated significantly over the years and has been closed to vehicular traffic since the construction of the Ferdon Avenue Bridge. Understanding the importance of this link to Piermont's past the County of Rockland Department of Highways was committed to reconstructing this Historic Bridge. In spring 2004 the County hired Bergmann Associates to prepare a rehabilitation design for the for this landmark structure.

What makes the Bridge Street Bridge unique and why is it important to spend taxpayers' money to preserve this important link to our past? This paper will discuss some of the unique characteristic of the basic bridge design. The paper will also discuss the efforts that were made during the rehabilitation of the Bridge Street Bridge to maintain an important link to Rockland County's historic heritage as well as our nation's history of creative bridge engineering.

1.1 *Piermont, NY—A brief history*[1]

The Village of Piermont part of Orangetown in Rockland County, NY is a small isolated community below the Palisades on the West bank of the Hudson River. Piermont is in the shadows of the Tappan Zee Bridge. The Village is located adjacent the Sparkill Creek which flows into the Hudson River through a break in the Palisades. Sparkill creek was a gateway to the interior portions west of the Hudson. It is no wonder this location became a trade center. The local Tappan Indians traded with Dutch settlers. The area known as the Slote (meaning ditch in Dutch) or the Landing, was teeming with wild life including: salt and freshwater fish, oysters, geese, swans and other shore birds, deer, bear, mountain lions and elk. Settlers moved west through the creek and their goods were transported back along the creek to market. A dam was built upstream of the

Bridge Street Bridge location to power mills. The dam is still standing today and the mill has been converted to residential property.

During the American Revolution the Slote was at the center of the conflict with numerous raids and canon fire from British ships in the Hudson River. In 1783 George Washington was headquartered in Tappan and met with Sir Guy Carleton, Commander—in—Chief of His Majesty's forces to develop the evacuation of British troops from New York.

The Slote boomed in the 1830's when it became the eastern terminus for the Erie Railroad which ran from Lake Erie to the Hudson River. The construction of the Erie Railroad would be the beginning of the end for the Erie Canal's dominance as an inland trade route. At the time of construction of the eastern terminus it was illegal for the Erie Railroad to cross the border into New Jersey. As part of the construction a long pier was constructed that extended a mile into the Hudson River. The pier was constructed from rock and fill from the adjoining hillside. It is from this pier that Piermont gets its name since Piermont is located between the pier on one side and the mountain on the other. In 1853 Erie Railroad would locate its eastern terminus in New Jersey at Jersey City, now known as the Erie Lackawanna Terminal on the Hoboken Jersey City border. The Erie terminal at Piermont would become superfluous and a rail link was constructed between Piermont and Jersey City.

2 BRIDGE STREET DESIGN

The Bridge Street Drawbridge was constructed in 1880 by the King Iron Bridge Company. The King Iron Bridge Co. played an important role in the development and construction of metal truss bridges, a product of American engineering and construction technology, nationwide during the later part of the Nineteenth Century. The King Iron Bridge & Manufacturing Co. was organized under that name in Cleveland in 1871 by Zenas King, who had started his career in building bridges in 1858. … During the 1880's the Company was the largest highway bridge works in the country.[(2)] … Included in the list of movable bridges that were built by the King Iron Bridge Company is the University Heights Bridge over the Harlem River in New York and the Bridge Street Bridge in Piermont. Bridge Street may be the only bridge of its type still in existence today.

The Bridge Street Bridge is a hand cranked drawbridge. The bridge design is a through truss drawbridge construction similar in design to a medieval drawbridge. The ends of the span are lifted by chains (see Figure 1). The drawspan rotates about pinned bearings at the north abutment. Although the bridge rotates about a fixed point, the design is not a true bascule design. The word bascule is derived from a French word bascule meaning seesaw or balance. The seesaw approach was an early solution to bridge balancing. A traditional bascule bridge utilizes a counterweight behind the pivot point of the seesaw so the net unbalanced moment is close to zero (see Figure 2). As the bridge opens and the center of gravity of the span moves closer to the pivot, the center of gravity of the counterweight also moves closer to the pivot. As the span arm shortens the counterweight arm shortens proportionately. Therefore, the bridge remains in a balanced state throughout the opening and closing operation. This balanced condition is true if you assume the bridge is a plane and there is no vertical variation in the centers of gravity of the span or the counterweight.

When opening a drawbridge similar to Bridge Street the movable span pivots about a fixed center of rotation or pivot point. As the bridge opens the center of gravity moves closer to the pivot point and the moment required to overcome the imbalance decreases (see Figure 3). The problem with hand operation is the significant force required to open the bridge when the span is down. At opening the initial load in each of the lifting chains would be greater than half the weight of the drawspan.

Early attempts to balance a drawspan were made by connecting the other end of the lifting chains to a counterweight. The lifting chain passed over a sheave located above and behind the drawspan. The sheave was free to rotate as the drawspan opened. The basic problem with the design is: if the counterweight is sized to balance the span when the bridge is in the down position the counterweight will be significantly heavier than required to balance the span as the bridge

Figure 1. Bridge Street Bridge prior to rehabilitation.

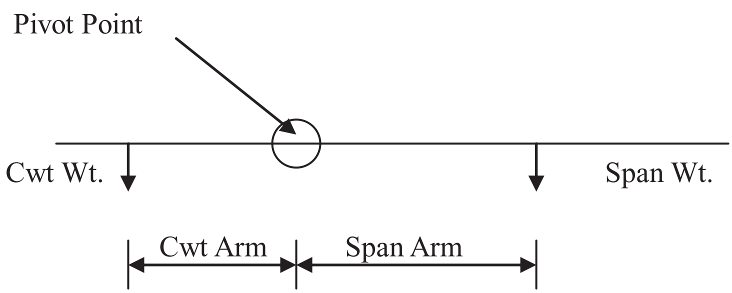


Figure 2. Schematic view of bascule.

opens. The result is excessive force is required to restrain the hand crank. The crank will rip out of the operators hand and slap them in the head at which point the counterweight will free fall until the span runs into the tower. Conversely if the counterweight is sized to balance the span in the open position excessive force would be required to control the span when closing.

The Bridge Street Bridge is based on a French design from the early 1800's which had a unique solution to dealing with the variation of bridge balance required while opening a drawspan.[(3)] There are two draw chains that lift the far end of the drawbridge. The draw chains wrap around sheaves as the bridge is opened. The sheaves are mounted to a shaft on top of a steel tower at the near abutment. The counterweights are hung from spiral cut sheaves mounted to the same shaft as the draw chains (see Figure 4). The spiral cut in the sheave means that the center of gravity of the counterweight moves inward toward the centerline of the shaft as the bridge is opened. Therefore the mechanical advantage from the counterweight changes as the as the center of gravity of the span moves closer to the pivot point. This system allowed the bridge to be opened and closed by hand with a relatively constant effort. A system of open gearing reduces the total torque required at the hand crank.

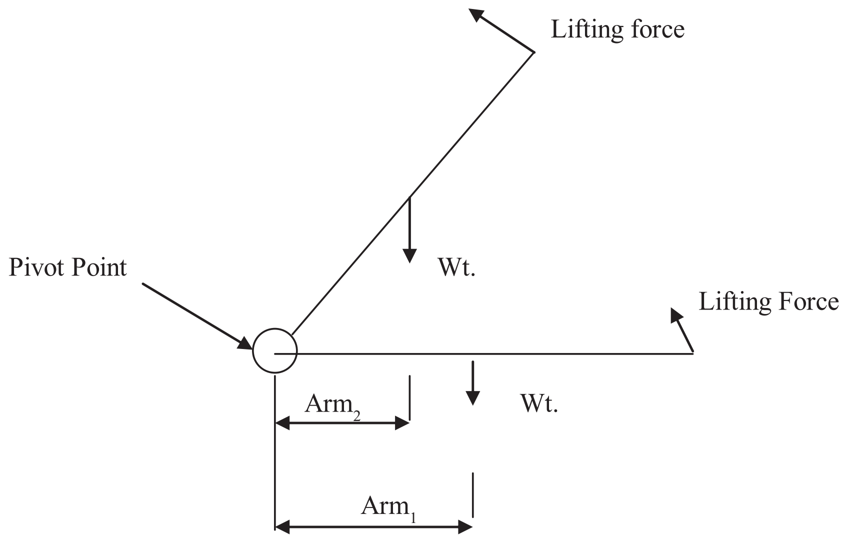


Figure 3. Schematic view of drawspan.

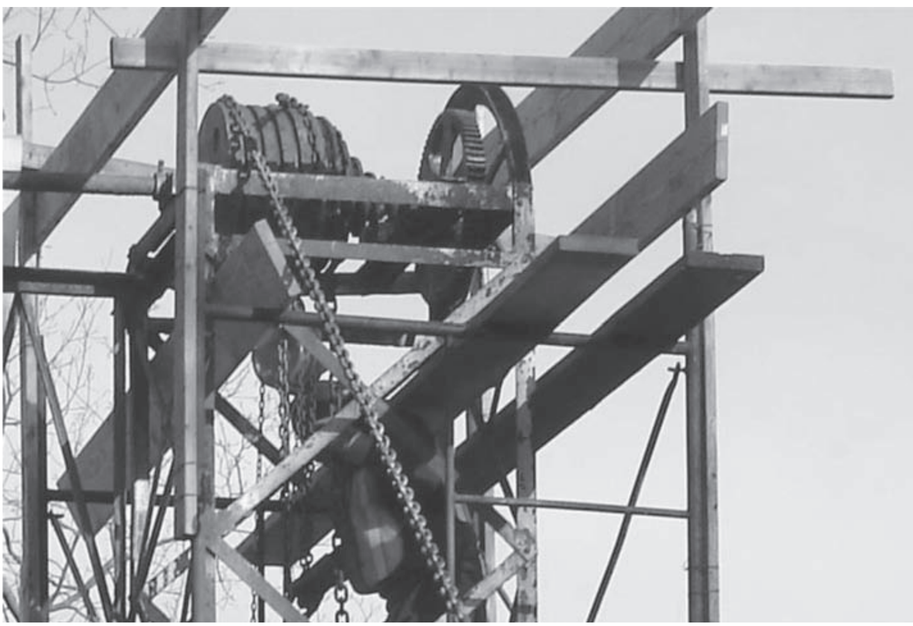


Figure 4. Constant diameter draw chain sheave and spiral cut counterweight sheave mounted on a common shaft.

3 REHABILITATION OF BRIDGE STREET BRIDGE

3.1 *Original design*

The original scope of work was to perform an in-depth inspection of the bridge, develop a bridge design report and to repair the bridge super structure so that it could adequately support pedestrian loading. The bridge was not to be opened to vehicular traffic. The lift span would be removed and repaired in the shop. The fixed portions of the bridge were to be cleaned and painted and rehabilitated in place. The existing concrete approach deck was to be resurfaced. Minor repairs were to be made to the substructure elements including repointing of stone abutments and resurfacing the concrete pier. Riprap scour protect would be placed around all substructure elements. The schedule was to get phase one: The Design Report approved and Phase Two: the final design completed in about 7 months so that Phase Three: Construction could start by summer of 2004.

The project was on a fast track with a tight budget. The Bridge Street Bridge is the only know bridge of its type, there were no available drawings, and there was no one alive today who was around in 1880 when the bridge was built.

Bergmann Associates performed a site survey, a bridge inspection and an underwater inspection was performed by Boswell Engineering during Phase One of the project in order to locate bridge and roadway elements and determine the extent of deterioration. Many bridge and roadway elements were inaccessible during the inspection. The approach span framing was encased in concrete with only the bottom of the stinger flanges exposed. The main span pivot bearings were buried and the base of the counterweight towers were buried in asphalt. The end floorbeams were inaccessible, as were connections to the deck trusses. Timber decking was covered with plywood and steel plates. The limits of the substructure elements were unknown. Stone abutments were partially surfaced with concrete, and the condition of stone work below the concrete facing was unknown. Excavating to determine hidden details was beyond the scope of work. We determined that we would provide documentation of the bridge details as was feasible but certainly sufficient for design purposes. We would require the contractor to provide precise documentation during demolition such that the contractor will be ability to reassemble bridge components.

It was difficult if not impossible in many instances to determine the original configuration and original construction materials. Was the original 1880 approach deck constructed of concrete or timber? It would be very unlikely to find a reinforced concrete bridge decks from 1880, but some unreinforced concrete jack arch type construction from the period is still in existence. We guessed that the original approach deck might have been timber and later replaced with concrete. That would explain the out-of plane deformation of the through truss top chords (see Figure 5). However, there were no records that would indicate the original approach deck construction. Similarly, we were unsure if the original pier construction was unreinforced concrete or if the concrete was added as a modification to an older stone pier. From the historic preservation point of view, we came to an agreement with the County that we would repair the bridge to show the history of the bridge rather than the worry about the initial design. For example if a timber deck were replaced with concrete the concrete is part of the bridge history. Therefore we would restore the concrete deck by patching and resurfacing with a new overlay. We would resurface the concrete pier. At the abutments we would re-point the exposed stone masonry and repair the concrete facing on the portions that had been resurfaced previously. This was the basic design approach that we pursued for the design report.

Some improvements were planned for the rehabilitated bridge. The new timber deck would be durable and low maintenance. A hardwood “Ipe” was chosen for the timber deck construction due it strength, durability and resistance to decay. New concrete bridge seats would be provided at the abutments.

The existing bridge was to be converted into a park setting. Timber benches and planter boxes will be incorporated into the design. An historic bridge plaque will be added to highlight the significance of the bridge and some of the unique characteristics of the bridge.

3.2 *Contract modifications to the design*

In September 2005 after the contract had sat a while and a new Project Manager took over for the County, some modifications were made to the design approach. It was decided that the existing approach span and the counterweight tower would be removed and rehabilitated in the shop. This would eliminate the cleaning and painting operations in close proximity to the residential

Figure 5. View looking south across the deteriorated timber deck on the bascule span at the start of construction. Also note the asphalt overlaid concrete approach deck and lateral deformation of the approach truss.

properties at the north side of the bridge. The existing concrete bridge would be replaced with a lightweight concrete reinforced deck to reduce the loading on the potentially overloaded approach span trusses and center pier foundations.

During flood conditions in the April 2006 the Sparkill Creek overtopped the bridge deck by as much as two feet. At this time it was decided to raise the bridge elevations to reduce the frequency with which the bridge. Along with the changes to the bridge elevation, the north roadway approach profile would be adjusted to remain ADA compliant and the pavement would be replaced with paving blocks as betterment to the community. In addition some planting areas were added so that the Village of Piermont can add flower beds along side the pavers at both approaches to the bridge.

4 CONSTRUCTION OF BRIDGE STREET BRIDGE

4.1 *Schedule*

The Contractor was given a Notice to Proceed on the construction in February of 2008. As per the contractor's schedule the estimated substantial completion would be in August 2008. Demolition of the existing structure would begin March 6, 2008.

4.2 *Removal of the existing structure*

The first step was to remove the steel towers (see Figure 6), including all machinery and counterweights, all carefully stored for delivery to the Contractor's yard for restoration later. Pieces to be reused were all marked and photographed. On the second day the Contractor cut the counterweight chain at the Northwest Tower Leg (see Figure 7). Since the chain is a historic element that was to be reused in the reconstruction of the bridge, the Contractor was informed that he would have to restore the chain to its original condition.

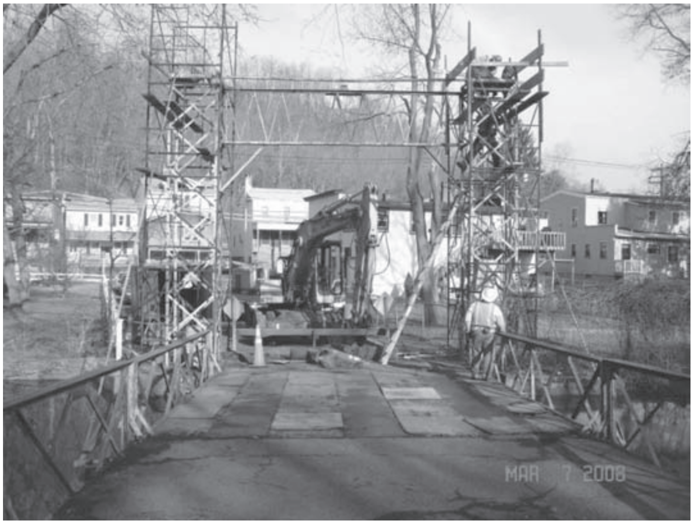


Figure 6. Demolition of steel towers.

Figure 7. Severed link at northeast counterweight chain.

After the removal of the Counterweight Towers, the bascule span was removed. The timber deck was dismantled and discarded, and the steel floorbeams and trusses were carefully brought onto land. Existing ornate cast-iron movable bridge bearings were uncovered and it was decided that they would be reused. At this stage it was apparent that the condition of the existing steel was much worse than had been expected. All of the transverse W10 floorbeams were so badly corroded they had to be replaced (see Figure 8). The lower chord, end angles, many diagonal members and gusset plates on each truss would have to be replaced. All bracing members including lateral bracing and truss knees braces would require replacement.

The approach span deck and steel framing was then removed, starting with the concrete slab spanning between the south abutment and the concrete center pier. The concrete deck was removed

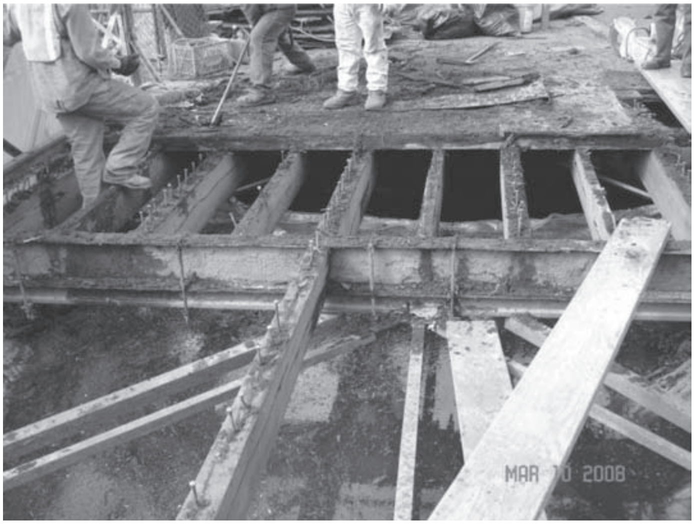


Figure 8. Demolition of timber deck.

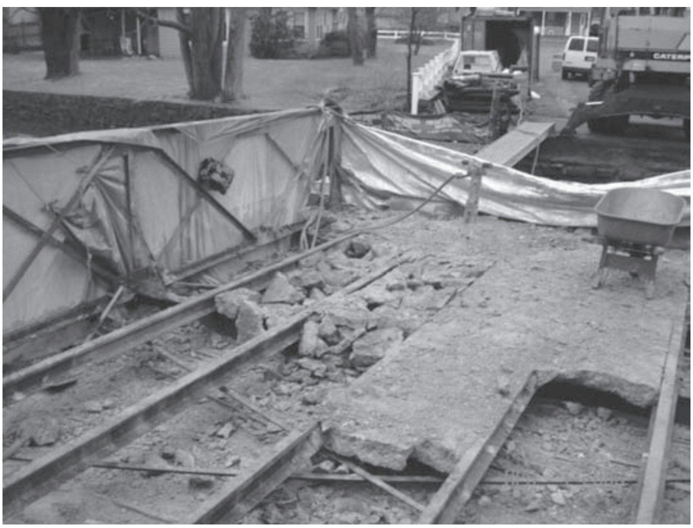


Figure 9. Demolition of the concrete approach deck uncovered used rails in the deck.

using saws and small jack hammers in an attempt to avoid damage to the existing steel trusses and steel beams embedded in the deck. During removal of the approach span concrete deck, the expected W12 steel beams embedded in the slab were not found. In fact the slab was supported by steel railroad rails cast into the bottom of the slab (see Figure 9). These rails apparently came from the dismantled Erie Rail Line into Piermont. It was immediately apparent that the concrete deck was not the original deck, but was rather from rehabilitation work in the first few decades of the twentieth century. It was also apparent at this time that a redesign of the approach span framing would be required.

Several days after the removal of the approach concrete slab the center pier started to rotate about the bottom north edge of the pier (see Figure 10). The approach slab had provided lateral

Figure 10. Concrete pier leaning toward the north.

restraint to the pier, and once removed the pier was free to rotate and translate. Exacerbated by an undermined condition, the top of the pier translated sideways approximately twelve inches.

4.3 *Reconstruction of the existing foundations*

The existing abutments consisted of dry stone walls seated just below the streambed. The contract called for pointing of the stone walls and construction of a new concrete bridge seat, including installation of stone riprap at the base of the wall. The contractor installed cofferdams and dewatered to gain access to the stone walls. During clearing of the riverbed with a backhoe for installation of the cofferdam, the wingwall adjacent to the abutment wall, which was constructed of loose stone with concrete stucco facing, was undermined and moved laterally several inches (see Figure 11). This wall had little foundation and eventually crumbled and collapsed into the river.

4.4 *Design changes during construction*

4.4.1 *Steelwork*

Due to a significant amount of deteriorated steel, many structural steel elements were replaced that were not included in the original design (see Figure 12). After the exact configuration of the structural elements to be replaced was determined, existing deteriorated steel pieces were carefully measured and redetailed. Elements that were redetailed were as follows: Bascule and approach steel trusses, including transverse steel floorbeams, angle knee braces, outriggers that attach to steel bascule span to the counterweight chains, and all lateral bracing members. Several machinery support members at the tops of the towers were badly deteriorated and were replaced.

4.4.2 *Approach span*

The concrete approach span was a recent addition to the bridge. It was determined that the concrete span was built c. 1930, and replaced what we believe was the original timber deck. It was suggested that for sake of consistency and bridge integrity that the approach span would be redesigned to provide a new timber deck and timber stringer system supported by transverse steel floorbeams similar to the drawspan. Several remaining steel elements strongly suggest that the original structure had been framed this way. It was also apparent that the weight of the concrete deck had caused truss buckling.

Figure 11. Movement of the southwest wingwall.

Figure 12. Severely corroded end floorbeam and truss gusset plate that were buried at the north abutment.

4.4.3 *Timber decking and beams*

The contract called for IPE hardwood decking and stringers. The contractor was not able to procure the member sizes specified in the desired species, and so the client agreed to modify the contract requirements to provide timber more readily available in the sizes required. The timber to be used was Alaskan Yellow Cedar, a relatively rare, but hard, durable timber, with excellent resistance to deterioration especially suitable to the marine environment (see Figure 13). The timber properties were not as specified for IPE and so the number and arrangement of the timbers beams was modified.

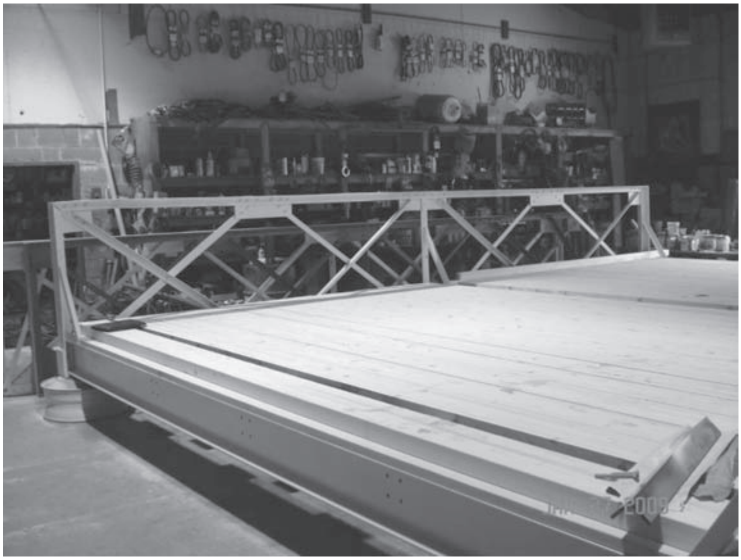


Figure 13. New timber deck and rehabilitated steel framing erected in shop.

4.4.4 *Stone wingwalls and abutments*

The collapse of the wingwall adjacent to the south abutment necessitated a design for the replacement of this wall. The wall was on the property of an adjacent landowner and so close coordination with the landowner and the local authorities was important. A gravity stone wall on a concrete foundation was designed to provide support the riverbanks and an adjacent local road.

4.4.5 *Center pier*

The collapse of the center pier was the most significant change to the scope of the project. The concrete pier, a more recent replacement of the original pier, was removed while the plan for its replacement was considered. Once removed, it became clear that the concrete pier had no significant foundation. It was merely sitting on top of two timber piles which had been cut-off below the base of the newer pier foundation. The presence of the timber piles suggested that the original pier was a timber pile bent. It was decided to redesign the pier and replace it with a timber pile bent. In order to determine the type and depth of the piles required, a boring and a geotechnical design was performed. The final redesign consisted of five-12" diameter piles with a timber piles cap and cross bracing (see Figures 14 and 15).

4.4.6 *Bridge geometry*

The Bridge geometry was modified particularly at the top of the abutments and the new timber pier to accommodate changes in pier design and approach span framing. It was also discovered during construction that the counterweight trunnion shaft was slightly skewed in reference to the bridge. The centerline of shaft at the west tower leg was set back about an inch relative to the east tower leg. At first glance it appeared as though there may have been some change in bridge geometry since the original construction. It is also that possible that the skew may have helped the draw chain to reeve along the fixed diameter sheave as the bridge opens.

4.5 *Rebuilding history*

4.5.1 *Steel detailing*

A thorough field inspection was performed during demolition to measure existing pieces to be replaced, and to determine what had been there, in some cases using existing rivet hole locations

Figure 14. View looking south from behind the north abutment. Note new timber piers and the reconstructed southwest wingwall.

Figure 15. View looking north at piles for new timber pier.

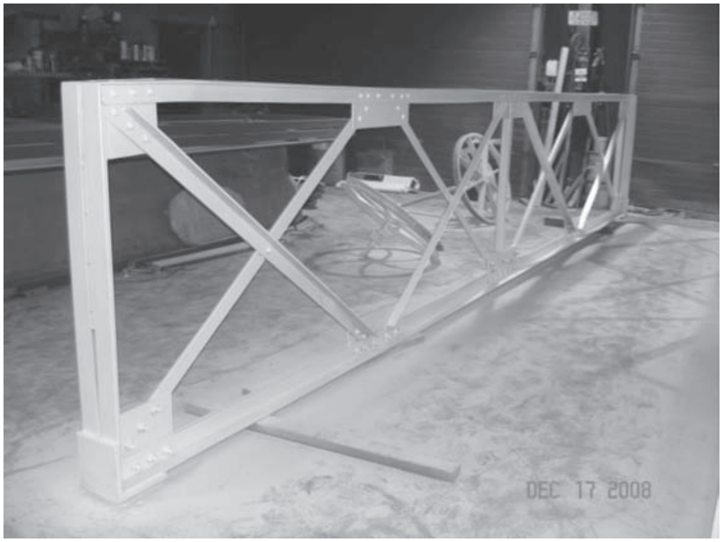


Figure 16. View of rehabilitated truss after prime coat was applied in the shop. Note shafts and gearing in the background.

to determine the configuration of replacement pieces that had long since disappeared. The extent of the repairs to the steel elements required careful detailing to be consistent with existing bridge elements. Truss members such as angles, structural tees and plates were measured and replaced exactly in kind (see Figure 16). Several angles to be replaced were of a size that is no longer rolled, and so were replaced with metric size angles that more closely matched the original pieces. For this effort, seven new sheets of steel details were produced.

4.5.2 *Bolts and connections*

The existing trusses are slender elements connected mostly with rivets. During the demolition we found there were several locations where square head bolts were used instead of rivets. Rivets that were required to be replaced due to deterioration of the rivet or the connecting member were replaced with round head bolts and the existing square head bolts that could not be reused were replaced with new square head bolts.

4.5.3 *Bridge bearings*

The existing cast-iron bridge bearings uncovered during demolition were found to be ornate and an integral part of the bascule span (see Figure 17). They had been planned for replacement but these were retained. In order to do so the new concrete abutments were redesigned to accept older and larger bearings.

4.5.4 *Bridge color*

The color of the bridge prior to construction was silver, but this was not the original paint color. Under the silver coat, and still very well intact in many areas, was found the original lead paint coatings. The client and others noted the importance of painting this structure in keeping with its natural surroundings and using a color that it had been in the past. Local agencies including the village government and the historic society were contacted to determine and agree upon the color the bridge would be painted. Eventually the bridge will be painted with a color to match as closely as possible to the color that was found on the existing steel members, namely Pantone 492U which is a brick red semi-gloss epoxy paint system (see Figure 18).

Figure 17. Existing movable bridge bearing uncovered during the removal of the existing draw span.

Figure 18. Paint sample for approved three coat system. Note rehabilitated sheave assembly in the background.

5 CONCLUSIONS

There are many potential unforeseen difficulties that are encountered during bridge rehabilitation projects. Rehabilitation of historic movable bridges adds several level of difficulty. Close coordination is required between the Engineer, Contractor, Owner, and the various stakeholders that have an interest in the project. During demolition it is necessary to accurately catalogue parts, sizes, fits and alignment since there are no available drawings. All parts should be piece marked to facilitate future erection of the rehabilitated components. Care must be taken at every step to guarantee the integrity of bridge components during the removal and rehabilitation. While the effort required may be painstaking the results are important. A snap shoot in time will be preserved for the community enjoyment. Bridge Street will be a place to sit and reflect on the past. Figures 19 and 20 show the North approach to the near complete construction of The Bridge Street Bridge.

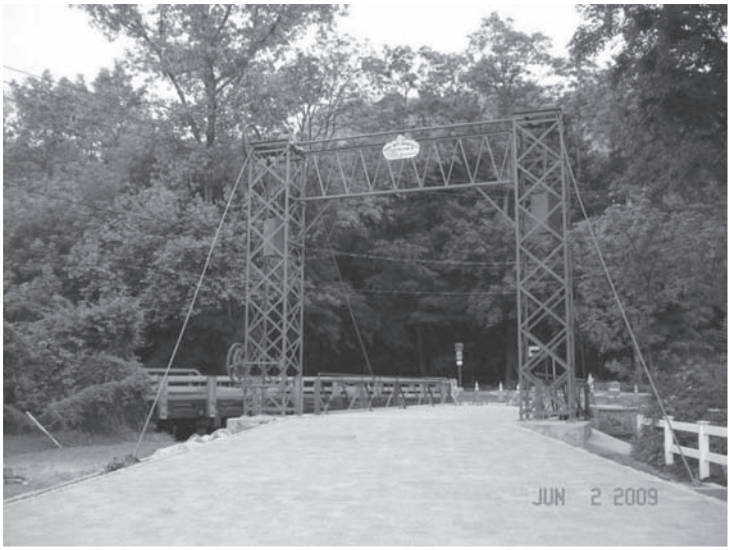


Figure 19. Substantial completion of the Bridge Street Bridge. Note the paving blocks used for the north approach.

Figure 20. Bridge Street Bridge looking northeast. Similar view to Figure 1.

REFERENCES

1. Fraser D.J. & Deakin M.A.B. Curved Track Bascule Bridges from Castle Drawbridge to Modern Application—7th Historic Bridge Conference.
2. King Bridge Company Website: http://www.kingbridgeco.com/
3. Piermont Public Library Website: http://piermontlibrary.blogspot.com/

Chapter 13

Replacement of a rare Hanover skewed bascule—The Hamilton Avenue Bridge, New York City, USA

K. Griesing
Hardesty & Hanover, LLP, New York, New York, USA

ABSTRACT: The rarely seen Hanover skew bascule, also known as a knee-girder bascule bridge is a unique and complex movable structure in terms of both design and construction. The replacement of a movable bridge during an accelerated construction period is also an incredibly difficult task to engineer and construct. Either one of these constraints would make a project difficult to execute. For the Hamilton Avenue Bridge project in New York City, however, these two levels of complexity combined to create a one-of-a-kind project that would challenge the owner, designers and constructor to achieve a near impossible goal: to replace a skewed bascule bridge with a new, fully operational span in sixty-four days.

This paper will address the design constraints of this project with respect to the complexity of a skewed bascule bridge. The impact of the highly constrained project site and schedule, 64 day construction window, will also be discussed. These aspects combined to make this $60M project unique and a highly challenging endeavor for all parties in the project team.

1 INTRODUCTION

At approximately 3 AM on August 31, 2007, four lanes of traffic heading northbound on Hamilton Avenue flowed freely across the Gowanus Canal. This may seem uneventful even though traffic is usually heavy at this early hour but it is especially notable since the traffic crossed the canal over the new East Span of the Hamilton Avenue Bridge. The new span was constructed during a traffic closure of only 64 days. During the closure the existing bridge was demolished and a new span constructed while traffic was maintained through a detailed maintenance and protection of traffic plan. The replacement of a relatively small urban arterial bridge in such a short time period may not be remarkable but this was not a typical bridge nor was this a typical, simple bridge project.

The existing Hamilton Avenue Bridge was constructed in 1945 based on the novel knee-girder bascule design patented by Clinton D. Hanover of the NYC Department of Public Works and later namesake of the movable bridge engineering firm, Hardesty & Hanover. The bridge comprises two separate single leaf bascule bridges with each leaf carrying four lanes of Hamilton Avenue. The East Span carries traffic northbound to the Brooklyn Battery Tunnel and lower Manhattan and the West Span traffic southbound to lower Brooklyn. The bridge carries approximately 55,000 vehicles per day with a high percentage of truck traffic. The traffic includes both commuters from Brooklyn to the businesses of Manhattan as well as commercial traffic for delivery of goods and services throughout the five boroughs.

The bridge lies directly beneath the Gowanus Expressway which stands approximately 30 m overhead and is the entrance point to the Gowanus Canal (See Figures 1, 2). The canal was formerly a working waterway of Brooklyn with numerous industrial and commercial businesses utilizing the waterway for access. The changing face of Brooklyn has led to a reduction in the use of the canal for commercial purposes for most of its length. This shift in use, however, has not occurred at the southern end (entrance end) of the canal.

The area between the Hamilton Avenue and Ninth Street Bridges continues to thrive as a commercial center. This segment of canal is the home of a major fuel oil distributor, a material reclamation center, and a concrete production plant. These three facilities rely extensively on the

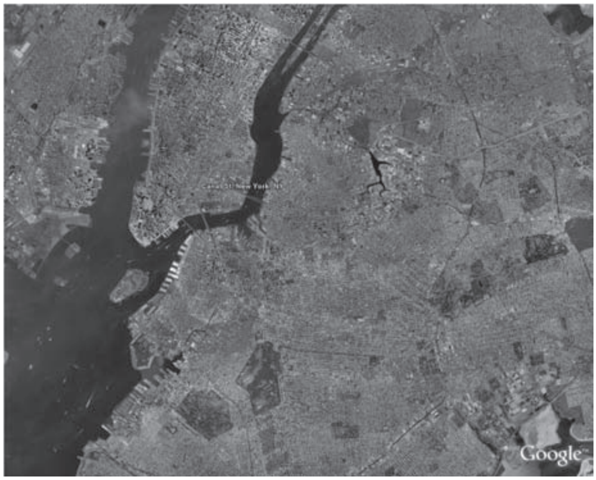


Figure 1. Project location.

Figure 2. Project site.

canal for the movement of material and the nature of these three businesses result in activity on the waterway throughout the year with no seasonal lulls. On average, the bridge opens approximately 900 times per year to clear vessels through the channel. The bridge is staffed full time by operations personnel from the New York City Department of Transportation, the bridge owner, and opens on-demand for vessels.

2 BRIDGE CONCEPT DEVELOPMENT

The knee-girder bascule spans currently under reconstruction are the second bridges at the site. The prior structure was a rolling lift bridge with a skewed deck. (See Figure 3). This type of framing was common for moderately skewed crossings with the deck in the shape of a parallelogram to match the channel crossing. The main structural elements of the rolling lift span (girders) are normal to the channel and are not parallel to the edges of the deck.

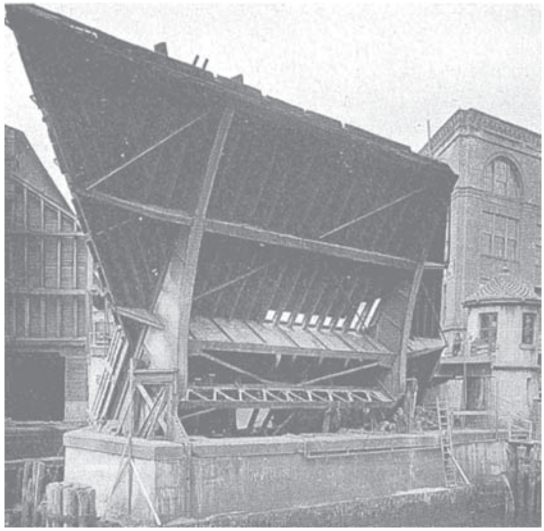


Figure 3. Original skewed deck bridge in open position.

There are a number of disadvantages to this framing configuration. The two large triangular portions of deck are cantilevered from the bascule girders with varying length floorbeam extensions. This results in variable flexibility of the deck system. In addition, the girders must be placed closer together than desirable in order to circumscribe the parallelogram about the rectangle formed by the girders and end floorbeams. This reduces the available space for both the counterweight and operating machinery. The efficiency of the bracing system is also moderately reduced but this is not of significant impact.

As the original bridge exhibited operational issues due to the design deficiencies noted above, which were compounded by the rolling lift nature of the span, Hanover, as lead engineer for the Department of Public Works of New York City, sought to improve the solution for the skewed crossing. In order to achieve this Hanover assessed the impacts of a skewed crossing on a movable structure and developed a list of criteria for the "ideal" solution. The keys criteria were:

- Effective span length should be as short as possible
- Piers should be parallel to the channel in order to disturb flow as little as practical
- Main supporting elements should be parallel to the roadway
- Main supporting elements should be normal to the channel to allow trunnion axis to be parallel to and as close as possible to the channel
- The loads on the two main supporting elements should be nearly equal to mitigate differential operating loads and potential racking of the span
- Length of the counterweight should be as large as possible and should utilize normal weight concrete and uniform fill.

Hanover studied numerous double and single leaf bascule options to meet these criteria. The end result was the development of a knee-girder single leaf bascule span in which the main supporting elements are bent at rigid joints to accommodate the skew (See Figure 4). This concept was patented by Hanover in 1943 prior to joining Waddell & Hardesty in 1945.

3 BRIDGE THEORY AND ANALYSIS

As with any bascule bridge, the location of the center of gravity of the moving mass is of critical importance to the operational efficiency and performance of the bridge. This importance is

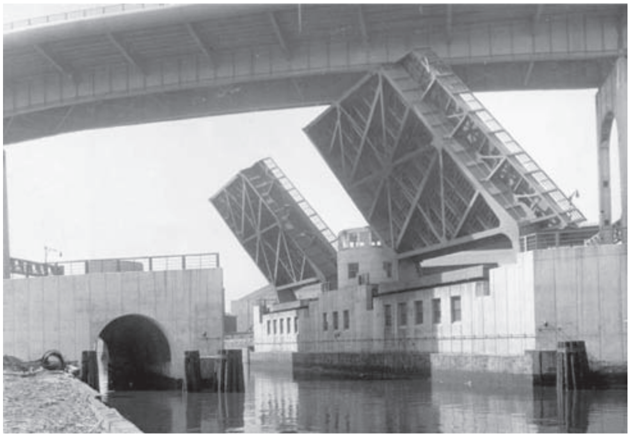


Figure 4. Bridge circa 1945.

accentuated in the knee-girder bascule design due to the geometric properties of the structure. It is readily seen that the center of gravity of the forward portion of the leaf is laterally much closer to the inner trunnion than the outer trunnion. From this it is easy to conclude that the center of gravity of the counterweight should be much closer to the inner trunnion. This location of the "natural" center of gravity of the counterweight would yield extremely heavy loads on the inner trunnion and substantially reduce the advantage of the large counterweight created by the rear portion of the bascule girders, referred to as tail girders. On the contary, if the center of gravity of the counterweight was placed at the geometric center of the counterweight the span would be subject to racking and warping. This would yield torsional stresses in the bascule girders and be highly undesirable.

The framing of the forward portion of the span and the inclusion of the rigid knee joints eliminates the conditions noted above. This is done by using the cross girder (heel end floorbeam) as the key component of the structural system.

The means for load equalization can be seen by analyzing the forces at the knee joints (See Figure 5). The cross-girder carries moments from the tension and compressive forces in the bascule girder flanges at the knee joint. Since the angular relationship of the bascule girder and tail girder portions for the two sides are equal, the moments induced in the cross-girder also act in the same sense and are additive. The sum of these moments is equal to the moment of the forward arm about the centerline of the rear arm and is independent of the framing of the floorbeams.

To maintain equilibrium in the cross-girder, the outer girder must exert a downward force on its end of the cross-girder and the inner girder must exert an upward force on its end. The counterbalancing forces of the cross-girder are then an upward force on the outer girder and a downward force on the inner girder at the knee joints. These forces tend to locate the center of gravity of the span nearer to the inner trunnion and the "natural" center of gravity discussed above.

This tendency is counteracted by the framing layout of the floorbeams. The floorbeams connect to the outer main girder at points further from the knee joint than the corresponding ends do on the adjacent girder. As the deck weight is nearly uniformly distributed, this results in higher moments at the knee joint of the outer bascule girder than at the inner bascule girder. This differential in moments between the inner and outer girders is offset by the cross-girder forces noted above.

In addition, due to the uniform distribution of deck weight, the vertical shear forces at the knee joints for the inner and outer knee joints are equal (or nearly so depending on the overall symmetry of the span cross section). Thus, in order to balance the moments in the girders, the center of gravity of the counterweight would need to shift from the "natural" center of gravity discussed previously to a location nearer to the geometric center of the counterweight. The combination

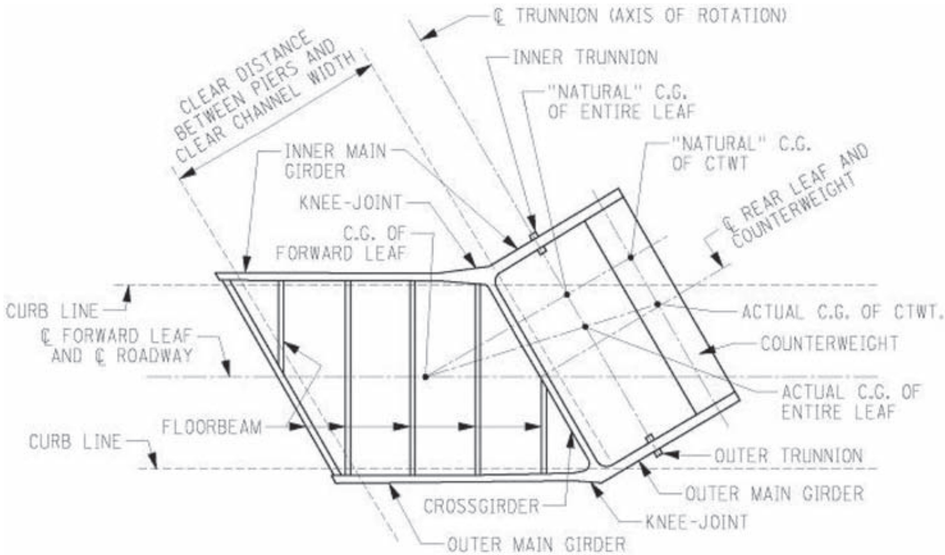


Figure 5. Key elements of the structural system.

of the cross-girder effect at the knee joint and the framing of the floorbeams relative to the knee joints alters the cumulative loads so that the loads at the trunnion are nearly equal and the center of gravity of the counterweight is located nearer the geometrical center.

The spacing of the floorbeams can be optimized in order to locate the center of gravity of the span on the geometric centerline. In order to achieve this, however, the floorbeams would need to be skewed based on their relative distances from the knee joints. For practical purposes and to minimize the number of skewed connections, a compromised solution is utilized and the floorbeams are kept orthogonal to the girders.

This innovation design allowed the bridge to be specifically adapted to the crossing with a minimal use of material. In addition, simple shear connections are used throughout with the exception of the knee joint connections of the cross-girder, bascule girder (forward portion) and tail girder (rear portion).

4 PROJECT SUMMARY

As part of its ongoing bridge evaluation and maintenance program, the bridge owner, the New York City Department of Transportation (NYCDOT), conducted an in-depth evaluation of the existing bridge in 1998. The conclusion of the study, performed led by prime consultant Greenman-Pedersen, Inc. (GPI) and Hardesty & Hanover as the movable bridge specialists, was that the bridge superstructure possessed a number of nonconforming roadway features (lane widths, bridge railings, etc) and was functionally obsolete. The original bridge was designed by the firm of Waddell & Hardesty who would later be named Hardesty & Hanover after Hanover joined the firm in 1945.

Additionally, the bridge machinery and electrical systems were nearing the end of their useful service life. The bridge substructure exhibited signs of age but the robust original design permitted the existing substructure to be retained. Based on these conditions, NYCDOT determined a superstructure replacement was the most cost-effective solution to improve both the span operation as well as the flow of traffic on Hamilton Avenue and in the canal beneath the movable bridge.

During the study phase of the project, July and August were determined to be the lowest vehicular traffic months. The reduced traffic was a result of the summer school recess (less school bus and car travel to schools) during the summer months. As a result, NYCDOT concluded a roadway closure for one direction of Hamilton Avenue was feasible for these two months and the bridge superstructure replacement must be completed in this 64 day period.

During the roadway closure period, traffic would be maintained on the adjacent roadway using bi-directional traffic on the formerly one-way roadway. Contraflow lanes were used to address peak northbound traffic demands (morning peak traffic).

During this roadway closure period, only intermittent navigation closures were permitted due to the continuous demand of the waterway users. The constrained site, limited construction duration and active waterway, combined with the inherent complexity of the skewed bascule design, made this project a highly complex enterprise for the project team.

5 REPLACEMENT BRIDGE DESIGN

The replacement bridges share many of the same geometric characteristics of the original spans (See Figure 6). Each leaf spans the Gowanus Canal with a clear channel width of 14.3 m between the fenders. The clear distance between the bascule and rest piers is 15.5 m with the piers oriented parallel to the channel. The roadway is skewed to the channel crossing at an angle of 56.3° and includes four-3.35 m wide lanes (total roadway width of 13.4 m on each leaf increased from 12.8 m on the original). Each leaf also includes a 600 mm access walkway for operation and maintenance personnel at the median side and a 2.45 m sidewalk at the outboard fascia for pedestrians and cyclists.

The bascule girders are spaced 13.4 m center to center and the distance from the centerline of trunnion to the toe bearings is 20.4 m. The girder and floorbeam spacing was maintained due to the interaction of the movable span with the existing pier to remain. The wider traffic lanes on the

Figure 6. New east span-gowanus expressway above.

new bridge result in a wider out-to-out dimension of the leaf. This required modifications to the bridge seat at the rest pier and the front wall of the bascule pier to accommodate the new span. The new span weighs approximately 600 metric tons and opens to an angle of 76°.

The bridge maintained the knee-girder design developed by Mr. Hanover (See Figure 7). The elegance of the original design was recreated during the design phase of this project. The classical design methodology developed by Hanover was used to initially proportion the main structural members. This analysis and design was combined with more modern finite element methods and computer modeling. The results of the two methods had a very high level of correlation which reinforced the soundness of the assumptions of the original design concept.

From a structural and machinery standpoint, the rear portion of the leaf (portion behind the cross girder) is similar to most simple trunnion style bascule bridges. The rear portion comprises the two tail girders and trunnions supported on steel trunnion towers and the counterweight for span balance. The bridge is operated by an electric motor driven gear train. Large rack gears with a radius centered on the trunnion shaft are attached to the bottom flange of each bascule girder. The rack is engaged by the main drive pinion and the adjoining open and enclosed reducer gearing. Due to spatial constraints in the retained bascule pier, an external gear (See Figure 8) was utilized to minimize the size of the enclosed gear reducers (See Figure 9) while maintaining the necessary gear reduction. The bridge also includes auxiliary operation through the used of a hydraulic motor (powered by an hydraulic power unit in the bascule pier) attached to the input of the primary reducer. Hand operation through the use of a capstan is also possible so the bridge can be operated under even the most unfortunate power outages.

The span also includes electrically driven span locks at the toe end to prevent unintended opening of the span. Lateral guides at the toe of the span and at rear of the counterweight serve as centering devices for span seating as well as shear blocks for lateral loads anticipated for seismic load cases.

Precision control of the leaf's speed and ability to resist cumulative applied moments will be provided by motor controllers which will be silicone controlled rectifier (SCR) thyristor drive. The bridge power system includes an auxiliary generated housed in a newly constructed building near the control house. The generator provides auxiliary electrical power in an event of power failure from the main power utility.

A programmable logic controller (PLC) will direct and monitor bascule span operation, including warning gates, barrier gates, traffic signals and toe span locks. Submarine cables are used for power, control and communication across the canal.

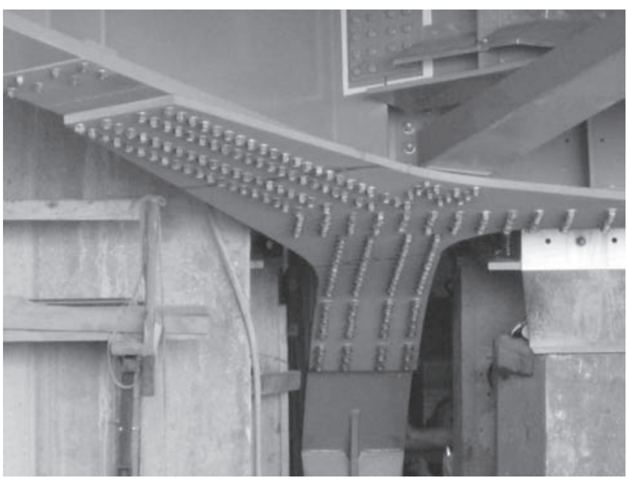


Figure 7. Interior knee joint connection.

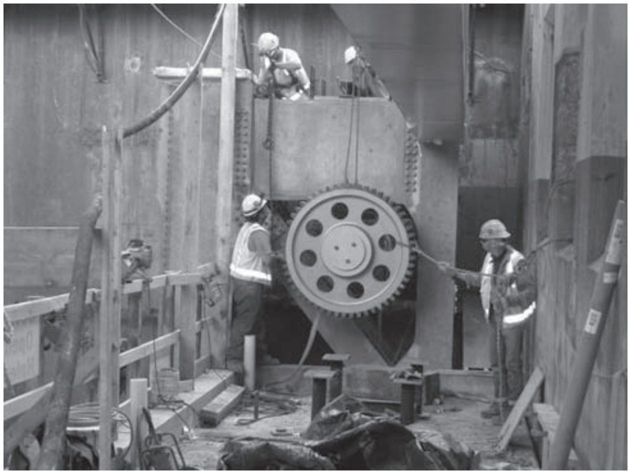


Figure 8. Trunnion tower and gearing.

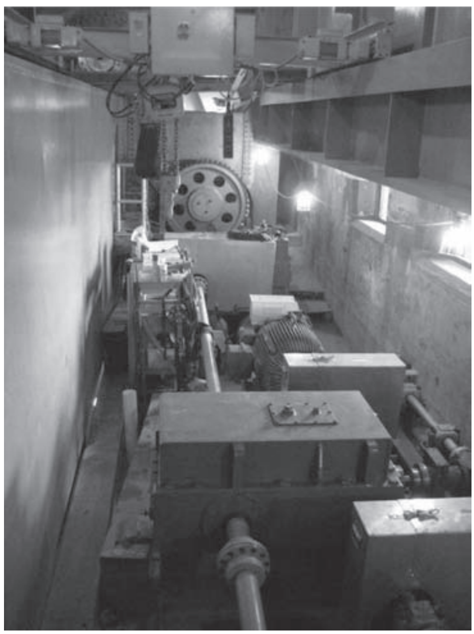


Figure 9. Machinery room.

6 PROJECT STATUS

The construction phase of the project began in August 2005 with the site preparation and mobilization phase. During this phase, extensive project technical and teaming meetings were held amongst NYCDOT, the designers and the contractor to prepare for the first roadway closure period in July/August 2007. The overall schedule includes replacement of the East Span in summer 2007 followed by the West Span in summer 2008. The period between the two closures (September 2007 through June 2008) was used for the rehabilitation of the architectural elements including the bridge operator's control house and the gate tender's house as well as the construction of an auxiliary generator house. The architectural work was designed by the project prime design consultant, GPI. The project is scheduled for completion in early 2009 after final movable bridge operational testing and commissioning.

Due to the limited construction duration of the closure period (62 days), the design included the use of temporary hydraulic drive motor system for the operation of the span. As an alternative proposal, the contractor opted to utilize hydraulic cylinders attached to the rear of the tail girders for span operation. The use of a temporary operating system permitted the bridge to be returned to traffic use and operation for marine traffic without the final alignment of the gear train. The final alignment work is highly specialized and requires skilled millwrights to ensure proper alignment. It was determined by the project team that the long term interests of the bridge were better served by performing this work after the 62 day closure period.

In July and August 2007, the East Span was removed and replaced with the new span. The work proceeded according to the anticipated schedule except for a few unforeseen issues. First, the key end floorbeam of the bascule span was damaged due to a truck accident while shipping to the project site. This required the entire floorbeam and its bolted attachments to be disassembled, inspected both visually and through other non-destructive means. Minor repairs were required prior to reshipment. One of issue encountered was with the trenching for the submarine cables. The permit required the use of an environmental clamshell bucket to mitigate the sedimentation of the disturbed canal bottom. As the work proceeded it was determined the environmental bucket could not penetrate the dense mud of the canal bottom. Through extensive discussions with the environmental agencies an alternative means of mitigation was developed and a standard clamshell bucket used.

The West span was removed and replaced in July and August 2008. The lessons learned from the lessons learned from the prior were utilized by the Contractor to more efficiently perform the field work and the West Span was completed in advance of the milestone date. The contract required full shop of assembly of the structure of the span. As an option, the contractor chose to include the roadway grid deck in the shop assembly to ensure alignment of the panels during the field assembly during the tight roadway closure period. This advance work permitted the contractor to rapidly erect the steel and deck system to achieve the goals of the milestone.

The alignment of the span operating machinery was performed in the fall/winter 2008. The shop assembly (See Figure 10) and preliminary alignment was beneficial in achieving the desired tolerances for the final field installation. To verify alignment, field operational tests were performed

Figure 10. Shop assembly of west span.

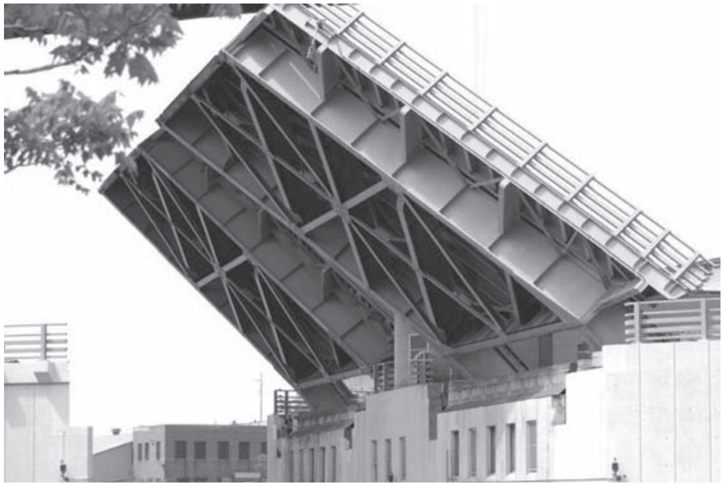


Figure 11. New east span with existing west span.

utilizing the auxiliary drive system (hydraulic motor) to power the gear train. Concurrent with this machinery work, the final field installation and commissioning of the electrical power distribution and span control system was performed. The bridge control system and equipment was installed in the renovated control house and the bridge systems were brought on-line and tested through a rigorous functional check-out program. Once each individual system was functioning, the final operational function tests commenced. The bridge functional checkout is currently ongoing and final acceptance and project close-out is expected in April 2009 (See Figure 11).

7 CONCLUSIONS

The reconstruction of the Hamilton Avenue Bridge was successful due to the collective effort of the project team. The owner provided clear objectives and the contractor and the various engineering consultants involved were dedicated to achieving these objectives for this complex movable bridge project. With this project, NYCDOT has embraced the historic design of the original bridge and build on this history to provide a new structure that will serve the traveling needs of the public well into the future.

ACKNOWLEDGEMENTS

New York City Department of Transportation – Owner
Greenman-Pedersen, Inc. – Prime Consultant
Hardesty & Hanover, LLP – Movable Bridge Designer
Kiewit Constructors, Inc. – General Contractor
Urbitran Associates – Resident Engineering Services

REFERENCES

Hanover, Jr., Clinton, D., A New Type Bascule for Skewed Crossings, *Engineering News-Record*, June 1944.

Hanover, Jr., Clinton, D., New Knee-Girder Bascule Design for Skewed Crossings, *Roads and Bridges*, October 1944.

Hanover, Jr., Clinton, D., Skew Bascule Bridge over the Gowanus Canal, *Engineering News-Record*, pp. 25–27. July 1944.

Chapter 14

Innovative redesign for cost savings on a vertical lift bridge

E. Kelly & W. Nickoley
HNTB Corporation, New York, New York, USA

ABSTRACT: After bids came in 73% higher than the Engineer's Estimate, the U.S. Coast Guard selected HNTB to perform a Value Engineering Review of a movable rail bridge over the Mobile River in Alabama. HNTB used their extensive experience on movable bridges to examine the foundations, superstructure, constructability as well as the mechanical and electrical systems to develop cost savings throughout structure.

This paper will discuss the original structure configuration, Contractor's input regarding the erection, staging, closures to rail and marine traffic associated with the original design, and main details of the redesign which resulted in cost savings.

The paper will discuss the cost effective redesign performed by HNTB to lower the estimated construction cost of the bridge without sacrificing any of the design requirements.

Topics discussed in the paper will be as follows:

- High cost details of original bridge
- Cost effective alternatives
- Details of alternate design
- Cost comparison of the two alternate structures

1 PROJECT BACKGROUND

1.1 *Existing bridge* (*See Figure 1*)

The existing structure is a single track, 330 foot (2 @ 165 feet) center bearing swing span that provides one clear channel of 146'-7" over the Mobile River. To the east of the swing span, the structure has 800 feet of timber approach trestle and one 208 foot variable depth truss span. To the west there are two 208 foot variable depth truss spans and an 80 foot long through girder span. The superstructure

Figure 1. Elevation of existing bridge.

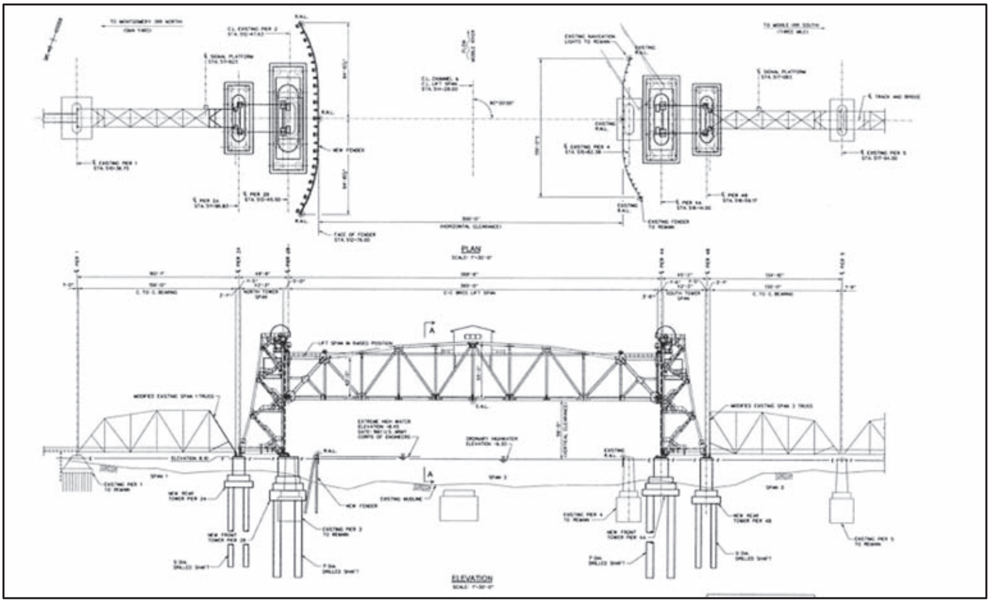


Figure 2. General plan and elevation of proposed replacement structure (CSX Plans, 2006).

is supported on un-reinforced concrete piers with soil bearing footings. Due to river geometry, the structure has been hit numerous times causing interruptions in rail and navigation traffic, along with an extensive amount of repairs. Pier 4 had previously been struck shearing the pier horizontally and requiring extensive repairs. The navigational clearance was determined to be inadequate by the United States Coast Guard and thus the agency requested CSX, the owner, to replace the movable span to provide more navigation clearance, funded under the Truman Hobbs Act (CFR 33, 2008).

1.2 *Original design for replacement structure* (*See Figure 2*)

The criteria for the replacement structure was to provide a 56 foot 5 inch vertical clearance over a 300 foot wide channel. A 365 foot through-truss, span-drive vertical lift bridge was determined to be the optimum solution by another consultant. The design incorporated a new pier behind Pier 4 and the existing Pier 2 was strengthened for the new lift span. Drilled shafts were used for the foundation of the new Pier 4A and strengthened Pier 2. The towers for the lift span were each to be four leg trussed steel elements. The rear tower legs would be supported on their own new separate piers. All piers were to be constructed utilizing cofferdams. A 42 foot tower span would be placed within each tower to span between the front and rear tower legs. To accommodate the towers for the lift span, the two fixed truss spans on either side of the swing span required modification to shorten their overall length. Existing Pier 4 with a recently replaced fender system was to remain and be utilized for pier protection. A new fender system was to be installed at Pier 2.

2 BIDS AND RESULTING WORK

2.1 *Bid tabulations*

The project was advertised on February 24, 2006. Questions were raised during bidding including a letter from one of the prospective bidders indicating certain requirements in the contract specifications made the cost of the project higher than anticipated. Three bids were received on May 4, 2006 with the low bid at $30 million above the Engineer's Estimate (See Figure 3). Since

BID TABULATION

PROJECT: Replace Exsting Swing Span with Vertical Lift Span

Item No.	Bid Item Description	Bid Quantities		Estimated Bid		Contractor 1		Contractor 2		Contractor 3	
		Est'd Qty.	Unit	Unit Price	Total Amount	Unit Price	Total Amount	Unit Price	Total Amount	Unit Price	Total Amount
2	Mobilization	1	L.S.	$1,629,000	$1,778,542.20	$6,990,000.00	$6,990,000.00	$7,830,000.00	$7,830,000.00	$9,000,000.00	$9,000,000.00
2A	Partnering	1	L.S.	$0.00	$0.00	$96,500.00	$96,500.00	$60,000.00	$60,000.00	$56,000.00	$56,000.00
3	Environmental Protection	1	L.S.	$10,000	$10,918.00	$140,000.00	$140,000.00	$150,000.00	$150,000.00	$50,000.00	$50,000.00
4A	Drilled Shaft (7' dia)	1,952	L.F.	$1,365	$2,909,079.26	$1,374.00	$2,682,048.00	$2,730.00	$5,328,960.00	$2,320.00	$4,528,640.00
4B	Drilled Shaft (5' dia)	1,328	L.F.	$740	$1,072,933.70	$701.00	$930,928.00	$2,255.00	$2,994,640.00	$1,240.00	$1,646,720.00
4C	Drilled Shaft 7' Casing	432	L.F.	$196	$92,444.89	$1,200.00	$518,400.00	$1,460.00	$630,720.00	$1,600.00	$691,200.00
4D	Drilled Shaft 5' Casing	448	L.F.	$143	$69,945.08	$1,076.00	$482,048.00	$1,040.00	$465,920.00	$705.00	$315,840.00
4E	Exploratory Borings	295	L.F.		$0.00	$202.00	$59,590.00	$150.00	$44,250.00	$200.00	$59,000.00
4F	Protection of Existing Structures	1	L.S.		$0.00	$45,000.00	$45,000.00	$100,000.00	$100,000.00	$975,000.00	$975,000.00
4G	Statnamic Axial Load Test	1	EA.	$70,000	$76,426.00	$500,000.00	$500,000.00	$1,000,000.00	$1,000,000.00	$135,000.00	$135,000.00
5A	Structural Excavation	6,700	C.Y.	$40	$292,602.40	$149.00	$998,300.00	$100.00	$670,000.00	$72.00	$482,400.00
5B	Cofferdams	4	EA.	$508,750	$2,221,813.00	$2,083,627.00	$8,334,508.00	$1,500,000.00	$6,000,000.00	$3,980,000.00	$15,920,000.00
6	Reinforcing Steel	449,600	LB	$1.20	$589,047.94	$2.10	$944,160.00	$1.10	$494,560.00	$1.06	$476,100.00
7A	Concrete Class A (AE) (Substructure)	4,800	C.Y.	$499	$2,615,079.36	$842.00	$4,041,600.00	$1,100.00	$5,280,000.00	$1,100.00	$5,280,000.00
7B	Concrete Class A (AE) (Slabs)	56	C.Y.	$642	$39,252.39	$1,350.00	$75,600.00	$975.00	$54,600.00	$1,200.00	$67,200.00
7C	Concrete Class A (AE) (Misc)	5	C.Y.	$554	$3,024.29	$825.00	$4,125.00	$775.00	$3,875.00	$3,300.00	$16,500.00
7D	Concrete Class B (AE) (Tremi Seal)	2,600	C.Y.	$499	$1,416,501.32	$600.00	$1,560,000.00	$825.00	$2,145,000.00	$1,040.00	$2,704,000.00
8A	Structural Steel (Lift Span)	2,200,000	C.Y.	$2.70	$6,485,292.00	$5.63	$12,386,000.00	$6.30	$13,860,000.00	$4.70	$10,340,000.00
8B	Structural Steel (Lift Span Misc)	240,000	LB	$2.25	$589,572.00	$6.75	$1,620,000.00	$6.55	$1,572,000.00	$5.50	$1,320,000.00
8C	Structural Steel (Towers)	1,700,000	LB	$2.98	$5,531,058.80	$5.75	$9,775,000.00	$6.55	$11,135,000.00	$5.65	$9,605,000.00
8D	Structural Steel (Towers Misc)	180,000	LB	$2.25	$442,179.00	$8.00	$1,440,000.00	$4.60	$828,000.00	$8.65	$1,557,000.00
8E	Structural Steel (Counterweights)	640,000	LB	$1.40	$978,252.80	$2.40	$1,536,000.00	$4.75	$3,040,000.00	$5.40	$3,456,000.00
8F	Structural Steel (Existing Truss Mods)	25,400	LB	$11.68	$323,906.49	$12.00	$304,800.00	$17.75	$450,850.00	$34.00	$863,600.00
10	Off-Line Trackwork	460	L.F.	$572	$287,274.42	$1,223.00	$562,580.00	$1,860.00	$855,600.00	$800.00	$368,000.00
11	Demolition of Existing Structure	1	L.S.	$1,798,000	$1,963,056.40	$675,000.00	$675,000.00	$1,100,000.00	$1,100,000.00	$2,000,000.00	$2,000,000.00
13	Machinery/Tender's House	1	L.S.	$212,000	$231,461.60	$150,000.00	$150,000.00	$250,000.00	$250,000.00	$150,000.00	$150,000.00
15	Span Drive Machinery	1	L.S.	$1,527,000	$1,667,178.60	$2,600,000.00	$2,600,000.00	$5,000,000.00	$5,000,000.00	$3,000,000.00	$3,000,000.00
16	Span Lock Machinery	1	L.S.	$382,000	$417,067.60	$400,000.00	$400,000.00	$600,000.00	$600,000.00	$600,000.00	$600,000.00
17	Uphaul & Downhaul Ropes & Accessories	1	L.S.	$255,000	$278,409.00	$500,000.00	$500,000.00	$600,000.00	$600,000.00	$200,000.00	$200,000.00
18	Counterweight Sheaves, Shafts & Bearings	1	L.S.	$2,926,000	$3,194,606.80	$3,650,000.00	$3,650,000.00	$4,500,000.00	$4,500,000.00	$5,000,000.00	$5,000,000.00
19A	Structural Concrete in Counterweight	630	C.Y.	$554	$381,060.04	$750.00	$472,500.00	$700.00	$441,000.00	$640.00	$403,200.00
19B	Scrap Metal for Counterweight	50,000	LB	$0.60	$32,754.00	$1.00	$50,000.00	$3.00	$150,000.00	$2.00	$100,000.00
19C	Balance Blocks	135,000	LB	$0.35	$51,587.55	$1.25	$168,750.00	$0.30	$40,500.00	$0.90	$121,500.00
19D	Span Balance	1	L.S.	$53,000	$57,865.40	$100,000.00	$100,000.00	$600,000.00	$600,000.00	$200,000.00	$200,000.00
20	Counterweight Ropes and Accessories	1	L.S.	$647,000	$706,394.60	$700,000.00	$700,000.00	$1,000,000.00	$1,000,000.00	$1,700,000.00	$1,700,000.00
21A	Bridge Electrical System	1	L.S.	$1,728,000	$1,886,630.40	$2,000,000.00	$2,000,000.00	$2,100,000.00	$2,100,000.00	$2,300,000.00	$2,300,000.00
21B	Aerial Cables	1	L.S.	$196,000	$213,992.80	$200,000.00	$200,000.00	$300,000.00	$300,000.00	$350,000.00	$350,000.00
22	Generator Building	1	L.S.	$18,000	$19,652.40	$500,000.00	$500,000.00	$40,000.00	$40,000.00	$215,000.00	$215,000.00
23	Bridge Standby Generator	1	L.S.	$400,000	$436,720.00	$250,000.00	$250,000.00	$175,000.00	$175,000.00	$200,000.00	$200,000.00
26	Operation & Maintenance Manuals & Training	1	L.S.	$53,000	$57,865.40	$2,000.00	$2,000.00	$20,000.00	$20,000.00	$100,000.00	$100,000.00
27	Fender Work	1	L.S.	$486,400	$531,051.52	$500,000.00	$500,000.00	$750,000.00	$750,000.00	$700,000.00	$700,000.00
28	Site Work	1	L.S.	$265,000	$289,327.00	$1,000,000.00	$1,000,000.00	$425,000.00	$425,000.00	$2,433,000.00	$2,433,000.00
	*(includes 3% escalation + 6% contingency)										
	Bridge Total =				**$40,241,826.43**		**$69,945,437.00**		**$83,085,475.00**		**$89,685,900.00**
	Days to Complete										
BOND	Performance Bond (if required)	1	L.S.	0.00		0.5333	$373,000.00	0.4735	$393,400.00	0.5229	$469,000.00
	Grand Total For Project (w/ bond) =				$40,241,826.43		$70,318,437.00		$83,478,875.00		$90,154,900.00

Figure 3. Bid tabulations from advertisement of proposed replacement structure (CSX Bid, 2006).

the variation was too great to award, the bids were rejected. The Coast Guard then contacted three consultants to solicit bids to perform a Value Engineering of the project in an effort to find cost saving suggestions. HNTB Corporation was selected for the assignment.

2.2 *Value engineering and proposal*

HNTB's approach to the work began with assessment of the contract plans and examination of the bid tabulations to determine the most costly items. HNTB did this by breaking the project up into four major components. These were Substructure, Superstructure, Mechanical & Electrical, and Construction. For each of these components, costliest items were examined to determine how they might be reduced.

2.2.1 *Superstructure and substructure* (*See Figure 4*)

The cost savings approach to the overall bridge focused on the interrelationship between the superstructure and tower substructure. The major cost of the substructure was two fold.

- The need to utilize cofferdams for all the pier construction.
- The quantities of drilled shafts, concrete and reinforcing steel being utilized for the construction of the piers.

HNTB's proposed solution was to utilize single steel box towers in lieu of the four legged towers thereby eliminating the need for the rear tower legs, tower piers, tower jump spans and the modifications required to the fixed approach steel truss spans. Although the quantity of steel for the box sections is comparable to the four column tower, its foot print is significantly smaller.

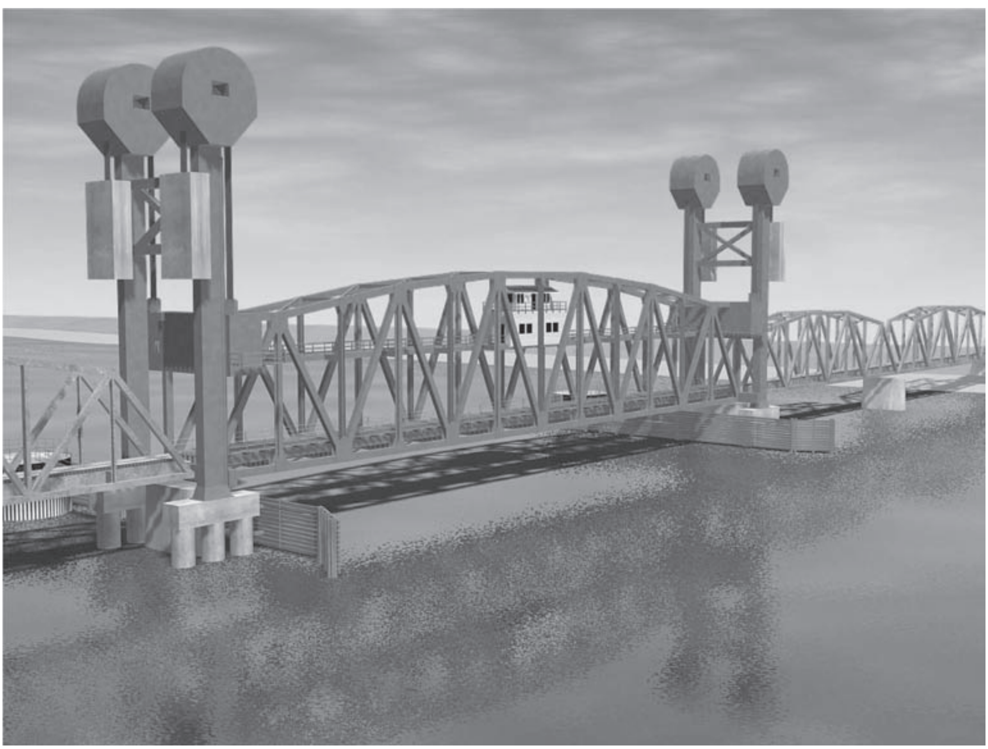


Figure 4. Rendering of conceptual arrangement of VE structure (HNTB, 2007).

HNTB also proposed to utilize waterline footings on drilled shafts to avoid the costly cofferdam construction. A portion of the new footings was underwater and thus floating forms were required similar to what has been used on other of HNTB Designs. With the form partially submerged, the form acts as a tub withstanding the hydrostatic pressure of the water and, once the concrete is poured, to hold the concrete in position while curing.

The replacement of the large deep piers with pier caps allowed significant reduction in the quantities of concrete and reinforcing. This also resulted in less dead load for the drilled shafts to carry. This, in turn, reduced the quantity of the drilled shafts.

2.2.2 *Lift span superstructure*

HNTB developed a more economical span length for the superstructure by placing the new footing for Pier 2 outside of the existing Pier 2 and getting the new pier 4A as close to the existing Pier 4 as possible. This could be done because the new drilled shafts are away from the existing piers. By doing this, the span length was able to be reduced from 365 feet to 357 feet. To still allow the new lift span to be supported on the new portion of the footings, the span width had to be increased to approximately 38 feet wide from the original 22 feet wide. This required the floorbeams to be heavier to accommodate the wider span. By utilizing this reduced span length, the truss modification to the existing eastern fixed approach span was no longer required.

2.2.3 *Mechanical and electrical*

Examination of the mechanical and electrical system revealed that the motors to lift the bridge were undersized in terms of the original design, and would be operating 60% over their full load capacity (AREMA, 2000). HNTB developed two alternates to accommodate the system as originally designed. The first alternate was to enlarge the motor to the proper size, but this would result in the need for a complete redesign of the mechanical and electrical systems. The second alternate was to develop a scheme to reduce the operating loads. This was done by changing the bronzed bushed counterweight sheave bearings to roller bearings. By doing so, the frictional load was dramatically reduced and overall operating loads were reduced.

2.2.4 *Construction*

During the bidding phase of the project, the contractors expressed concern with the requirements for minimizing the fouling of the tracks. Strict railroad guidelines exist that any crane or equipment has to be clear of the tracks a certain time before a train is passing. Since this was a main line track, the approximately 20 trains per day expected would result in the crane being stored more than utilized, resulting in a very inefficient and costly use of the cranes. The original tower design required the erection of numerous truss members within the track envelope resulting in costly erection. In contrast, HNTB's proposed single towers, which are located outside the new lift span truss panels, could be partially erected off site and installed in sections, resulting in a more efficient erection procedure and less time with the cranes deployed within or around the track fouling area.

Utilization of the water line footings resulted in multiple advantages from a construction standpoint. First it eliminated all need for cofferdams. These cofferdams were very expensive due the tight regulation on not interrupting rail traffic as well as the very restrictive clearance underneath the existing approach span where the cofferdam had to be installed. The placement of the cap beams outside of the existing piers allows their construction to occur with minimal interference to rail operations. This will expedite their construction assisting in lowering the unit cost of material installation. The unit price for the concrete would increase slightly to compensate for the necessity of utilizing the more costly form system.

2.2.5 *Comparison of costs*

Once the structure type was developed and major details were worked out for the Value Engineering Alternate, quantities were determined to compare the existing and new designs and establish the cost savings. HNTB developed a spreadsheet that showed the original unit prices, original

Bridge No. 193 @ M.P. 653.44
(Hurricane, AL)
Comparison of Quantities between Bid and VE Recommendations

Item No.	Items Affectedby VE	Quantities				Unit Price	Savings
		AS BID		HNTB			
4A	Drilled Shaft (7' dia)	1,952	L.F.	1,808	L.F.	$1,374.00	$197,856.00
4B	Drilled Shaft (5' dia)	1,328	L.F.	0	L.F.	$701.00	$930,928.00
4C	Drilled Shaft 7' Casing	432	L.F.	400	L.F.	$1,200.00	$38,400.00
4D	Drilled Shaft 5' Casing	448	L.F.	0	L.F.	$1,076.00	$482,048.00
5A	Structural Excavation	6,700	C.Y.	0	C.Y.	$149.00	$998,300.00
5B	Cofferdams	4	EA.	0	EA.	$2,083,627.00	$8,334,508.00
6	Reinforcing Steel	449,600	LB	130,700	LB	$2.10	$669,690.00
7A	Concrete Class A	4,800	C.Y.	1,307	C.Y.	$842.00	$2,941,106.00
	Cost Differential for Floating Forms	0	C.Y.	1,307	C.Y.	$252.60	-$330,148.20
7D	Concrete Class B	2,600	C.Y.	0	C.Y.	$600.00	$1,560,000.00
8A	Structural Steel (Lift Span)	2,200,000	LB	2,320,000	LB	$5.63	-$675,600.00
8B	Structural Steel (Lift Span Misc)	240,000	LB	240,000	LB	$6.75	$0.00
8C	Structural Steel (Towers)	1,700,000	LB	1,320,000	LB	$5.75	$2,185,000.00
8D	Structural Steel (Towers Misc)	180,000	LB	180,000	LB	$8.00	$0.00
8E	Structural Steel (Counterweights)	640,000	LB	640,000	LB	$2.40	$0.00
8F	Structural Steel (Existing Truss Mods)	25,400	LB	10,160	LB	$12.00	$182,880.00
19A	Structural Concrete in Counterweight	630	C.Y.	660	C.Y.	$750.00	-$22,500.00
27	Fender Work	$500,000	L.S.	$500,000	L.S.		-$0
New Item	**Existing Pier Rehabilitation**	**$0**	**L.S.**	**$250,000**	**L.S.**		**-$250,000.00**
						Total Cost Savings	$17,242,467.80

Figure 5. Potential cost savings from VE review (HNTB, 2007).

quantities and the new quantities for the proposed VE Alternate Structure. This provided a side by side comparison of both structures to clearly indicate where the cost savings could be achieved (See Figure 5).

3 FINAL DESIGN

Upon approval of the revised structure based on the Value Engineering study, CSX and USCG pursued an agreement with HNTB Corporation to provide final design services. To the extent possible, this work was to utilize the former design as a basis. The new dual bid package was to provide both the original proposed replacement structure, redesigned with improvements obtained through the VE, along with the HNTB proposed structure. It was to be advertised having two bids with the lower cost of the two bids determining which structure was to be built.

While a good share of the existing design could be reused, the entire design required review and acceptance by HNTB as its own. Existing calculations, especially, were reviewed carefully. Significant review of constructability issues also led to changes in the staging as presented in the bid documents. This review led to further discovery that the former design would not have allowed the existing swing span to operate after the towers were erected, so additional work had to be done to provide clearance. Also, in addition to the under-sizing of the motors, the former electrical design also lacked sufficient information to keep the bids low. Improvements were thus required by all disciplines.

The new work also included addressing additional concerns regarding the vulnerability of the piers and towers to collisions from barges. A Vessel Impact Study was added to the contract to assess this vulnerability, and to provide protection for the new structure. The Study approached the problem under two scenarios—that of a fully loaded barge impacting the substructure, and that of an empty or partially loaded barge mounting the new piers and impacting the towers while riding high in the water. It was found that the existing fender to remain was insufficient to withstand either event, and was useful only to guide a vessel that may be in need of redirection. It was also found that the design of a stronger fender, either to replace the existing fender to remain or to provide new protection on the other side of the channel, would not be cost effective. Instead, the drilled shafts and piers were found capable of sustaining the high impact of a fully loaded barge, and the bases of the towers would be fortified to protect them from high-riding empty barges.

The superstructure design for the towers includes external access to the lift span from any position, plus internal access to the tower columns and struts for inspection purposes. These towers will be fabricated and erected mainly off-site so that the least amount of time is spent working within or above the traffic envelope. The new piers and substructure provide the same benefit during construction.

Regarding the operating system, the use of antifriction bearings was selected to keep the required size of the motors and machinery components the same as originally designed. A new mounting system for the sheaves was required at the tops of the new towers as well as redesign of the trunnion shafts to meet the configurations required by the new bearings. The mounting of the span locks and the centering devices also required redesign.

Submittal of the 90% review package of both alternates revealed that even with the refinements from the VE study, the cost of the original proposed replacement structure could not compete with the HNTB alternate. The tabulated estimate for the HNTB structure was approximately \$8 million dollars less than the revised proposed replacement structure. Due to the significant disparity between the costs of the structures, it was determined that the two costs would not be competitive and therefore the bid package would only have one structure advertised. This would be the HNTB structure (See Figure 6).

Even without accounting for inflation, the improvements made to the substructure to withstand vessel impact, a wider lift span, and the upgrading of the undersized operating system, a savings of several million dollars is anticipated. The new alternative design was completed in early February, 2009. At the time of this writing, the project is being advertised for bid.

Figure 6. Rendering of HNTB proposed final design alternate (HNTB, 2009).

4 CONCLUSIONS

This project emphasizes the need for bridge designers to carefully assess all project constraints when preparing designs. As in this case, such constraints can go beyond the physical characteristics of the site, the configuration and condition of existing structures, the function of the new or modified facility, the availability of labor, materials and equipment, and the usual codes and standards used for design. Other possible constraints can include local or owner-specific standards and rules (particularly with respect to safety on railroads) and the practicality of normally accepted construction techniques in specific applications. The challenge to identify such constraints by investigation early on, and to think creatively in order to meet them, can help to avoid unanticipated cost overruns and delays.

REFERENCES

AREMA, 2000, American Railway Engineering and Maintenance-of-Way Association *Manual for Railway Engineering*, Section 6.7.5.5, Motor Torque for Span Operation.

CFR 33, 2008, Code of Federal Regulations (USA), 33 CFR 116 (Truman-Hobbs Act, 1940).

CSX Plans, 2006, CSX Transportation Plans for the Modification of Bridge 193 at Mile Post 653.44 Crossing the Mobile River Near Hurricane, AL (February, 2006).

CSX Bid, 2006, CSX Transportation/US Coast Guard.

HNTB, 2007, HNTB Corporation, *CSX Bridge 193 Mobile River Value Engineering Report*.

HNTB, 2009, HNTB Corporation, renderings.

4 Advanced materials

Chapter 15

Lightweight concrete for long span bridges

R. Castrodale & K. Harmon
Carolina Stalite Company, Salisbury, North Carolina, USA

ABSTRACT: Lightweight concrete has been used for long span bridges for many years, including its recent use in some major structures in the US and overseas. This paper begins with a brief overview of lightweight aggregate and lightweight concrete and the properties affecting construction, structural performance and durability. Issues to consider when using lightweight concrete for long-span bridges are then presented. Selected long-span bridge projects on which lightweight concrete has been used are discussed, including bridges where lightweight concrete has been used for the entire superstructure and for only the deck, including the suspension spans of the San Francisco-Oakland Bay Bridge and the Chesapeake Bay Bridge and some examples of long-span segmental concrete bridges in the US and Norway. Several projects where lightweight concrete was used for rehabilitation of long-span bridges are discussed, including the Brooklyn Bridge and the Newburgh-Beacon Bridge which are in the New York City area.

1 INTRODUCTION TO LIGHTWEIGHT CONCRETE

1.1 *Definition of lightweight concrete*

Lightweight concrete is a structural concrete in which some or all of the coarse and fine aggregate has been replaced with aggregate that is lighter than normalweight aggregate. Structural lightweight concrete in the US generally uses lightweight aggregates that are manufactured using a rotary kiln process. The density of lightweight concrete typically varies from about 100 to 125 pcf. For the upper end of this range, "sand lightweight" concrete is used, which is concrete in which the coarse aggregate is lightweight but the fine aggregate is normalweight. To reach the lower end of the range, "all lightweight" concrete is used, in which all of the aggregate is lightweight. Currently, "sand lightweight" concrete is more commonly used, since it provides a significant weight reduction at a reduced cost compared to "all lightweight" concrete. Densities between 125 pcf and normalweight concrete can also be achieved using a blend of lightweight and normalweight coarse aggregates. See the ACI Committee 213 report (2003) and a report by Holm and Bremner (2000) for more basic information on lightweight aggregate and lightweight concrete.

1.2 *Types of manufactured lightweight aggregate*

While structural lightweight concrete can be made using natural aggregates such as pumice and scoria, almost all lightweight concrete for bridges has used expanded aggregates that are manufactured from shale, clay and slate as the raw material. After crushing and grading, the raw material is fed into a rotary kiln, where it is heated to 1,800 to 2,300 deg. F. At these temperatures, the raw material softens and expands as gases are released. The released gas forms many small, mostly discontinuous, pores which remain as the material cools and hardens after leaving the rotary kiln. The result is a vitrified, inert material that is significantly lighter than the raw material, yet still retains much of its strength. The aggregate may or may not be crushed to obtain the desired particle shape and size for use as aggregate in concrete.

Expanded aggregates manufactured using different types and sources of raw materials and processed using different methods will have somewhat different properties. However, with proper attention to qualification of aggregates, quality control and mixture proportions, many structural grade lightweight aggregates have been successfully used for concrete bridge decks and several have been successfully used in the construction of long-span prestressed concrete bridges.

1.3 *Properties of lightweight aggregate*

Manufactured lightweight aggregate is simply a concrete aggregate that weighs significantly less than typical aggregates. While it has some properties that differ from normalweight aggregates, it is used in essentially the same way as conventional aggregates.

The basic requirement for a manufactured lightweight aggregate to be used in structural concrete is that it meets the requirements of ASTM C 330 (2004a). This specification requires that the aggregate can be used to produce a concrete compressive strength of at least 2,500 psi, and that the maximum dry loose density is 70 pcf for fine aggregates, 55 pcf for coarse aggregates and 65 pcf for the loose density for the combination of coarse and fine aggregates. The Owner or Engineer may specify other properties such as soundness, abrasion resistance and freeze-thaw resistance, which are also required for normalweight aggregate.

The relative density (also known as “specific gravity”) of manufactured lightweight aggregates typically ranges from 1.3 to 1.6, while the relative density for normalweight aggregates typically ranges from 2.6 to 3.0. Therefore, a given volume of aggregate weighs about half as much for lightweight aggregate than for normalweight aggregate.

Because of their cellular structure, lightweight aggregates absorb more water than normalweight aggregates. Based on 24 hour tests, lightweight aggregates typically absorb from 5% to more than 25% by mass of dry aggregate (Holm & Bremner 2000), while absorption of normalweight aggregates is typically less than 2%. With adequate premoistening of lightweight aggregates and proper consideration of the absorption in mixture proportioning, batching and control, lightweight concrete can be consistently produced with the workability and mechanical properties required for bridge construction.

2 PROPERTIES OF LIGHTWEIGHT CONCRETE

To be successfully used in a long-span bridge, the properties of a material must be understood for all stages of the life of the structure, including construction, structural performance and the long-term durability of the structure. The properties of lightweight concrete related to these stages in the life of a bridge are discussed in this section.

2.1 *Construction*

It has been widely noted that lightweight concrete can be used for bridge construction in the same ways as normalweight concrete. A significant difference in construction using the two types of concrete is that lightweight aggregate must be premoistened prior to batching to satisfy the greater absorption of the lightweight aggregate. When properly proportioned, lightweight concrete can be batched, transported, placed, finished and cured using the same methods and equipment as for normalweight concrete. Because of its reduced density, it can be placed and screeded with less physical effort than that required for normalweight concrete. Excessive vibration should be avoided to prevent driving the heavier mortar fraction down from the surface where it is required for finishing. The same attention to curing should be taken for lightweight concrete as for normalweight concrete.

2.2 *Structural*

The basic structural property specified for all concrete is its minimum compressive strength. For lightweight concrete, the maximum density of the concrete is also specified. The equilibrium

Table 1. Fresh and equilibrium densities for range of concrete compressive strengths.

	Lightweight concrete*		Normalweight concrete**
Compressive strength ksi	Fresh density pcf	Equilibrium density pcf	Density pcf
4.5 (Deck)	114.7	110.1	145
6	118.3	114.4	146
8	120.2	117.3	148
10	123.6	121.6	150

* LWC densities are fresh and equilibrium densities from "Suggested Mix Designs" at www.stalite.com.
** NWC densities are computed using the equations for in AASHTO LRFD Table 3.5.1-1.

density as defined in ASTM C 567 (2004b) (similar to an "air-dry" density) is usually specified, although the fresh density is also needed for quality control and acceptance. Definitions for these terms, as well as a discussion of how to specify lightweight concrete, are given by Castrodale & Harmon (2008a). The range of strengths and densities that can be achieved using expanded slate aggregate are shown in Table 1.

Structural properties of concrete that are affected by the reduced density of lightweight concrete include:

– Tensile strength
– Shear
– Development of reinforcement
– Creep
– Shrinkage
– Modulus of elasticity
– Prestress losses

Current bridge design specifications address these issues, providing guidance for the designer in properly using lightweight concrete. For some long-span structures, designers have specified minimum criteria for some of these properties that the lightweight concrete must meet for the adequate performance of the project.

2.3 *Durability*

Lightweight concrete has proven to be a very durable construction material for bridges. However, many engineers are hesitant to use lightweight concrete in bridge construction because of their perceived concerns about the durability of lightweight concrete exposed to weather and traffic conditions experienced by bridges. They suspect that lightweight concrete, made with what appears to be a porous aggregate, could not provide durability that is comparable with concrete made with normalweight aggregate. However, considerable evidence from long-term field performance and laboratory tests indicates that lightweight concrete can provide the excellent durability that is needed for bridges (Castrodale & Harmon, 2008b). This is illustrated by the concrete permeability test results from a bridge deck recently constructed for the Virginia DOT, which are shown in Table 2.

3 DESIGN CONSIDERATIONS FOR LONG-SPAN LIGHTWEIGHT CONCRETE BRIDGES

This section presents the ways in which the use of lightweight concrete can affect the design of a long-span bridge. The advantages and disadvantages of the use of lightweight concrete will be noted.

Table 2. Concrete permeability for the VA Route 33 bridge over the Pamunkey River*.

	Permeability at 28 days coulombs	
Specification requirements	2500	
Average	989	17 samples over 6 months
Maximum value	1467	
Minimum value	593	
Standard deviation	245	

* Castrodale & Harmon 2008b.

3.1 *Construction*

When elements are precast, lightweight concrete can significantly reduce handling, shipping and erection requirements and costs. In some cases, the number of truck loads of elements can be reduced, which can improve traffic congestion near a construction site and reduce pollution.

3.2 *Structural*

Using lightweight concrete for any bridge element reduces the total structure weight. This reduction in dead load may result in overall project cost savings by improving the structural efficiency of the bridge in the following ways for new construction:

– Increased bridge span lengths, possibly reducing the number of substructure units
– Reduced superstructure mass, reducing seismic loads on substructure and foundation
– Reduced substructure and foundation loads, possibly reducing the size and/or number of foundation elements, including piles
– Improved load balancing for cantilevered construction when span lengths differ significantly on either side of a pier
– Improved load balancing between spans of continuous structures when a span layout must be used that is not ideal for balancing dead loads
– Reduced reinforcement and prestressing in superstructure and substructure elements
– Wider girder spacings, possibly reducing the number of girders required for a span
– Reduced design forces because of increased substructure flexibility when lightweight concrete is used for substructure elements
– Reduced requirements for bridge bearings
– The use of lightweight concrete can also lead to benefits for the rehabilitation of bridges:
– Increased deck width and/or thickness with little or no modification to the superstructure and substructure
– Increased live load rating using the same superstructure and substructure
– Reuse of substructure elements with a lengthened or widened structure

The effect of the decreased stiffness of lightweight concrete must be considered in bridges, since it may result in several effects, including increased deflections and increased elastic shortening and camber of prestressed members. The slightly reduced tensile strength of lightweight concrete may also have some effect on bridge designs.

3.3 *Durability*

Durability is an essential and critical aspect of the performance of a long-span bridge. To preserve the proper and economical functioning of the structure throughout its design life, the structure and any concrete used in it must meet all of the necessary material and structural requirements. To be durable, concrete must resist the intrusion of water, chlorides and other materials that can lead to corrosion or other processes that can lead to the deterioration of the structure. Therefore, the concrete must have low permeability, few if any cracks, resistance to wear, retain adequate skid resistance, and be able to resist or prevent deterioration from freezing and thawing and other chemical attack.

Castrodale & Harmon (2008b) have shown that lightweight concrete can satisfy durability and other performance requirements for the following issues which are critical for bridges:

– Freezing-thawing resistance
– Permeability
– Alkali-silica reactivity (ASR) and sulfate attack
– Carbonation
– Fire resistance
– Fatigue resistance
– Coefficient of thermal expansion
– Abrasion resistance
– Skid resistance

In the same paper, the durability enhancements found in lightweight concrete are attributed to one or more of the unique features of lightweight concrete which include:

– Internal curing by slowly releasing moisture absorbed into the aggregate prior to batching
– Elastic compatibility, where the stiffness of the aggregate and paste are more closely matched, which eliminates stress concentrations and micro-cracking
– Enhanced bond of aggregate and paste, because of coarse surface texture of aggregate particles and slightly pozzolanic nature of the surface of the aggregate
– Reduced stiffness of lightweight concrete, which allows the concrete to accommodate greater strains prior to cracking

Well-proportioned lightweight concrete using a qualified aggregate that has been properly mixed, conveyed, placed, consolidated and finished should prove to be very durable, generally performing better than similar normalweight concrete.

3.4 *Cost*

Lightweight concrete almost always costs more than normalweight concrete because of the additional cost for processing and shipping the manufactured lightweight aggregate. The price of manufactured lightweight aggregate is sensitive to energy costs since the manufacturing process requires a significant input of energy to heat and expand the raw materials. Because the number of plants currently producing structural lightweight aggregate in the US is limited, transportation costs can be a significant component of the total cost of lightweight aggregate.

It is difficult to make a general statement regarding the cost premium for lightweight concrete compared to normalweight concrete because of differences in the cost of lightweight and normalweight aggregates and factors related to shipping and handling of lightweight aggregate. The additional cost for lightweight concrete, compared to a normalweight concrete with the same basic characteristics, typically ranges from \$20 to \$65 per cubic yard, with higher costs when the aggregate must be shipped a long distance.

Although the material cost of lightweight concrete is greater than normalweight concrete, the advantages of using reduced density concrete for a bridge design can offset the additional cost. To obtain a clear understanding of the difference in cost between a normalweight and specified density concrete design, it is important to consider the impact of the use of reduced density concrete on all elements of the bridge, including bearings, substructure units and foundations. The potential cost savings for handling, transportation and erection of precast concrete elements, as well as long-term savings resulting from the enhanced durability of lightweight concrete, should also be considered.

The number of bridges that have incorporated lightweight concrete demonstrates that its use can reduce overall project costs. An example is the Rugsund Bridge in Norway where use of lightweight concrete, for which the aggregate was imported from the US, resulted in a 15% lower bid price for the lightweight concrete design alternate than for the original normalweight concrete bridge alternate (Harmon 2004). A design comparison using normal and lightweight concrete performed by Bender (1980) for a precast segmental box girder bridge demonstrated a nearly 15% overall cost reduction when using lightweight concrete. See Castrodale & Harmon (2005) for more discussion and comparisons regarding the cost of lightweight concrete bridges.

4 USE OF LIGHTWEIGHT CONCRETE IN THE INITIAL DESIGN OF LONG-SPAN STEEL BRIDGES

4.1 *San Francisco-Oakland Bay Bridge—CA, USA*

Sand lightweight concrete was used for the upper roadway deck for the suspension spans on the famous San Francisco-Oakland Bay Bridge, which has main spans of 2,310 ft. Using lightweight concrete for the upper deck in this bridge, which was completed in 1936, reduced the dead load by 25 lbs per square foot for a total weight reduction of 31.6 million pounds for the entire structure (ESCSI 1971). This decreased dead load made possible the reduction in area and cost of members in the superstructure, as well as reducing foundations loads and stresses on foundations and superstructure due to seismic forces. In all, the cost savings from using lightweight concrete were estimated at $3 million for the $77 million dollar total original cost.

The 5⅜ inch thick deck was initially protected by a ½ inch layer of hard rock mortar (Andrews 1953a). Although the overlay has been replaced several times over the years, the deck is still in service today. When the bridge was reconfigured for two-way traffic in 1961, sand lightweight concrete was also used for the new lower deck.

4.2 *Tacoma Narrows Bridge—Tacoma, WA, USA*

The Tacoma Narrows Bridge was opened to traffic in 1950 as the replacement for the previous bridge at the site which collapsed in 1940 and has become known as "Galloping Gertie." The replacement structure used a sand lightweight concrete deck which allowed the designers to reuse the exiting towers while widening of the bridge deck from two lanes to 4 lanes (Andrew 1953b). The bridge deck was 5⅜ inch thick and was initially protected with a ⅝ inch asphalt wearing surface. The deck is still in service today. The bridge was the third longest suspension bridge in the world when it was completed, with a main span of 2,800 ft.

The savings in construction material and costs by using lightweight concrete have been reported to be (ESCSI 1953):

– Savings in structural steel	735,000 lbs
– Savings in suspension cables and wire	725,000 lbs
– Savings in floor weight	4,120,000 lbs
– Savings in cost	$320,000

4.3 *William Preston Lane, Jr. Memorial Bridges over Chesapeake Bay—Annapolis, MD, USA*

The William Preston Lane Jr. Memorial Bridge, popularly called the Bay Bridge, carries US-50/US-301 over the Chesapeake Bay near Annapolis, MD. A 4.5 mi long two-lane bridge was opened to traffic in 1952. The main span of the suspension bridge portion of the structure is 1600 ft long. The deck slab on the suspension spans was all lightweight concrete with a design strength of

3.5 ksi and an air dry density of 105 pcf. The deck was replaced in 1988 after 36 years of service. The remaining portion of the bridge consists of beam spans and deck truss sections and was constructed with normal weight concrete decking (Vaysburd 1996).

With the completion of a second bridge at the site in 1973, the original structure was used to carry eastbound traffic and the newer structure carries westbound traffic. The new structure has a 6.5 in. thick lightweight concrete deck on the truss and suspension spans, and a 7 in. thick normalweight concrete deck on the girder span approaches. The newer suspension span provides a 1500 ft horizontal clearance (Bay Bridge Overview Team 2005). In 2006 a project was initiated to replace the deck of the westbound suspension bridge which was constructed in 1973 (Wolfe 2008). The deck on this bridge had a service life of 33 years, similar to the performance on the older bridge. Precast lightweight concrete panels are being used for the new deck.

4.4 *Mark Clark Expressway over the Cooper River—Charleston, SC, USA*

This 10,900 ft long bridge that carries the Mark Clark Expressway (I-526) over the Cooper River and tidal marshes was completed in 1992. The deck of the entire structure was lightweight concrete. The main river crossing is a through truss with three 800 ft spans. For the steel truss spans and the adjoining steel girder spans, the lightweight concrete deck was placed on stay-in-place steel deck forms, while prestressed lightweight concrete stay-in-place panels were used to form the lightweight concrete deck on the prestressed concrete girder approach spans.

4.5 *Coleman Bridge over the York River—Yorktown, VA, USA*

This major steel deck truss crossing the York River at Yorktown, VA, includes two 500 ft long swing spans. Lightweight concrete was used for the decks on the truss spans, which were constructed on falsework in Norfolk, VA, then barged as completed units to the site for erection. The lightweight concrete decks were cast on removable forms. Installation of the truss bridge segments was completed in a single nine day closure. Normalweight concrete was used for the decks on the approach spans (Castrodale & Robinson 2008).

While this project was a rehabilitation project, it involved the complete replacement of the superstructure. The main truss spans of this bridge were replaced and widened. Lightweight concrete was used in the bridge deck of the new structure to reduce the weight of the deck, resulting in a reduced quantity of steel required for the trusses. With the reduced superstructure weight, the existing piers could be widened without requiring the driving of additional piles. Lightweight concrete had been used successfully for decks on other bridges in Virginia, so the DOT was open to its use on this major crossing. The same specification was used for the lightweight concrete deck as the standard VDOT normalweight concrete deck, with the exception of a higher concrete compressive strength (4,500 psi instead of 4,000 psi) to account for the reduced shear capacity of the lightweight concrete and a density of 115 pcf (Abrahams 1996).

The lightweight concrete deck on the truss spans was ground as required to achieve the desired roadway profile and rideability, then transversely grooved for skid resistance. This work was completed prior to final erection of the trusses to make possible the rapid opening of the bridge to traffic. The surface of the deck remains exposed to traffic.

During a visit to the bridge in 2005 by one of the authors, no cracks were visible on the top or underside of the lightweight concrete deck. However, the normalweight concrete deck on the approach spans, which was conventionally constructed onsite, had suffered from significant transverse cracking at an early age and had been repaired. It was learned from a VDOT inspector familiar with the bridge that the lightweight concrete deck had performed better than the normalweight concrete deck. It should be noted, however, that the conditions for placing the two types of concrete decks were probably different, which may have affected the performance of the decks (Castrodale & Robinson 2008).

5 USE OF LIGHTWEIGHT CONCRETE IN THE INITIAL DESIGN OF LONG-SPAN CONCRETE BRIDGES

5.1 *Benicia-Martinez Bridge over the Carquinez Strait, CA, USA*

The new Benicia-Martinez Bridge is a high level bridge structure that carries one direction of I-680 across the Carquinez Strait approximately 30 miles northeast of San Francisco. Several different structure types were considered in preliminary and final design, but the designers determined that a lightweight concrete segmental box girder bridge was most economical solution.

The lightweight concrete reduced the mass of the superstructure up to 20% and was more flexible. These two factors helped to minimize the seismic forces on the foundations and resulted in significant cost savings in the foundations.

The total bridge length is 8,790 ft, of which 6,500 ft are a cast-in-place segmental box girder units that use lightweight concrete for all but the pier segments. The spans vary in length from 276 ft to 659 ft and vary in depth from 37 ft at the piers to 15 ft at midspan. The designers specified a number of material properties, including a minimum compressive strength of 6500 psi, an equilibrium density of 125 pcf, a modulus of elasticity not less than 3.4×10^6 psi, a specific creep not to exceed 70×10^{-6}/MPa, a shrinkage strain not to exceed 0.05%, and a tensile strength not less than 450 psi. To allow for an aggressive casting schedule, the contractor developed a mix design that gained strength very rapidly. This mix achieved a compressive strength of at least 10 ksi at 28 days and met or exceeded all of the specified properties (Castrodale & Harmon 2008a).

5.2 *Stolma Bridge, Hordaland, Norway*

This bridge is representative of several long-span concrete box girder bridges that have been constructed in Norway. The three-span bridge was constructed using the balanced cantilever technique with a main span of 301 m and side spans of 94 m and 72 m. When completed in 1998, the bridge held the world record span for construction of this type.

Lightweight concrete was used in the center 184 m of the main span. Use of lightweight concrete in this portion of the bridge allowed designers to achieve a better balance for dead loads between the main and anchor spans. The lightweight concrete was grade LC 60 while the normal-weight concrete was grade C65 (Melby 2000).

5.3 *Sandhornøya Bridge, Nordland, Norway*

This long-span concrete box girder bridge also used both lightweight (LC 65) and normalweight (C45) concrete in the box girder. However, because the 110 m side spans were relatively long compared to the 154 m main span, lightweight concrete was used in the side spans to improve the balance of dead loads in the bridge (Melby 2000).

5.4 *Nordhordaland Bridge, Hordaland, Norway*

This long-span concrete bridge was completed in 1994 combines a floating structure with lightweight concrete pontoons with a 369 m long concrete cable-stayed terminal structure that uses lightweight concrete for the deck in its main span. Jakobsen (2000) estimates that using lightweight concrete for the pontoons saved between 3.3% and 7.5% of the cost of the floating portion of the bridge, because the smaller lightweight concrete pontoons reduced the wave forces, which reduced the quantity of steel in the box girder superstructure. Fergestad & Jordet (2000) report that using lightweight concrete for the cable stayed structure saved about 0.83% of the total contract cost. They noted that the savings mainly came from reduced cost of the stays and hold down structure. Even though the savings appear minor, they feel that it still provided noticeable savings even for this relatively small bridge.

6 USE OF LIGHTWEIGHT CONCRETE IN THE REHABILITATION OF LONG-SPAN BRIDGES

6.1 *Newburgh-Beacon Bridge over the Hudson River—NY, USA*

In 1963, the two lane Newburgh-Beacon Bridge opened spanning the Hudson River. Traffic on this bridge quickly increased, making a second bridge necessary. In 1980, a second bridge with three lanes was opened and the original bridge was widened to three lanes. The bridges, which each have an overall length of about 7800 ft (2377 m), including a 1000 ft (305 m) through truss main span, now carry I-84 over the Hudson River.

The existing normalweight concrete deck on the first bridge was replaced with a lightweight concrete deck to compensate for widening the deck from 30 to 39 feet (9.1 to 11.9 m). The new 6½ in. (165 mm) thick lightweight concrete bridge deck was covered with a 1½ inch (38 mm) latex modified concrete wearing surface. The latex modified concrete overlay has required repair in several locations. However, when the New York State Bridge Authority removed the overlay to make repairs, they found that the lightweight concrete deck was in excellent shape and only the wearing surface needed to be repaired. (Wolfe 2008).

6.2 *Brooklyn Bridge over the East River—New York City, NY, USA*

The Brooklyn Bridge is one of the most well-known bridges in the country. It is the oldest bridge spanning the East River and connects Manhattan and Brooklyn. The entire deck was replaced in an emergency contract using prefabricated panels of grid deck filled with lightweight concrete. The maximum specified concrete density was 118 pcf with a minimum compressive strength of 3600 psi. The new floor system was designed to have the same weight as the existing floor system, which was 41 psf. Panels were installed at night to avoid impacting rush hour traffic. After installation, the deck was covered with an overlay (ESCSI 2002). Construction was completed in 1999.

6.3 *Lewis and Clark Bridge over the Columbia River—Longview, WA, USA*

The Lewis and Clark Bridge crosses the Columbia River between Longview, WA and Ranier, OR. The main structure is a cantilever truss with a maximum span of 1,200 ft. The bridge was opened to traffic in 1930. The original lightweight concrete deck was replaced in 2004 using large panels with a lightweight concrete deck on a floor system of steel girders. Lightweight concrete was used to minimize the dead load on the existing truss. Using lightweight concrete also helped to reduce the weight of the largest prefabricated deck panel by about 28 kips, where the total panel weight was 184 kips. Use of large deck panels permitted the rapid replacement of the deck with minimal disruption of traffic. The design unit weight for the lightweight concrete was 119 pcf. The lightweight concrete deck received a microsilica overlay prior to installation (Weigel 2005).

6.4 *The Bourne and Sagamore Bridges over the Cape Cod Canal—MA, USA*

The Bourne and Sagamore Bridges are nearly identical bridges crossing the Cape Cod Canal. They provide the only link between Cape Cod and the mainland. Each bridge is a fixed-truss steel arch carrying four lanes of traffic. The main spans are 616 feet (187 m) with an overall bridge length of about 2,384 (727 m) for the Bourne Bridge and 1,408 (429 m) for the Sagamore Bridge. The bridges were constructed from 1933 to 1935 with a 7 inch (178 mm) lightweight concrete deck with a 2¼ inch (57 mm) bituminous concrete wearing surface. The bridge decks were replaced around 1980 after they had provided 45 years of service, which was an excellent record considering the climate and deicing chemicals to which the bridge decks had been exposed, and the concrete technology that existed in the 1930s when the decks were originally placed. The decks on both bridges were replaced with a 5 in. (127 mm) thick lightweight concrete-filled steel grid covered with a waterproofing membrane and a 2 in. (51 mm) bituminous concrete wearing surface (Philleo 1986).

6.5 *Francis Scott Key Bridge—Baltimore, MD, USA*

The Francis Scott Key Bridge is an 8,636 foot (2,632 m) structure that carries the Baltimore Beltway (I- 695) over the Baltimore Harbor at its easternmost crossing. The main span is a continuous truss structure with a 1,200 foot (366 m) main span and two 722 foot (220 m) back spans. The bridge was opened to traffic in 1977.

The bridge deck was constructed with structural lightweight concrete with a specified maximum fresh density of 112 pcf and a maximum 28 day dry density of 108 pcf. Test results for the bridge deck concrete indicated 28 day compressive strengths averaging 4,829 psi (33.3 MPa) with 8.1% air and a fresh density of 109.4 pcf (1,753 kg/m^3). The performance of the lightweight concrete deck has been impressive with no major deck work since the bridge was constructed (Wolfe 2008).

7 CONCLUSIONS

A variety of long-span bridges have been described for which lightweight concrete has been used, including construction of new bridges and rehabilitation of existing bridges. The use of lightweight concrete in these structures demonstrates that lightweight concrete can be used to improve the economy of a bridge. The performance of these structures demonstrates the good durability of lightweight concrete even when exposed to the severe conditions that can be experienced on long-span bridges. This information should be helpful for engineers designing new long-span bridges, or developing plans for the rehabilitation of existing long-span bridges.

REFERENCES

AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications, 4th ed*. Washington, DC: American Association of State Highway and Transportation Officials (AASHTO).

Abrahams, M.J. 1996. The Use of Lightweight Concrete for the George P. Coleman Bridge Reconstruction—Yorktown, Virginia. *International Symposium on Lightweight Concrete Bridges, Sacramento, CA, September 10, 1996*. Sponsored by Caltrans & Pacific Custom Materials.

ACI Committee 213. 2003. *Guide for Structural Lightweight-Aggregate Concrete (ACI 213R-03)*. Farmington Hills, MI: American Concrete Institute (ACI).

Andrew, C.E. 1953a. Use of Expanded Shale Concrete in Bridge and Ship Construction. *Expanded Shale Concrete Facts* 1(1): 4.

Andrew, C.E. 1953b. Structural Uses of Expanded Shale Concrete. *Expanded Shale Concrete Facts* 1(2): 5.

ASTM. 2004a. Specification for Lightweight Aggregates for Structural Concrete, ASTM C 330. In *Annual Book of ASTM Standards, Vol. 04.02*. West Conshohocken, PA: American Society for Testing and Materials (ASTM).

ASTM. 2004b. Standard Test Method for Determining Density of Structural Lightweight Concrete, ASTM C 567. In *Annual Book of ASTM Standards, Vol. 04.02*. West Conshohocken, PA: American Society for Testing and Materials (ASTM).

Bay Bridge Overview Team. 2005. *Bay Bridge Deck Investigation*. Baltimore, MD: Maryland Transportation Authority.

Bender, B.F. 1980. Economics and Use of Lightweight Concrete in Prestressed Structures. *PCI Journal* 25(6): 62–67.

Castrodale, R.W. & Harmon, K.S. 2005. Increasing Design Efficiency Using Lightweight Concrete for Prestressed Girder Bridges, Paper 55. *National Bridge Conference, Palm Springs, CA, October 16–19*. Chicago: Precast/Prestressed Concrete Institute (PCI).

Castrodale, R.W. & Harmon, K.S. 2008a. Specifying Lightweight Concrete for Bridges, Paper 147. *Concrete Bridge Conference, St. Louis, May 2008*. Skokie, IL: National Concrete Bridge Council (NCBC).

Castrodale, R.W. & Harmon, K.S. 2008b. Durability of Lightweight Concrete Bridges, Paper 48. *National Bridge Conference, Orlando, FL, October 2008*. Chicago: Precast/Prestressed Concrete Institute (PCI).

Castrodale, R.W. & Robinson, G.M. 2008. Performance of Lightweight Concrete Bridge Decks, Paper 75. *Concrete Bridge Conference*, *St. Louis, May 2008*. Skokie, IL: National Concrete Bridge Council (NCBC).

ESCSI. 1953. Bridges and Ships of Lightweight Concrete. 1953. *Expanded Shale Concrete Facts* 1(1): 5.

ESCSI. 1971. *Lightweight Concrete*. Washington, DC: Expanded Shale, Clay and Slate Institute (ESCSI).

ESCSI. 2002. Structural Concrete was used for Emergency Deck Replacement on Brooklyn Bridge. *Featured Project Bulletin 06-2002*.

Fergestad, S. & Jordet, E.A. 2000. The economical potential of LWAC in 4 Different Major Bridges. In Helland, S., Holand, I. & Smeplass, S. (eds.), *2nd Intl Sym on Structural Lightweight Aggregate Concrete, Kristiansand, Norway, June 18–22, 2000*: 355–364.

Harmon, K.S. 2004. Norway Bridges Using High Performance Lightweight Aggregate Concrete. In J.P. Ries & T.A. Holm (eds) *High-Performance Structural Lightweight Concrete, SP-218:* 189–197. Farmington Hill, MI: American Concrete Institute (ACI).

Holm, T.A. & Bremner, T.W. 2000. *State-of-the-Art Report on High-Strength, High-Durability Structural Low-Density Concrete for Applications in Severe Marine Environments, ERDC/SL TR-00-3*. Vicksburg, MS: US Army Corps of Engineers, Engineer Research and Development Center, Structures Laboratory. *Available for download at ESCSI.org*.

Holm, T.A. & Ries, J.P. 2007. *Reference Manual for the Properties and Applications of Expanded Shale, Clay and Slate Lightweight Aggregate*. Salt Lake City, UT: Expanded Shale, Clay and Slate Institute (ESCSI).

Jakobsen, S.E. 2000. The use of LWAC in the Pontoons of the Nordhordland Floating Bridge, Norway. In Helland, S., Holand, I. & Smeplass, S. (eds.), *2nd Intl Sym on Structural Lightweight Aggregate Concrete, Kristiansand, Norway, June 18–22, 2000*: 73–78.

Melby, K. 2000. Use of High Strength LWAC in Norwegian Bridges. In Helland, S., Holand, I. & Smeplass, S. (eds.), *2nd Intl Sym on Structural Lightweight Aggregate Concrete, Kristiansand, Norway, June 18–22, 2000*: 47–56.

Philleo, R.E. 1986. Lightweight Concrete in Bridges. *Concrete International* 8(11): 19–22.

Vaysburd, A.M. 1996. Durability of Lightweight Concrete Bridges in Severe Environments. *Concrete International* 18(7): 33–38.

Weigel, J.A. 2005. Lewis and Clark Bridge. Presentation to AASHTO Technical Committee for Construction (T-4). *2005 Annual Meeting of the AASHTO Subcommittee on Bridge and Structures, Newport, RI, June 28, 2005*.

Wolfe, W.H. 2008. Lightweight Concrete Bridge Deck Performance in Severe Climates. Paper 146. *Concrete Bridge Conference*, *St. Louis, May 2008*. Skokie, IL: National Concrete Bridge Council (NCBC).

Chapter 16

Experimental study on hybrid steel-concrete beam connected with perfobond ribs

S-H. Kim, J-H. Won & J-H. Ahn
School of Civil & Environmental Engineering, Yonsei Uinversity, Seoul, Korea

C.G. Lee
GS Engineering & Construction, Seoul, Korea

S-J. Kim
Sampyo Engineering & Construction, Seoul, Korea

ABSTRACT: This paper describes the experimental results of hybrid steel-PSC (prestressed concrete) beam connected with perfobond rib shear connectors. In order to enhance structural behaviors of hybrid beams and convenience of construction, the perfobond rib shear connectors are applied to the upper and lower plate located between a steel girder and a PSC girder instead of using the shear studs. Under the three point loading condition, various beams with a different length are tested to evaluate the flexural characteristics of hybrid beams and joints. Results show that the applied joint type with perfobond rib shear connectors has good functionality on transmitting the load to the steel and PSC parts. The tested joint is an effective system as a joint of the steel-PSC hybrid structural system.

1 INTRODUCTION

Recently, the steel-PSC (Prestressed concrete) hybrid bridges attract bridge engineer's attention due to their economical and structural advantages. In the construction of hybrid structures, the most important and basic problem arises in the connection of steel part and PSC part since the joints should clearly and effectively transmit loads to each part. Thus, the careful attention should be given to select the types of joints.

There were some researches to find an effective way connecting the different material such as steels and concretes. Hino et al. (1985) carried out the bending tests to steel-PSC beams and steel-reinforced concrete beams with three types of mechanical joints, which consist of anchor bars, shear studs, and H.T. Bolts (High tension blots). Dunai et al. (1996) evaluated behaviors of steel-concrete connections with end-plate-type under combined compressive axial forces and cyclic bending moments.

The shear stud connectors are generally used in joints to transmit the member forces between structural members with different materials. However, in this study, the new joint type with the perfobond rib shear connectors is proposed for the connection of hybrid steel-PSC beams to improve the constructional convenience and structural stiffness. By experimental tests, the static behaviors of hybrid beams with proposed joint are evaluated.

2 EXPERIMENTAL PROGRAM

2.1 *Details of tested beams*

Three beams with 3.9 m length (3.6 m span length) and two beams with 8 m length (7.5 m span length) are tested (Figures 1–2). All beams consist of two PSC parts, two joint parts and one

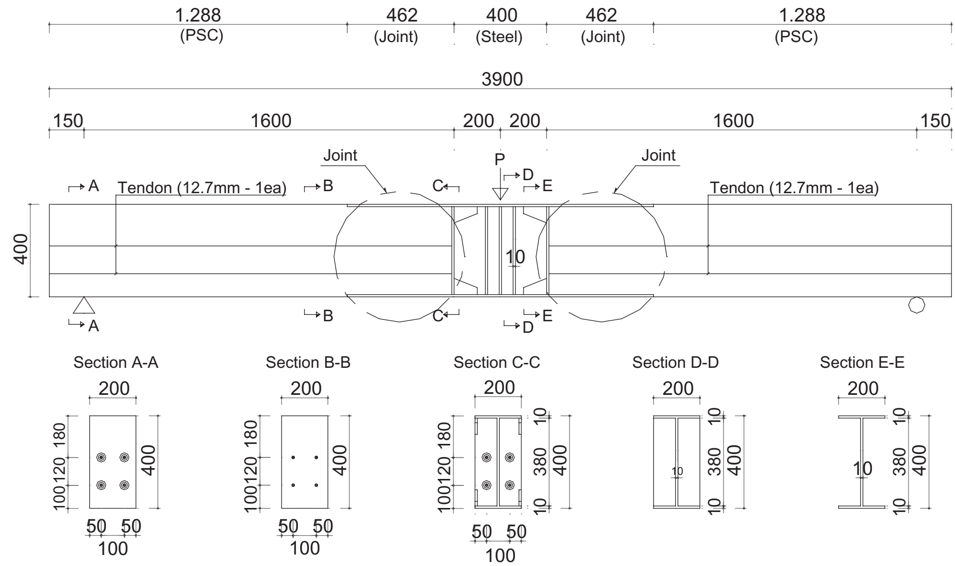


Figure 1. Layout of a beam with 3.9 m length (unit: mm).

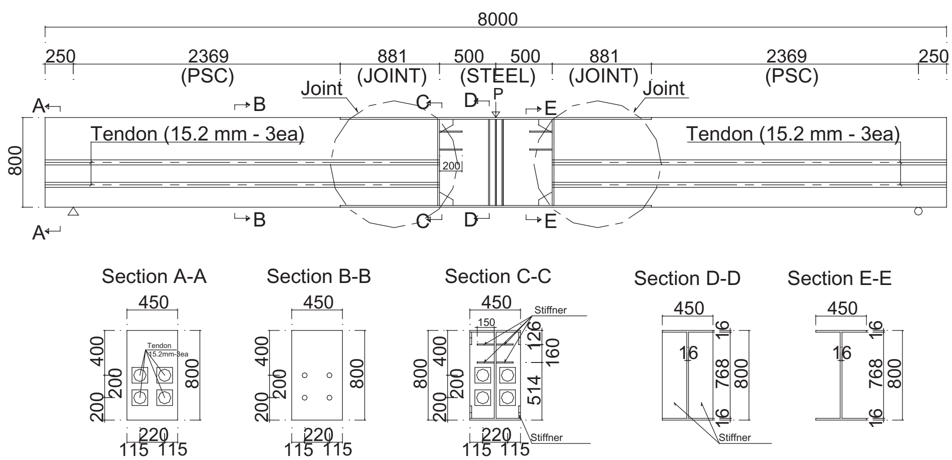


Figure 2. Layout of a beam with 8 m length (unit: mm).

steel part. PSC parts are located at both sides of beams and steel part is located at the middle of beams. Joint parts are located between PSC part and steel part.

Four tendons are installed for all beams. Each tendon has 1 strand (SWPC7B 12.7 mm–1ea) in the beams with 3.9 m length while, in case of the beams with 8 m length, each tendon has 3 strands (SWPC7B 12.7 mm – 3ea). The design compressive strength of a concrete is 50 MPa and the used steel is SS400, which has a nominal yield strength of 240 MPa.

2.2 *Joint details*

The tested joints with perfobond ribs are shown in Figures 3 and 4. The joints have a rear plate, an upper plate, and a lower plate. In case of beams with 3.9 m length, parallel perfobond ribs are

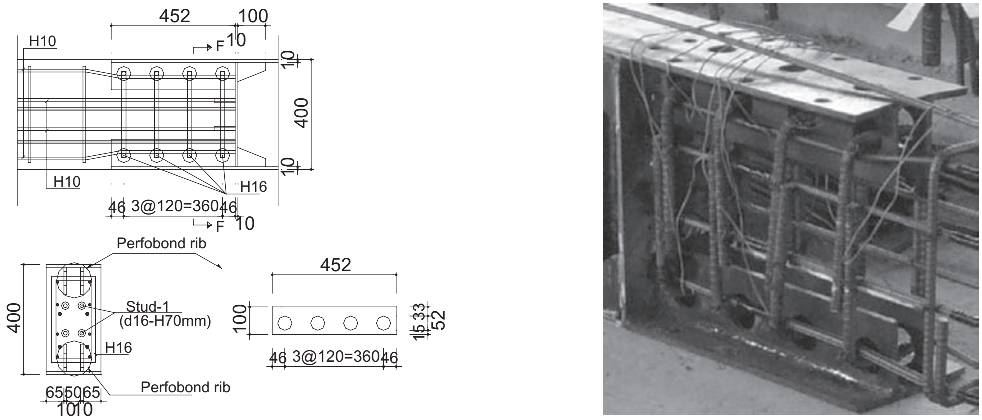


Figure 3. Configuration of joints in hybrid beams with 3.9 m length.

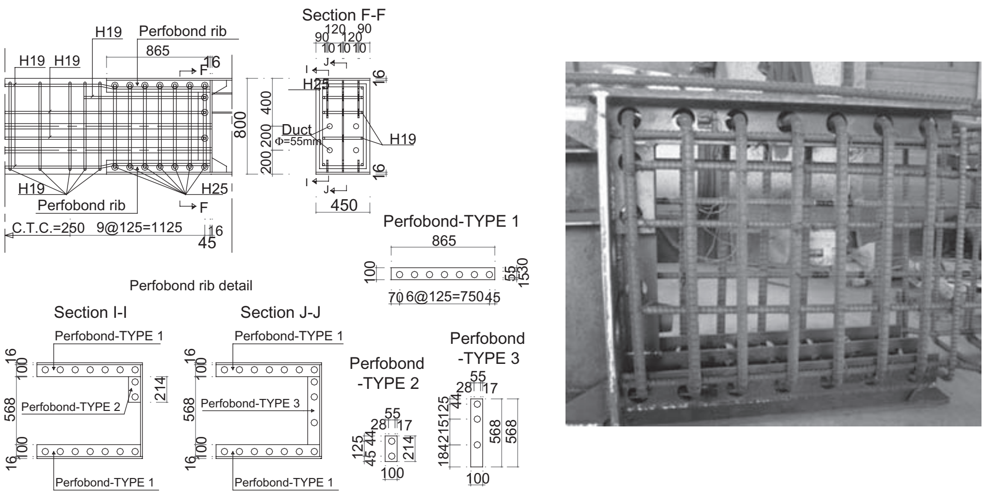


Figure 4. Configuration of joints in hybrid beams with 8 m length.

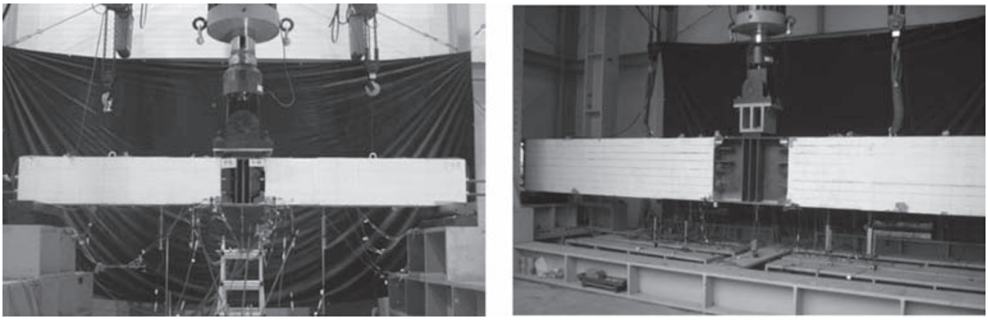
attached to the upper and lower plate and shear studs are attached to rear plate. In case of beams with 8 m length, parallel perfobond ribs are attached to all plates.

The shear capacity of perfobond ribs in the joint parts is determined by a following equation (Oguejiofor & Hosain 1997).

$$Q_{u,pr} = 4.50htf_c{}' + 0.91A_{tr}f_y + 3.31nd^2\sqrt{f_c{}'} \tag{1}$$

where $Q_{u,pr}$ = shear capacity per perfobond rib (N); h = height of perfobond rib (mm); t = thickness of perfobond rib (mm); $f_c{}'$ = compressive strength of concrete (MPa); A_{ty} = total area of transverse reinforcement (mm^2); f_y = yield strength of reinforcement (MPa); n = number of holes; and d = diameter of holes (mm).

It is assumed that the allowable stress of perfobond ribs is about 1/4 level of its shear capacity (ultimate strength) in order to design perfobond ribs using the allowable stress design method.



(a) 3.9 m beam (b) 8 m beam

Figure 5. Photograph of the test setup.

The designed joint has parallel and multiple perfobond ribs while the above equation considers the single perfobond rib. Thus, the shear capacity of the applied equation is reduced by 20% to reflect the group effects.

2.3 *Load test*

The experimental tests are carried out using an actuator with a capacity of 2000 kN. The test condition is simply supported beam under three-point loads as shown in Figure 5. At the beginning of the test, each beam is loaded using force-control method until the crack is visible. After the visible crack is occurred, the test is switched to the displacement-control method. For two beams with 3.9 m length, displacements are gradually increased without unloading (3.9 m-ST-1 and 3.9 m- ST-2). For one beam, the loading and unloading is repeated three times by displacement-control (3.9 m-RE). For the two beams with 8.0 m length, one is tested under gradually increased displacement-control (8.0 m-ST) and the other repeated displacement-control (8.0 m-RE).

3 TEST RESULTS

3.1 *Failure mode*

The failure modes of the tested beams are investigated (Figures 6–7). The initial cracks are developed at boundary between joint part and PSC part for all beams as shown in figures. As applied loads increase, the propagation of flexural cracks is shown. The flexural capacity of PSC part dominates the failure of all beams. The joints have sufficient strength and stiffness, resulting in inducing the failure of PSC part without the failure of joints. Thus, it is thought that

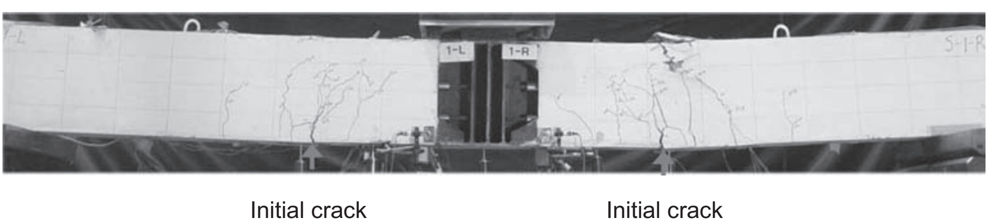


Figure 6. Failure mode of 3.9 m beam.

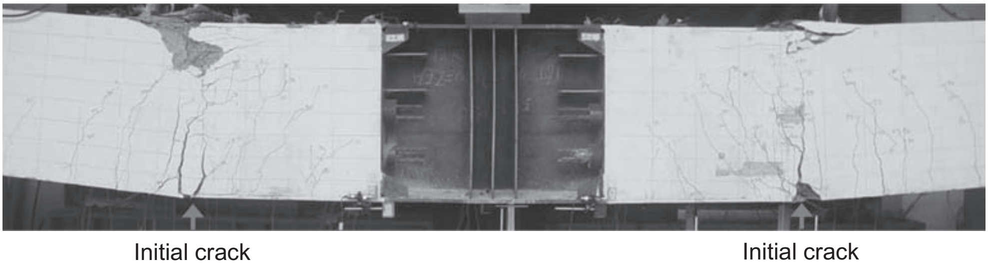


Figure 7. Failure mode of 8 m beam.

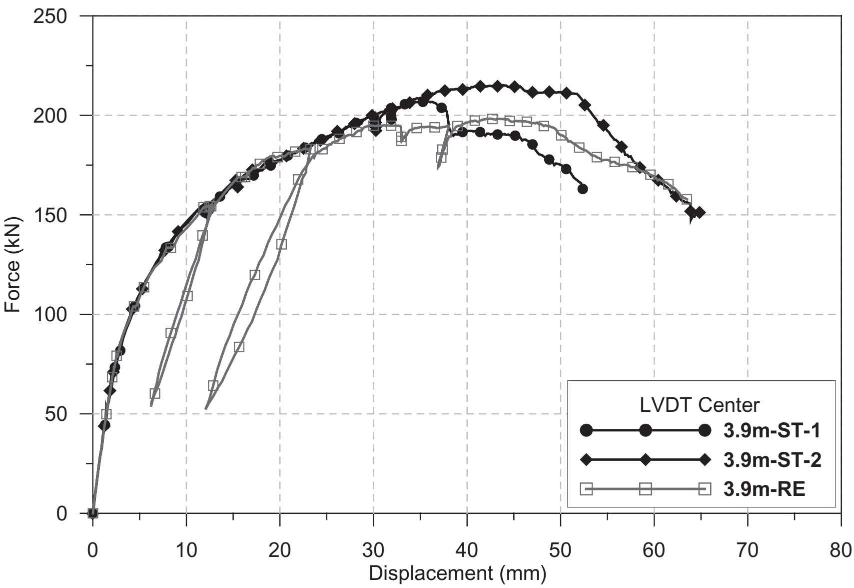


Figure 8. Load-displacement curve at the mid-span of 3.9 m beams.

the applied joints between steel part and PSC part are valid to transmit the each member force to the other part.

3.2 *Load-deflection relationship*

The load-deflection curves of tested beams are measured at the mid-span. Figure 8 shows the results of 3.9 m beams. Initial crack starts at a load of about 70–75 kN. After the initial crack is developed, the nonlinear behavior is shown. Load-deflection curves of three beams are similar within a load of approximately 190 kN. Thereafter, the curves are shows the different configuration. The average value of ultimate strengths is about 207 kN, which is similar values to the ultimate strength of a PSC section at the end part of PSC near joints.

The load-deflection curves of 8 m beams are represented in Figure 9. Before the initial crack is occurred at a load of 300–350 kN, two beams show the linear behavior. Two curves are same to a load of 850 kN. When the applied loads exceed 850 kN, the curves starts separation. The average value of ultimate strengths in 8 m beams is about 940 kN.

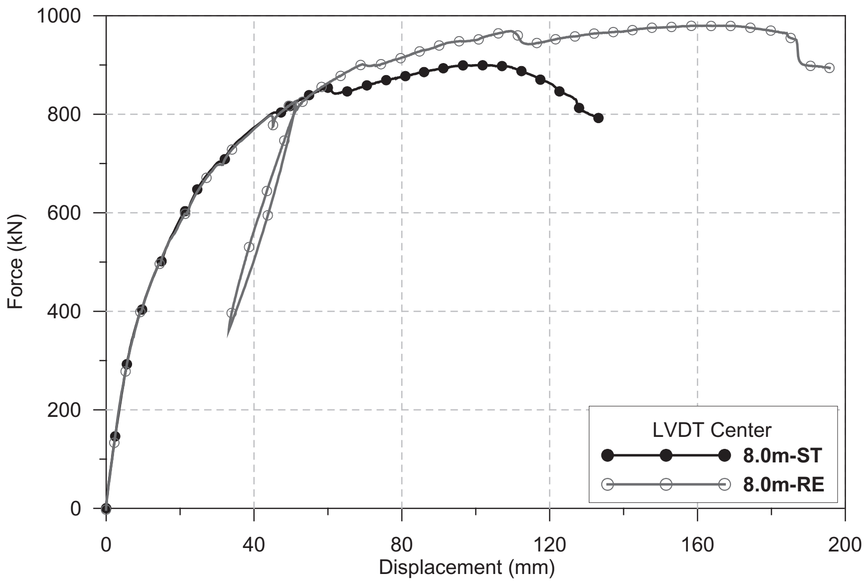


Figure 9. Load-displacement curve at the mid-span of 8 m beams.

4 CONCLUSIONS

In this study, the hybrid PSC-steel-PSC beams with perfobond rib connectors are proposed and tested to verify their structural effects. Experimental results show that the flexural capacity of PSC part dominates the failure mode and the ultimate strength of all beams. The proposed joints have a sufficient strength and stiffness, and thus the failure of PSC is induced without failure of joints. It is thought that the proposed joint is an effective system as a joint of the steel-PSC hybrid structural system.

ACKNOWLEDGEMENT

This study has been supported by GS Engineering & Construction and Sampyo Engineering & Construction. This study also has been supported in part by Yonsei University, Center for Future Infrastructure System, a Brain Korea 21 program, Korea.

REFERENCES

Dunai, L. Fukumoto, Y. & Ohtani, Y. 1996. Behavior of steel-to-concrete connections under combined axial force and cyclic bending. *Journal of Constructional Steel Research*: 36(2): 121–147.

Hino, S. Hamada, S. Kaneyuki, K. Hasegawa, H. & Nakano, H. 1985. Flexural Behavior of Mixed Steel-Concrete Beams with Various Joints. *Memories of the Faculty of Engineering, Yamaguchi University* 35(2): 291–299.

Oguejiofor, E.C. & Hosain, M.U. 1997. Numerical Analysis of Push-Out Specimens with Perfobond Rib Connectors. *Computers & Structures* 62(4): 617–624.

Chapter 17

Rehabilitation of bridges using Ultra-High Performance Fiber Reinforced Concrete

E. Brühwiler
Ecole Polytechnique Fédérale de Lausanne (EPFL), Lausanne, Switzerland

ABSTRACT: An original concept is presented for the rehabilitation of concrete structures. The main idea is to use Ultra-High Performance Fiber Reinforced Concrete (UHPFRC) to "harden" those zones of the structure that are exposed to severe environment and high mechanical loading. All other parts of the structure remain in conventional structural concrete as these parts are subjected to relatively moderate exposure. This conceptual idea combines efficiently protection and resistance properties of UHPFRC and significantly improves the structural performance of the rehabilitated concrete structure in terms of durability and life-cycle costs. The concept is validated by means of four applications demonstrating that the technology of UHPFRC is mature for cast in-situ and prefabrication using standard equipment for concrete manufacturing.

1 INTRODUCTION

Reinforced concrete structures show excellent performance in terms of structural behavior and durability except for those zones that are exposed to severe environmental and mechanical loading. Rehabilitation of deteriorated concrete structures is a heavy burden also from the socio-economic viewpoint since it also leads to significant user costs. As a consequence, novel concepts for the rehabilitation of concrete structures must be developed. Sustainable concrete structures of the future will be those requiring just minimum interventions of only preventative maintenance with no or only little service disruptions.

Over the last 10 years, considerable efforts to improve the behavior of cementitious materials by incorporating fibers have led to the emergence of Ultra-High Performance Fiber Reinforced Concretes (UHPFRC). These novel building materials provide the structural engineer with an unique combination of (1) extremely low permeability which largely prevents the ingress of detrimental substances such as water and chlorides and (2) very high strength, i.e., compressive strength higher than 150 MPa, tensile strength higher than 10 MPa and with considerable tensile strain hardening (up to more than 2‰ of strain) and softening behavior (with fracture energy of more than 15'000 J/m^2). In addition, UHPFRC have excellent rheological properties in the fresh state allowing for easy casting of the self-compacting fresh material with conventional concreting equipment. Consequently, UHPFRC have an improved resistance against severe environmental and mechanical loading thus providing significantly improved structural resistance and durability to concrete structures.

This paper presents an original concept for the rehabilitation of concrete structures. The concept is described and validated by means of four applications.

2 CONCEPTUAL IDEA

The basic conceptual idea is to use UHPFRC only in those zones of the structure where the outstanding UHPFRC properties in terms of durability and strength are fully exploited; i.e. UHPFRC is used to "harden" the zones where the structure is exposed to severe environmental conditions (f.ex., deicing salts, marine environment) and high mechanical loading (f.ex. impact, concentrated

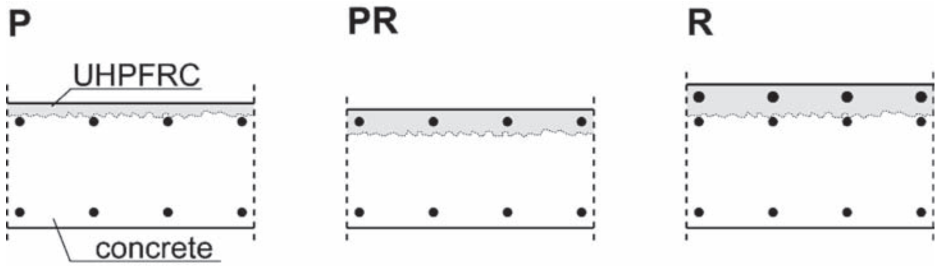


Figure 1. Basic configurations for composite structural elements combining UHPFRC and conventional structural concrete.

loads, fatigue). All other parts of the structure remain in conventional structural concrete as these parts are subjected to relatively moderate exposure. This concept (which is also applicable to new construction) necessarily leads to composite structural elements combining conventional reinforced concrete and UHPFRC.

The combination of the UHPFRC protective and load carrying properties with the mechanical performance of reinforcement bars provides a simple and efficient way of increasing the stiffness and load-carrying capacity keeping compact cross sections (Figure 1). Depending on the structural and material properties of the composite system, more or less pronounced built-in tensile stresses are induced in the UHPFRC due to restrained deformations at early age. This stress state needs to be analyzed and evaluated.

UHPFRC is applied on existing reinforced concrete bridges as thin watertight overlays (in replacement of currently used waterproofing membranes), as reinforcement layers combined with reinforcement bars, or as prefabricated elements such as curb elements. The relatively high material cost of these materials imposes to use them only where they are "worth their money" and it is possible to take the maximum benefit of their outstanding mechanical properties.

The original conceptual idea (developed in 1999) has been investigated by means of extensive researches (see scientific papers under (mcs.epfl.ch 2009)) aimed at characterizing UHPFRC materials and the structural behavior of composite structural members. The concept is well-suited for bridges and can also be implemented for buildings, galleries, tunnels or retaining walls. Validation by means of four applications will be described in the following after describing UHPFRC properties.

3 PROPERTIES OF UHPFRC

The uniaxial tensile behavior of two different recipes of the UHPFRC CEMTEC$_{\text{multiscale}}$® type (Rossi 2002) has been determined by means of a tensile test on unnotched dogbone specimens (Denarié & Brühwiler 2006). The average curves from five tests for each material are represented on Figure 2, showing the range of possible strain hardening responses. Both recipes are self-compacting.

Recipe CM0 is reinforced with a 468 kg/m^3 of 10 mm long steel fibers with an aspect ratio of 50. It has a water/binder ratio of 0.140, 1051 kg/m^3 cement, a fluid consistency.

Recipe CM23 has more binder (1437 kg/m^3 cement) and a lower water-binder ratio (0.125). It is reinforced by a multilevel fibrous mix of macro steel fibers (10 mm long, aspect ratio 50) and microfibers (steel wool) with a total dosage of 705 kg/m^3. It can hold a slope of the substrate up to 2.5%.

The effect of the addition of microfibers is revealed on Figure 2 by three aspects, namely, (1) significant increase of the pseudo-elastic domain from 8 to above 11 MPa, (2) increase of the strain hardening domain, and (3) the increase of the load carrying capacity in the descending branch due to the indirect action of the microfibers on the progressive pull-out of the macrofibers.

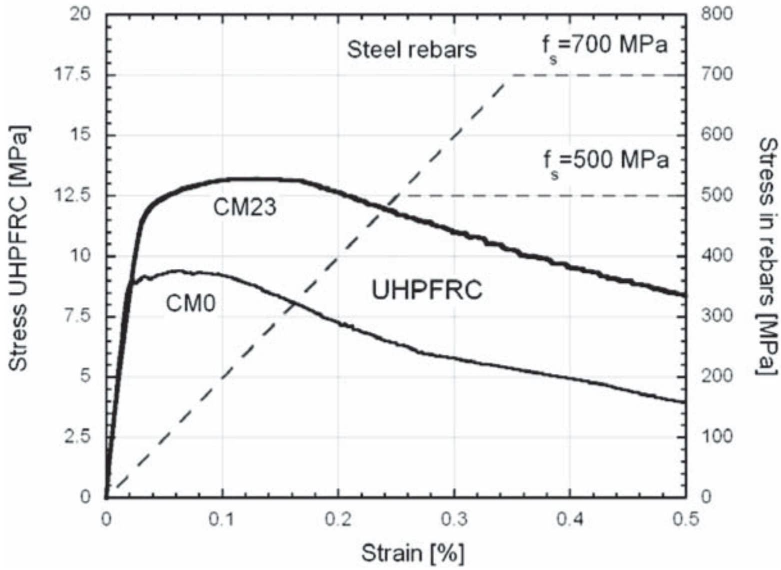


Figure 2. Tensile behavior of two UHPFRC recipes (unnotched specimens, fixed rigid boundary conditions, average curves at 28 days).

It is worth mentioning that the magnitude of strain hardening of UHPFRC such as CEMTE$_{multiscale}$® falls into the range of the yield strain of construction steel, Figure 2. This property opens up the combination of UHPFRC with reinforcement bars with high yield strength (700 MPa or above).

The fractured surface of a UHPFRC specimen after a tensile test shows numerous steel fibers, pulled out from the matrix. The work of pull-out of these numerous micro-reinforcements explains the extremely high specific work of fracture of UHPFRC (up to 30’000 J/m^2 compared to 200 J/m^2 for normal concrete). A significant part of the work of fracture of UHPFRC is dissipated in the bulk of the material, during the strain hardening phase, in the form of finely distributed, multiple cracks.

One should keep in mind that the mechanical response of fibrous composites such as UHPFRC is very much application dependent. Strong anisotropy effects can be induced by the casting procedure of the materials or the width and shape of the moulds and these effects have to be considered for the analysis of test results and for design; adding reinforcing bars to the UHPFRC reduces these anisotropy effects (mcs.epfl.ch 2009).

4 REHABILITATION AND WIDENING OF A ROAD BRIDGE

A short span road bridge with busy traffic has been rehabilitated and widened using UHPFRC (Denarié & Brühwiler 2006, SAMARIS 2005). The entire deck surface of the bridge with a span of 10 m was rehabilitated in three steps during autumn 2004 (Figure 3). Firstly, the downstream curb was replaced by a new prefabricated UHPFRC curb on a new reinforced concrete beam which was necessary for the widening. Secondly, the chloride contaminated concrete of the upper surface of the bridge deck was replaced by 3 cm of UHPFRC in two consecutive steps such that one traffic lane could be maintained open. Thirdly, the concrete surface of the upstream curb was replaced with 3 cm of UHPFRC.

The UHPFRC mix contained 1430 kg/m^3 Cement, Microsilica, fine quartz sand with a maximum grain size of 0.5 mm; the Microsilica/Cement and Water/Binder ratio were 0.26 and 0.125 respectively. The reinforcement of this ultra compact matrix was provided by a mix of microfibers

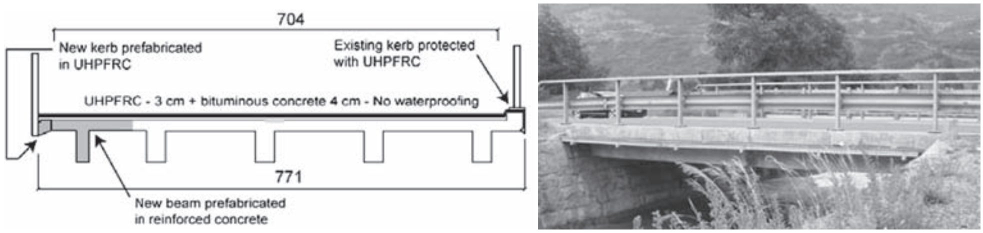


Figure 3. Bridge cross section after rehabilitation (dimensions in cm) and photo taken in 2006.

Figure 4. UHPFRC casting and handling of UHPFRC using simple tools.

(steel wool of 2 to 3 mm length) and macrofibers of 10 mm length and an aspect ratio of 50, with a total dosage of 706 kg/m^3 (or 9 vol.%).

The fresh self-compacting UHPFRC material was prepared at a local concrete prefabrication plant with a standard mixer, brought to the site by a truck and then poured on the hydrojetted deck surface (Figure 4). The UHPFRC was easy to produce and place with standard tools and very robust and tolerant to the unavoidable particular site conditions. The bituminous pavement was applied on a bituminous emulsion placed on the UHPFRC surfaces after 8 days of moist curing, and the corresponding lane was reopened to traffic the next day. The bridge was fully reopened to traffic one month after the beginning of the construction work.

The protective function of the UHPFRC layer was verified by air permeability tests according to the Torrent method (Torrent 2007). These tests confirmed the extremely low permeability of the UHPFRC layer; i.e. about 30 times lower than for excellent conventional concretes. The average compressive strength and modulus of elasticity at 28 days were respectively 182 MPa and 47 GPa. Uniaxial tensile tests performed at 28 days in the laboratory on unnotched dogbone specimens cast on site showed the expected remarkable average properties: tensile strength of 13.3 MPa and maximum tensile deformation in the strain-hardening domain of 1.5‰.

The analysis of the construction costs showed that the rehabilitation realized with UHPFRC was about 10% more expensive than the conventional solution (providing lower quality in terms of durability and life-cycle costs) with waterproofing membrane and repair mortar. However, in the latter case the duration of the construction site would have been largely increased by the required drying period of the mortar, prior to the application of the waterproofing membrane. It can be expected that with a wider use, application of UHPFRC for the rehabilitation of bridges, this technique will become less costly than traditional ones, not to mention its outstanding advantages of long term durability and reduction of traffic disruptions (and subsequent user costs) due to multiple interventions.

5 UHPFRC ON A CRASH BARRIER WALL

Specific parts of reinforced concrete structures such as crash barrier walls on highway bridges suffer from severe exposure to concrete aggressive substances such as de-icing salts and impact like action. Such elements often show insufficient durability when built using conventional reinforced concrete. Moreover, as UHPFRC have very low permeability as well as high strength and deformability, it is suitable to significantly improve the durability and mechanical performance of such structural elements.

A layer of UHPFRC has been applied in September 2006 to the concrete crash barrier walls of a highway bridge covering the areas subjected to splash exposure (Class XD3: reinforcement corrosion induced by chlorides) (Figure 5). The main design requirement was to obtain long-term durable crash barrier walls since traffic interruption for future rehabilitation interventions are prohibitive due to the very high traffic volume on this highway. Long-term durability is obtained when transverse macro-cracks in the UHPFRC layer are absent and the permeability of UHPFRC layer to ingress of water and chloride ions is extremely low.

The UHPFRC recipe contained 1100 kg/m³ cement, 26% silica fume related to the cement content, quartz-sand, 6% steel fibers by volume, superplasticiser and a w/c-ratio of 0.17. The rheological properties of UHPFRC were adapted for easy pouring into the 3 cm wide formwork to fill a height of 120 cm including a small horizontal part at the bottom of the wall that provides continuity with the conventional bridge deck with a waterproofing membrane.

Due to restrained early age deformation of the UHPFRC (mostly due to thermal and autogenous shrinkage) bonded to the existing reinforced concrete wall, an internal stress state is built up in the composite element including, in particular, tensile stresses in the UHPFRC layer. These tensile stresses, which can cause macrocrack formation, and the capacity of the UHPFRC to resist to these stresses were investigated by means of numerical analyses prior to the intervention (Oesterlee et al. 2007).

The fresh self-compacting UHPFRC was fabricated in a conventional ready mix concrete plant, transported to the site by a truck and successfully filled into the thin slot to realize the UHPFRC coating. The required mechanical properties and the protective function of the UHPFRC layer have been confirmed by in-situ air permeability tests and laboratory tests on specimens cast on site.

The aesthetic aspect was very appealing showing a smooth surface with very few voids. Four months after application no crack could be found confirming the predictions made by the numerical simulations.

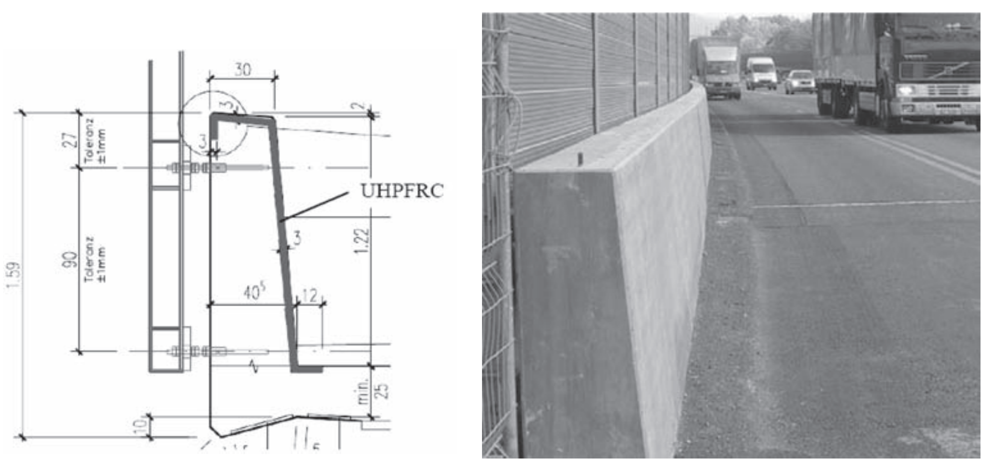


Figure 5. Typical cross section of the crash barrier wall and view after rehabilitation.

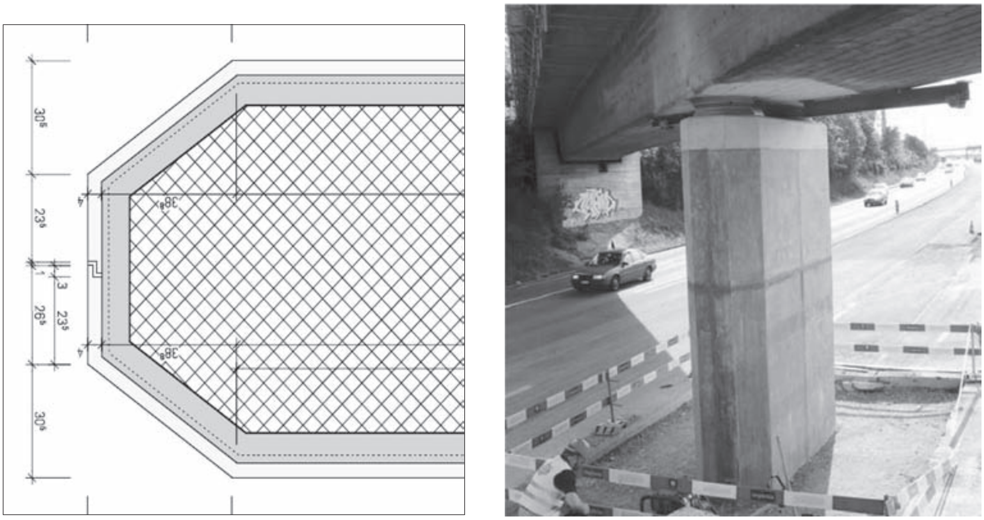


Figure 6. Cross section and general view of the rehabilitated bridge pier.

6 REHABILITATION OF A BRIDGE PIER USING PREFABRICATED UHPFRC ELEMENTS

Bridge piers and retaining walls in the splash zone of highway traffic suffer from severe exposure to de-icing salts and impact like action. Such elements usually show premature deterioration when built in conventional reinforced concrete. In order to significantly improve durability and mechanical strength of such elements, UHPFRC is used following again the concept of locally "harden" the zones of severe exposure.

In this application, 4 cm thick UHPFRC shell elements have been prefabricated to form an outer protection shield for the existing 40 year-old reinforced concrete bridge pier which is located in the middle of a busy highway which makes it virtually not accessible for future maintenance interventions (Figure 6).

In spring 2007, the UHPFRC elements (maximum element height of 4 m) were cast in a prefabrication plant, transported to the construction site and mounted, after removing of up to 10 cm of chloride contaminated concrete by hydrojetting. The joints between the different UHPFRC shell elements were glued using an epoxy resin. The remaining space between the UHPFRC elements and the existing reinforced concrete was filled with self-compacting mortar.

The used UHPFRC recipe contained about 1300 kg/m^3 of cement, a rather small amount of silica fume related to the cement content, quartz-sand, 3.5% of steel fibers by volume, superplasticiser and a W/C-ratio of 0.155. Long-term durability is expected since transverse cracks in the UHPFRC protection shield are absent and the permeability of UHPFRC for ingress of water and chloride ions is extremely low as confirmed by permeability tests.

7 REHABILITATION AND STRENGTHENING OF A BRIDGE DECK SLAB

The deck slab of a more than 70 year old reinforced concrete bridge of high cultural value had to be rehabilitated and strengthened to accommodate for future traffic demands of a village in a mountainous area. The intervention consisted in casting a layer of reinforced UHPFRC, i.e. ordinary reinforcing steel bars as main reinforcement were arranged before casting the UHPFRC, on top of the deck slab with the objective to improve the durability and to restore the structural safety (Figure 7). The fresh UHPFRC with a composition similar to the previous application has been

Figure 7. Rehabilitation and strengthening of a bridge deck slab with reinforced UHPFRC.

fabricated in a nearby ready mix concrete plant and transported to the construction site. Gravel was dispersed on the fresh UHPFRC such as to obtain the required roughness of the surface fit for road traffic.

8 PROJECT FOR NEW OVERPASS BRIDGE IN UHPFRC—PC CONSTRUCTION

The above described conceptual idea for rehabilitation obviously may also be applied for the design and construction of new bridges. In the case of the project for a new road bridge overpass (Figure 8) (Brühwiler et al. 2007), the severely exposed zones include the top surface of the deck slab, the curb and sidewalk overlay elements and the zone above the middle support including a unique UHPFRC hinge. All other parts of the bridge structure remain in conventional reinforced and prestressed concrete as these parts are subjected to only moderate exposure. The developed bridge concept consists of a superstructure as a continuous two-span multi-girder system resting on a middle support and the abutments.

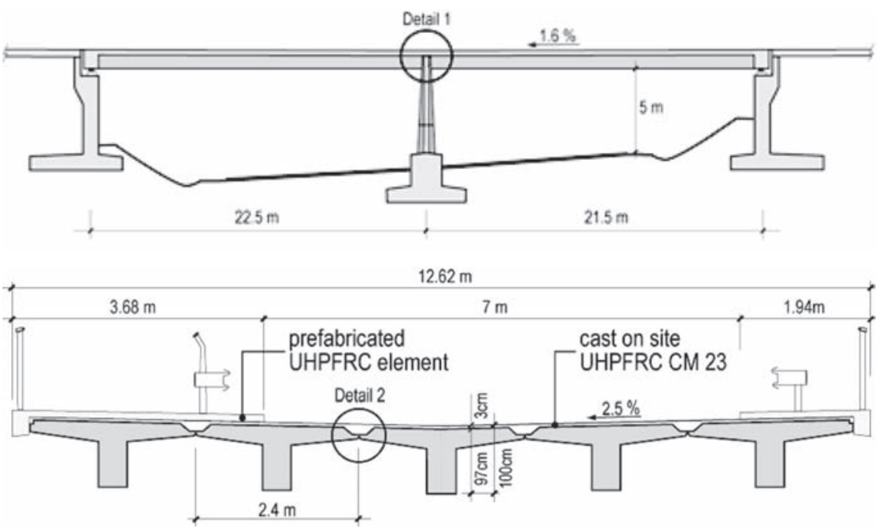


Figure 8. Project for a new overpass road bridge: elevation and typical cross section.

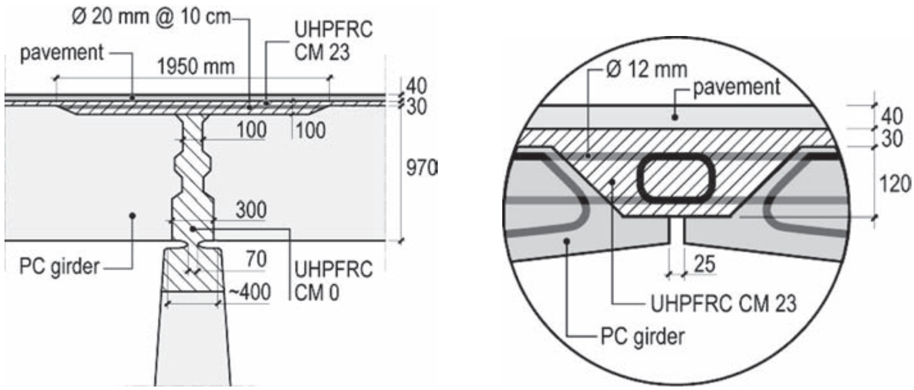


Figure 9. (a) Detail 1: middle support, and (b) Detail 2: longitudinal joint between prefabricated girders.

The main girders are prefabricated in a construction plant and are prestressed and posttensioned in the longitudinal direction. After their alignment on the construction site, the space between the girders over the middle pier is filled out with UHPFRC creating at the same time an UHPFRC hinge over the finger piers (Detail 1, Figure 9a). Then, a layer of UHPFRC is cast to connect the prefabricated girders along the longitudinal joint (Detail 2, Figure 9b) and to provide a 3 cm thick (waterproofing) protection layer on the whole top surface of the deck slab. The curb elements are made of prefabricated UHPFRC elements that are glued to the top surface. UHPFRC is thus applied both on the construction site and in the prefabrication plant by using different adapted mixes.

The duration of the construction works shall be limited to a minimum in order to reduce disturbance for the road user. The total time of construction of the superstructure may be optimized and reduced to about 25 to 30 days. In addition, critical and time consuming steps of the construction process such as application of waterproofing membranes, on-site concreting or compaction by vibration are eliminated, and the associated sources of errors avoided. The construction process becomes then simpler, quicker and more robust with an optimal use of prefabrication. The estimated construction cost is similar than for a conventional concrete bridge.

The originality of Detail 1 (Figure 9a) resides in the casting of the top part of the pier, hinge and connection between the prefabricated girders in one onsite casting sequence using UHPFRC. The 70 mm wide UHPFRC hinge is subjected to significant rotations requiring certain deformation capabilities of the UHPFRC. The top part of the middle support is a tension chord consisting of UHPFRC reinforced with steel reinforcing bars to take the tensile forces due to the bending moment over the support.

Detail 2 (Figure 9b) provides a stiff load carrying connection between the prefabricated girders in the longitudinal direction. The 150 mm thick UHPFRC joint with steel reinforcement bars is highly resistant to account for concentrated wheel loads on the deck slab.

The 30 mm thick UHPFRC layer provides the required mechanical performance and extremely low permeability as shown by structural analysis (Oesterlee et al. 2008). This UHPFRC protection layer contributes favorably to the load carrying behavior of the deck slab in terms of stress membrane resisting against compression and tension forces without undergoing crack formation.

9 CONCLUSIONS

An original concept using Ultra-High Performance Fiber Reinforced Concrete (UHPFRC) for the rehabilitation of concrete structures has been presented and validated by means of four applications.

This conceptual idea combines efficiently the protection and resistance properties of UHPFRC with conventional structural concrete. The rehabilitated structures have significantly improved structural resistance and durability.

The full scale realizations of the concept under realistic site conditions demonstrate the potential of these applications, proving that the technology of UHPFRC is mature for cast in-situ and prefabrication using standard equipment for concrete manufacturing.

The original concept is also applicable for the construction of new bridges with improved durability and structural resistance.

ACKNOWLEDGMENTS

The support of the Swiss National Science Fund, The Swiss Secretary of Education and Research, The Committee of Technology and Innovation, Cemsuisse, Holcim, Swiss Steel AG and the Swiss Federal Roads Office is gratefully acknowledged. The applications were possible thanks to the Road Administrations of the Swiss Cantons of Wallis and Aargau.

The UHPFRC Group at MCS-EPFL consists of the following current and former members: Dr Emmanuel Denarié, Dr Katrin Habel, Prof. Jean-Philippe Charron, Dr Hamid Sadouki, Prof. Minoru Kunieda, Dr Aicha Kamen, Dr John Wuest, Dr Andrin Herwig, Cornelius Oesterlee, Agnieska Switek and Talayeh Noshiravani.

REFERENCES

Brühwiler, E., Fehling, E., Bunje, K. & Pelke, E. 2007. Design of an innovative composite road bridge combining reinforced concrete with Ultra-High Performance Fibre Reinforced Concrete. Proceedings, IABSE Symposium "Improving Infrastructure Worldwide", Weimar, September 2007.

Denarié, E. & Brühwiler, E. 2006. Structural rehabilitations with Ultra High Performance Fibre Reinforced Concretes, *International Journal for Restoration of Buildings and Monuments*, Aedificatio, Vol. 12, No. 5 and 6, pp. 453–467.

mcs.epfl.ch 2009. http://mcs.epfl.ch/ see under reviewed journal papers.

Oesterlee, C., Denarié, E. & Brühwiler, E. 2007. In-situ casting of UHPFRC protection layer on crash barrier walls. Proceedings, Advances in Construction Materials—Symposium in honour of Hans W. Reinhardt, July 2007, University of Stuttgart, Germany.

Oesterlee, C., Sadouki, H. & Brühwiler, E. 2008. Structural analysis of a composite bridge girder combining UHPFRC and reinforced concrete, Proceedings, UHPC-2008: The Second International Symposium on Ultra High Performance Concrete, March 2008, Kassel, Germany.

Rossi, P. 2002. *Development of new cement composite material for construction*, K. Dhir, P.C. Hewlett, L.J. Csetenyi Editors, Proceedings of the International Conference on Innovations and Developments In Concrete Materials And Construction: 17–29, University of Dundee, Dundee, Scotland.

SAMARIS 2005, Report D22, *Full scale application of UHPFRC for the rehabilitation of bridges—from the lab to the field,* European project 5th FWP/SAMARIS—Sustainable and Advanced MAterials for Road Infrastructures—WP 14: HPFRCC, http://samaris.zag.si/

Torrent, R., Fernandez Luco, L. Non-destructive Evaluation of the Penetrability and Thickness of the Concrete Cover, State of the Art Report, RILEM TC 189-NEC, 2007.

Chapter 18

Lightweight and recycled materials for the construction of the Messina Strait Bridge, Italy

G. Di Giannantonio
Laboratory International, Inc., Chagrin Falls, OH, USA

ABSTRACT: When built, the Messina Straight Bridge in Italy will be the largest suspension Bridge in the world, with a center span length of two miles. The local traffic congestion, lack of parking, and the enigmatic environmental problem solutions are examined and taken into consideration as reference for the bridge construction. The input of innovative formulation of "Microballoons/Marmettola" for the end products specifically applied for the Mega-Bridge will allow to address many interesting Lightweight structural materials. In fact, these Lightweight material applications and methodologies from paving Asphalt Rubber Technological System (ARTS), combined and connected to embedded sensors, will permit to reduce the Bridge load, monitoring its traffic vibration, to prevent failure of its structural components, thus controlling construction and maintenance costs with the most of benefits to the surrounding environment.

Especially, the Light and Ultra-Lightweight end-products embedded sensors for the Mega-Bridge construction should be useful also for other small and large bridges design and construction connected to a total computer system which could provide the database necessary to enhance further project development. It is through the re-use of these specific Lightweight materials such as Microballoons/Marmettola/AMICA/Sensors/ and Ultra-Lightweight metallic components are essential to safeguard the local environment as well as the construction of the Bridge.

Ultra-Lightweight support components for bridge construction consists mainly of two or more skin plates (e.g. aluminum alloy sheeting) with an intermediate core of sheet—aluminum "honeycombs"—This method of construction for bridges called sandwich structures, its characteristics are low maintenance and high strength (see Figure 1). These bridge structures must withstand bending, shear, torsion and internal/external pressure. These ultra-Lightweight support components (aluminum alloys), for instance, can have strengths as high as 70,000 lbs/sq inch while weighing only 0.1 lb/cubic inch (2.7 grams per cubic centimeter). These alloys have the strength of steel plate at ⅓ the density.

1 INTRODUCTION

In the 17th and 18th centuries, bridge building became a science. Early in this period, scientists including Galileo Galilei, had investigated the theory of beams and framed structures and before the end of the period bridge builders were required to work to detailed specifications. A structural form based on the triangle, the truss had long been used to support roofs when it was adapted to bridge design by the Italian Renaissance architect Andrea Palladio. In the 18th century, Swiss carpenters used it to build a covered timber bridge having spans of 193 feet (59 m) and 171 feet (52 m) over the Rhine at Schatthausen, Switzerland. This feat was followed by a similar timber bridge with the 240 foot span at Reichenau, Switzerland. The barn-like covering of timber–truss bridges was necessary to protect the structural members against the weather. In North America, many outstanding timber–truss bridges were built, the first probably that over the Connecticut River at Bellows Falls, Vermont in 1785. The so called Colossus Bridge, with a 340 foot span over the Schuylkill River at Fairmount Park, Philadelphia, was constructed in 1812. One of the best long span trust designs was developed by Theodore Burr, of Torrington, Connecticut and based on a drawing by Palladio; a truss strengthened by an arch, which set a new pattern for covered

Figure 1. Caption of sample Lightweight material blocks and an illustration of "Sandwich Construction" for ultra lightweight support application.

bridges in the United States. Burr's McCall's Ferry Bridge (1815 on the Susquehanna River, Lancaster, Pennsylvania) had a record-breaking span of 360 feet (110 m) and hundreds of his bridges are still in use. The timber bridge covered by the filler dirt for the highway I-71, Brookside Park, Cleveland, Ohio, was not relocated but buried under that heavy embankment load and its traffic (a working experience at Ohio Department of Highway, Project 221, year 1964–1965).

2 SUSPENSION BRIDGES

In view of the fate of most of the early suspension bridges in both Europe and the United States, it is credited to Telford that the Menai Bridge survived for 115 years. Another chain suspension bridge that had long life was a span over the Danube River at Budapest, Hungary. Completed in 1849, it survived for nearly 100 years, until it was destroyed in World War II. Before and after the first and second world war, multiple span suspension bridges were built almost all over the globe. The weakness of the early suspension bridges and storms or under repeated rhythmic loads were fatal for most of them. In 1831 the Broughton Suspension Bridge collapsed because of oscillations set up by a body of troops marching in step. Four other bridges in the United States and Britain were destroyed simply by the impact of flocks of sheep or droves of cattle and others were blown down for some reason or another but in the United States the Fairmont Bridge, supported by a number of small wire cables, over the Schuylkill River, was a success. Instead, the Ohio's span bridge over the Wheeling River, West Virginia, survived only five years. The credit for designing and building the first suspension bridge that was rigid enough to withstand not only wind action but also the impact of railway traffic belongs to John A. Roebling, an immigrant from Germany to the United States. Roebling used parallel wrought iron wires spun in place, bunched together and wrapped, a process he had patented in 1841; each cable was 10 inches (2.5 cm) over in diameter. The bridge was completed in 1855 and survived for 42 years, although not without considerable repair work and reconstruction caused by the wear and tear of traffic. The famous Brooklyn Bridge (1869–83), with a record-breaking span of 1595 feet (486 m), was designed by John Roebling and erected under the direction of his son, Washington. It has four cables with an overall diameter of 15.75 inches, built up of parallel steel wires. The method of cable spinning devised by Roebling was so simple and effective that it has been used in principle, although now much elaborated, for the large suspension bridge such as the "Ponte Stretto Di Messina" in Italy.

3 THE STRAIT OF MESSINA BRIDGE

Project design, conferences at all levels, the ups and downs of the political controversies have been and still are enormous, and so the publicity and the cost along with it. The straight of Messina Bridge is a planned Suspension Bridge that would cross the straight of Messina, a narrow section of water between the eastern tip of Sicily and the southern tip of mainland Italy; the collaborator region for centuries, perhaps millennium, since the Roman times, discussions and planning have been taking place. Major interest groups in Italy as well as around the world have followed very closely the bridge development; technical, financial, economical, environmental, political and otherwise. In the last few years of Project Bridge development, it should have started construction beginning in 2006, but under the Romano Prodi government the program was canceled. Currently the new government, under the leadership of Berlusconi, the bridge and other connected infrastructures, it is announced that the plans will be quickly restarted. According to late information the construction should start during the current year 2009 and be completed in the year 2015. When the Suspension Bridge is completed, it will be the largest bridge in the world, larger than the main span of the Akashi-Kaikyo in Japan. The Straight of Messina Bridge plan called for a single span suspension with the central span of 3300 m (about 2 miles). This is 60% longer then the Akashi-Kaikyo bridge in Japan (at present the largest suspension bridge in the world at 1991 m). The plans of the straight of Messina Bridge are designed for six traffic lanes and one emergency lane in each direction, two railway tracks and two pedestrian lanes. In order to provide a minimum vertical clearance for the navigation of 65 m, the heights of the two towers will be 382.6 m. These two towers will be taller than the Millau Viaduct in France (currently the tallest bridge in the world at 341 m). The bridge's suspension system will rely on two pairs of steel cables, each with a diameter of 1.24 m and a total length between the anchor blocks, of 5300 m. The designs show 20.3 km of road links and 19.8 km of railway links to the bridge. On the mainland, the bridge will be connected to the new stretch of the Salerno-Reggio Calabria Motorway (A3) and to the planned Naples–Reggio Calabria high speed railway line; on the Sicily side, to the Messina–Catania (a 18) and Messina–Palermo (a 20) motorways as well as the new Messina railway station (to be built by Rete Ferroviaria Italiana (brochure and pictures are provided by Stretto di Messina). During participation in conferences and scheduled personal meetings in different locations; Trip–memos–to–Italy during the course of years, particularly those undertaken activities related and focused to the Straight of Messina Bridge, the feasibility study "In-situ" of the White Coal Basic Material (WCBM) and its derivatives are analyzed to make the proper determination. Samples from selected locations are examined and tested from bench scale laboratory performance to scale up and products demonstration. Although (WCBM) samples are taken from a variety of different locations and sources, mainly for comparison and selection compatibility, the target of sites determination is within a radius of 200 miles from the Straight of Messina Bridge construction. But, the most important samples raw materials are taken within a radius of 40 miles. The purity of these basic materials and separation of impurities are sampling priority for Lightweight and ultra–Lightweight applications aimed essentially for span suspension bridges. Mixtures of these Lightweight material powders with aggregates and additives are used to make and products to reduce the bridge load.

4 "MICROBALLOONS" AND OTHER LIGHTWEIGHT MATERIALS

The use of these basic material mixtures are recognized to offer important advantages for the one span Suspension Bridge, its lanes paving and the whole building trade, due to the relative ease of handling such end-products, during their application and installation. Given the Lightweight of the product, savings are appreciated in reducing costs of transportation first and construction application subsequently. Lightweight structural materials are used also in the case of fabricating interior walls and partitions for ships and aircraft. For these reasons, many Lightweight materials such as "Microballoons," pumice, perlite, vermiculite, fly-ash, aluminum and others are suggested by the prior art for incorporation into particular construction application according to related patent description. It is used as fillers and/or as aggregates for light concrete, pavement asphalt, blocks, as well as pigments for paint extenders. Activities carried on at Laboratory International bench scale samples experimentation and demonstration and able to produce Lightweight structural end products such as blocks, panels and pavement asphalt coating by using mixture of specific elements which are agglomerated by either high specialty and/or common cement whereby (WCBM) upgrade in the clinker formulation can fluctuate between 60–70%. Laboratory Intl. bench scale activities introduce an entirely new and unique techniques and know how of the mixtures which are highly desirable not only for the Mega-Bridge, but for all kinds of small and large bridges, building trade (see Figure 2).

All kind of small and large bridges building trade modifications of mixture formulation are suggested for improvement according to specific skills of the art. Especially "Microballoons", tiny, hollow spheres, a sodium borosilicate glass process, are making the samples agglomeration much lighter, more flexible, better insulation and less critical to environmental conditions. The colorless particles are about two one thousandths of an inch in diameter and weigh about one third as much as water. The hollow spheres remain unchanged up to temperature of about 1200°F, and give 30% stronger at weight savings of 20% to 50% samples. Offering unique dielectric properties, these Microballoons/WCBM combination formulations are very close to quartz range on a dielectric materials chart and are transparent to high energy wave transmission. It must be also mentioned that applications of these precursors light weight particles are used in radar lenses, microwave antenna windows, randomes and as thermal insulation for electronic assemblies. Small volume, high-priced military, aerospace and electronic markets–tailored products made in a batch type operation, these precursors are not expected to compete against other connected compatible Lightweight materials for construction.

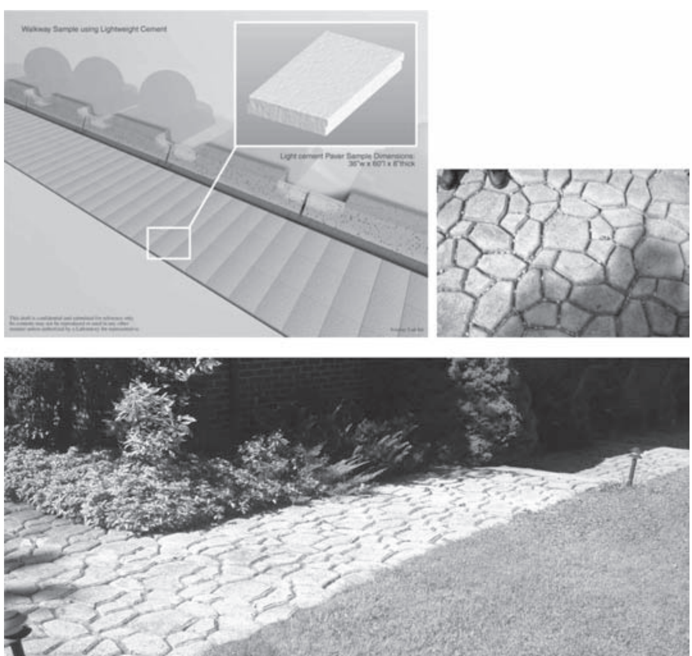


Figure 2. Caption of proposed bridge walkway section illustration and photos of sample walkway made of Lightweight material pavers.

5 UTILIZATION OF COMPATIBLE LIGHTWEIGHT MATERIAL (CLM)

The largest use of (CLM) is located within a radius of 40 miles from the Straight of Messina construction site. The (CLM) and the binder agent such as high specialty cement and selected water (H_2O) from (ALF) samples-source. The mixture is used in precast masonry units and/or toward Lightweight concrete. The (CLM) is used as an aggregate in place of sand and gravel, considerably reducing the weight of the slab concrete which commonly weights from 140 to 150 pounds per cubic foot. When it is made in this nonconventional manner its weight can be as little as 50 pounds per cubic foot. This lightness in weight reduces the dead load on the support metal frame in large bridges and buildings to permit important savings in costs. The lighter concrete slabs are easier to handle, to transport and to install. Strength of (CLM) concrete slabs usually vary inversely with lightness in weight. Blocks that have a low apparent density, about 60–70 pounds per cubic foot have a crushing strength from 600–1000 pounds per square inch. Denser blocks our proportionately stronger.

Other advantages of (CLM) concrete are heat and sound insulation, fire resistance, ease of cutting and shaping, and the ability to hold nails and screws. Thermal insulation efficiency varies inversely with the density. For instance, (CLM) blocks of low density have a thermal conductivity 1/3–1/9 that of ordinary concrete and 1/3–1/6 that of common brick. A combination of these Lightweight materials are used advantageously as admixtures in concrete, replacing a portion of the cement without impairing the strength. Therefore, values for bench scale material samples are based on formulations which contain a binder characteristic having an alternate compressive strength of 20,000 pounds per square inch and hollow glass spheres which are reported to show approximately 50% breakage at 10,000 per square inch. Formulation for normal bench scale samples are based essentially on optimum binder to hollow sphere ratio and two supplemental compromised Lightweight material.

Development of higher controllable strength binder characteristic and higher strength ratio to supplemental comprised Lightweight material in equal particle distribution, the compressive strength is improved. Results for increases in compressive strength properties as well as other samples related properties are based on the following specific bench scale laboratory activities. A recovery recycled technique and designed apparatus for the improvement in low/high binder density made suitable for specific sample mixture formulation which is identified as Advanced Material In Construction Application (AMICA). It's compressive yield strength and related characteristics are compatible with other marketable higher cost binders. Thus, an increase of 50% over the present strength and potential applicability's are the present strength monitored for the Straight of Messina construction under pending considerations.

An improvement in the compromised strength formulation of basic light material such as Microballoons (synthetic) and Pumex Lapilli derivatives (natural) are used as hollow spheres. Also the development of scale up related projects feasibility studies and "In-situ" R/D activities of improving binder agents and products technology as well as and process procedures can be expected to enhance other connected characteristics such as tensile, sheer, fatigue strengths, permeability and floatability.

To produce an optimum Lightweight sample material, one with highest buoyancy, the problem of relatives sphere size, sphere thickness and spacing of sphere distribution in the mixture formulation is studied both mathematically (St. Andrea Cross calculation) and experimentally (Laboratory International performances). At present, specific activity is carried on to make larger bench scale samples a basic formulation with the target of developing a buoyancy system capable of withstanding a pressure of 13,500 pounds per square inch.

6 PROPERTIES AND CHARACTERISTICS OF BASIC MATERIALS

There is a definite shift toward physical (foams) mainly because producers of chemical Lightweight material cannot control cell size and strength. With physical foams, it will accurately predict the extent of "foaming" in the end product because the cell structure is prefabricated via addition of the pre-made spheres; it does not depend on a relatively unpredictable chemical reaction carried out "In-Situ" as do structures based on chemical light weight materials. Micro-balloons offer good thermal and electrical insulation properties, as well as exceptionally high strength to weight ratios that is particularly demonstrated from specific samples. The micro-balloon spheres are tiny, hollow bubbles of glass ranging in size from 10–270 microns in diameter. They can be manufacture in various particle size ranges: however, the typical average particle size is approximately 100 microns. The bulk density is approximately 12–15 pounds per cubic foot with a true density of each particle (measured by liquid displacement) of 0.30–0.35 grams per cubic centimeters. The high melting (1400ºF), glass composition, along with the low thermal conductivity of the particle resulting from its hollow structure, gives to the end product very interesting insulating properties.

It must be underlined that this new mixture formulation of the basic light materials can not be considered as a simple laboratory sampling activity. The unique properties of these combined ingredients are not only reflected in handling and processing techniques, but also in the physical properties of the final product. In using these basic Lightweight materials in (AMICA) binder solution it is defined that particular mixing techniques, (AMICA) viscosity, curing times, as well as weight–volume relationships are affected by the low density of material used. Sampling experimentations enable to anticipate physical changes and an understanding of the process conditions so that the full utility of this basic formulation can be applied.

7 LIGHTWEIGHT MATERIAL SYNTHETIC PRODUCTION

Microballoons in general can be produced from cellulosics, starches, polystyrene–practically any film of foaming material. However, the basic Lightweight material used for Laboratory International sampling formulation, identified as Density Volume MicroBalloons (DVMB), is described in a process which a volatile solvent solution, containing film forming material and the substance that readily generates gas (such as conventional blowing agents) is subdivided into droplets at the top of a spray drying chamber. The droplets then fall through the drying atmosphere, where the solvent is evaporated and a whole free, tough surface skin is formed on the particle simultaneously with the liberation of gas from the gas generating material.

In this way, gases liberated within the particle coincident with formulation of the particle and is trapped within the surface skin. The contained gas prevents collapse of the particle's wall under pressure of the atmosphere. A later process improvement of (DVMB) has particular pertinence for sampling formula experimentation. Its main difference is that the feed is introduced near the bottom of a furnace into an ascending column of hot furnace gas, instead of at the top of a spray-dryer. The latter process provides better safeguards against overheating or under heating of the particles, thereby producing a Lightweight product that has a lower bulk density and often more industrial–commercial possibilities.

The particles are very resistant to water, alkali, acid and hydrocarbon. At a 6°F mean temperature, their thermal conductivity is .38 BTUs/hour/F/square foot/inch. The particles start to soften at 1200°F and melting is completed at 1400°F. The physical properties of this material are compared with those of other sources. The true density of (DVMB) is about 1/3rd that of wood flour and only 1/9th that of the mineral source. The mineral source is considerably smaller than the (DVMB) whereas wood flower is in the same particle size range. Because fillers are used for their volume rather than their weight and because of the wide density differences between fillers, oil absorption is expressed on a filler weight basis. On this basis, the (DVMD) are more readily wet then the mineral filler. From this data, it is evident that many of the properties of the ideal filler are fulfilled. Furthermore, by using information from specific sites, data collected and generated from the raw material sources to finished products and mainly the experience gained in the learning by doing approaches from the carried on activities in the recovered advanced material in construction application (AMICA), as a resinous additive, the results are much lighter.

8 CARBON FIBER AND MICROBALLOONS: (PANE) POLYACRYLONITRILE ENHANCEMENT

Bridges, highways, railroads of today and tomorrow will continue to be built with lighter materials such as composite derivatives from Polyacrylonitrile Enhancement (PANE) to carbon fibers, "Microballoons" and others. Now, cars, aircraft, boats and rockets are already built with these lightweight materials. The most important aspect of these undertaken activities, is to figure out how to reduce the cost of the light weight precursors of the carbon fiber (Polyacrylonitrile and "micro balloons" as per related special issued alternative patent). High volume renewable sources of carbon fiber feedstocks such it has (PA and E.) are necessary to be provided mainly by the recycled particular materials, mainly from cars, tires, carpets and other compatible scraps from cars, aircraft, etc.

Today, the cost to purchase commercial grade carbon fiber ranges from \$8–\$10 per pound, and to be successful, it must be reduced by 60–70% between \$2.50 to \$4.50 per pound. At this market price it would be feasible for the auto industry alone choose more than 1,000,000 tons of composites. The manufacturing of a car, annually, requires approximately 300 pounds of composites per vehicle. Much more if it's needed for aircraft and an enormous amount for bridges and road pavement. One of the biggest advantages of carbon fiber is that it is 1/5 the weight of steel and yet just as strong and stiff. This makes it an ideal material for structural components, not only for automobiles, aircraft and others, but mainly for one span long suspension bridges. These lightweight materials could reduce a vehicle's weight by 60% and fuel consumption by 30%, according to some studies. Reducing vehicles' weight together with bridge dead loads, it allows to span the bridge much longer also the resulting gains in fuel efficiency, made in part because smaller engines could be used with lighter vehicles, would reduce above all the omission of CO_2 and other greenhouse gases by 10–20% at least. Another focus is on developing an efficient carbon fiber oxidation process, which would significantly increase production and lower cost of this material (see white coal basic material activities (WCBM) and an alternative to produce (PANE)'s precursor Polyacrylonitrile; patent description). This is a process that can generate acrylonitrile as (PANE) precursor by recycling mainly the "Marmettola" and other scraps from a specific location. This is a technique used "in situ" in non-traditional research and development activities were the materials rest in the open atmosphere instead of in a "pseudo-controlled" closed system environment. These techniques to produce a precursor to carbon fiber and other lightweight materials permit to evaluate these related revolutionary alternative processes on a comparable basis against other conventional commercial industrial processes of today and tomorrow's generations.

9 ULTRALIGHTWEIGHT MATERIAL CONSTRUCTION (ULMC)

To make the end products such as blocks, bricks, concrete slabs, pavements and asphalt, roofs, multilevel parking garages, buildings and bridges of all kinds, a new element is added into the mix formulation. Aluminum powder is used for the blocks of aerated concrete. The process in itself involves making mixture of Portland cement, Lightweight material as suitable aggregate, aluminum powder and water. The batch is poured into a form or a mold and allowing it to expand and set. Such a process is aimed mainly to span bridges to reduce the dead load. The outstanding results of (ULMC) are lightness, high thermal and acoustic insulating properties and resistance to damage by heat and flame. The principal uses in building construction are for floors, floor fill, roof, ceiling and partition slabs (see Figure 3). The lightness of construction of this particular formulation permits in addition of two more extra floors and parking garage structures as well as for larger ones in suspension bridges such as the "mega bridge", the straight of Messina Bridge taken in reference.

The principal requirement of batch, adding aluminum powder, is that it must be maintained inactive when mixed with the cement before the addition of water. The chemical reaction of this mixture, after the concrete is mixed, the aluminum powder should react with the water and the (WCBM) free of the cement to produce hydrogen (H_2). The evolution of (H_2) is going to be gradual and yet complete, so that the Lightweight mixture is uniformly expanded with small gas bubbles, which cause it to swell to about twice its usual volume and give a porous sponge-like texture. This type of ultralight concrete (ULC) will vary with the properties desired in the tailored finished product. The product can be manufactured in precast form and weights as low as 25 pounds per cubic foot. Ordinary concrete weights about 150 pounds per cubic foot. The lightest material is usually made with cement and microballoons alone, while for heavier products, mixing perlite, pumice, fly-ash, sand, marmettola or recycled crushed concrete which is mixed with cement. A general formula, which is used for samples demonstration and can be varied within the limits indicated to produce a material weighing from 40–80 pounds per cubic foot would be:

Portland cement	80 pounds–1 bag
Light weight (pumice, perlite)	0 to 3–4 cubic ft.
Water	6.5 to 8.5 gallons
100 mesh aluminum powder	3.25 to 4 ounces

This must be underlined that the proper water-cement ratio is extremely important in the production of (ULC). It must be greater than that of ordinary concrete, because it as usually made, concrete is not sufficiently fluid to permit satisfactory expansion. The water-cement ratio for expanded (ULC) should be between 6.5–8.5 gallons per bag (80 lbs.) as compared with 5 gallons per bag for ordinary concrete.

The quantity of water is very important to result in a rather "soupy" batch which is necessary if maximum expansion is to be obtained. It is in fact possible to alter the weight per cubic foot of (ULC) by varying the watered cement in the Lightweight materials (Microballoons, Pumice, Perlite, Vermiculite) cement ratios. However, it may be also necessary to vary the aluminum-cement ratio slightly, especially in the production of heavier weights of (ULC). The heavier material, containing larger proportions of fly-ash, silica sand or other aggregate, requires additional quantities

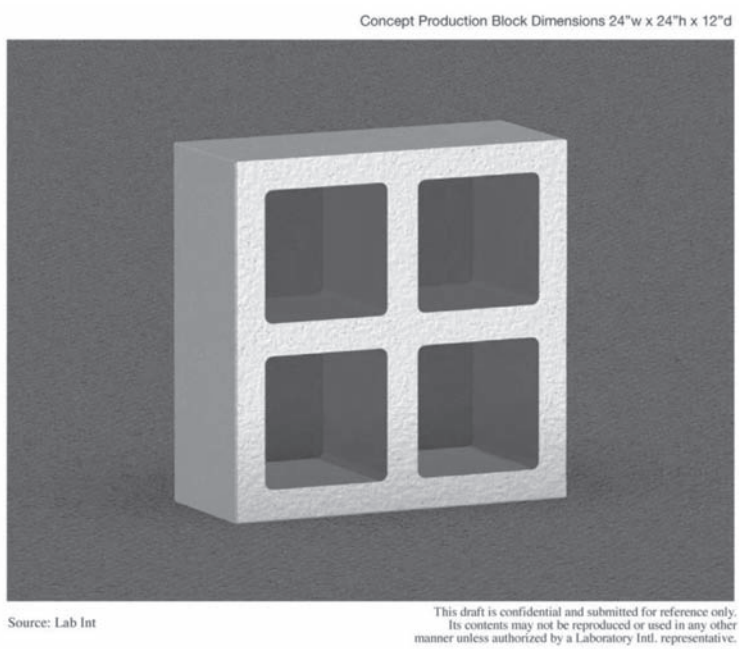


Figure 3. Caption of a proposed ultra light concrete (ULC) construction block.

of aluminum powder to compensate for its greater weight. Because most light aggregates are more absorbent than sand, the amount of water used with them in making (ULC) is greater; about 9–10 gallons per bag of cement as against 6.5–8.5 gallons for expanded concrete made with sand.

The density and other properties of (ULC) will vary with the character of the filling material and the amount of gas expansion. The compressive strength of (ULC) will range from about 100–200 pounds per square inch for the lightest products to about 1000–1500 pounds per square inch for the mixtures weighing 70–80 pounds per cubic foot. (ULC) is an excellent fireproofing material because of its substantial volume of voids and low thermal conductivity, coupled with good resistance to cracking or spalling when heated. In one test, the lower flanges of steel beams, protected by 2 inches of 60 pound expanded (ULC) is exposed to fire on three sides. Although the heat of exposure during the four hour test averaged 1700°F, the temperature of the steel protected by the expanded (ULC) increased less than 230°, from 47–221°F. Such protection is important because fire damage to the structural supports (structural steel or honeycomb aluminum alloy) of bridges, buildings or multilevel parking garages increases warping and buckling.

10 TRIP MEMO TO ITALY (SICILY) JULY 31 TO AUGUST 5, 2008—DRIVING THROUGH THE CONSTRUCTION SITE AND THE "PONTE DI MESSINA" SENSORS WARNING SIGNALS

Many types of sensor systems currently exist in the open market for bridges, roads, railroads, tunnels and other applications to constantly monitor traffic/construction operations. However, long-term or political/economic financial signals from specific sensor systems quite often are employed for diplomatic, political, technological and economic indicators to monitor and at the same time to forecast not only the weather temperature, atmospheric pressure, precipitation, earthquake tremors and wind direction as well as the traffic volume, but mainly to forecast hostilities against the bridges construction such as the "Messina Suspension Mega-Project". The bridge promoters may react by strengthening the efforts by negotiating treaties or concessions, or by taking other alternative actions. The political warning signals, is often an equivocal and incapable of disclosing fully a protester demonstration, the results are on evaluated intentions and then neglected precarious situation in terms of finance/risky capital investment construction target to make sensors imperative.

Against the "Ponte di Messina" construction protesters could be a medium term political maneuvers, or strategic warning, which usually may involve a time span of a few days or weeks, a notification or judgment that those who oppose the bridge construction may have initiated hostilities maybe proven imminent. The sensors can be complementary for short-term, or tactical warning often hours or minutes in advance, is a notification that the "against the bridge" protesters have initiated hostilities. Obviously, warning and detecting from contemporary sensors are separate functions. Today's sensors or detection devices either for bridge construction plans or other purpose applications are perceived prevention of failures, possibilities of an attack, the reality and animosity of "no ponte" protesters, the nearness of the factions interest in the bridges golden pot, the location history, the culture and relationship to the "Traghetti"(ferryboat) business, the activities licit and illicit, the retaliation weapon capability, or it's more swiftly changes in the political, economic, finance and technical pursuit.

The "Ponte Stretto di Messina" sensors are above all warning systems that include detection devices, but also imply the judgments, decisions and actions that follow receipt of the sensor's information for construction and connected project plans development, especially for capital investment (see article "Spectrum Bridge Acts", Friday, September 5, 2008 page 36 of the Wall Street Journal). Warning encompasses communications, analysis of information, decisions and appropriate actions that go from the raw materials procurement to the practical bridge construction site and capital investments specific program plans. Visual observations still remains important, supplemented by telescope, cameras, heat sensing devices, low light level devices, radar, acoustic, seismic, chemical and nuclear detection devices. The products for the "Ponte Stretto di Messina", the suspension bridge and/or the output of these particular sensor systems are not only complicated but sophisticated enough to require computers to condense and summarize the data for the proper decision maker. It is obvious to assert that quite often, the most expensive portion and weakest link of these warning sensor systems is not the device in itself but the communication and evaluation assessment of the particular system. Technology of any kind is required especially for this mega project, the largest bridge in the world. Like infrared sensors on the ground, or installed on the bridge, can detect such hotspots as motor vehicle engine (particularly truck cement mixer and site delivery account. (My own experience working on I-71; highway in Cleveland, Ohio 1964–1965–a bridge covered up) these sensors have a great location accuracy and high sensitivity to signals, without registering such false targets as sun reflections. Given the nature of the "Ponte di Messina" location, seismic detectors have priorities and are a necessity to monitor infiltration and vehicle detection. Discussions with engineers and others concerning bridge construction- trip memo to Sicily, driving through Salerno-Reggio Calabria to find out directly July 31 to August 1, 2008.

11 CONCLUSIONS

The Messina's mega bridge, like many of today's large and small bridges around the globe, is no longer an epic engineering project that would have taken years of construction work and enormous amounts of heavy steel and concrete. New designs, proven new materials and up to date construction techniques, the whole undertaken project activities means that bridges can be put together in a matter of days. Bridges made of new materials, particularly "Microballoons" in fiber reinforced polymers have been around for some time, mainly in Europe and North America. They are built and traverse almost everything from rivers, sea and mountain valleys. These new Lightweight materials have advantages over old types of materials; they require minimal maintenance during the Bridges' life spans. Bridges have up to date new designs and related Lightweight material components also have other advantages in cases of catastrophic events; they can simply disassemble and reassemble at a moments notice.

Sensors for bridges are not new, although a new type of sensor is based on a conventional optical fiber, the end of which has been modified in various applications. A very short laser pulse can be launched into the fiber and each grading also reflects a small amount of the pulse. These reflections from nearby gratings arrive much sooner than those of gratings at the far end of the fiber and the intensity of each reflection can reveal the local temperature, strain profile for an entire bridge from a single embedded fiber sensor. Evidently, optical fibers have already reduced costs while delivering ever-increasing amounts of data extending the sensing bridges constructions.

REFERENCES

"A Bridge With Three Double Lanes—Traffic and Parking Garages in Italy." A cooperative endeavor and effort to efficiently meet the regional environmental problems. Project AUA, June 1973.

"A Bridge With Three Double Lanes In Retrospective." Project AUA. Trip memo—to—Italy; November 6–20, 1987. Di Giannantonio, Gioia, personal file.

Di Giannantonio, Gioia, personal file.

Junius D. Edwards and Robert I. Wray, 1955. *Aluminum Paint and Powder*, New York, NY: Reinhold Publishing Corporation.

(Lightweight Materials such as pumicite, perlite, vermiculite, diatomite).

"Microballoons." Publication and Article, a process to make hollow-glass-spheres, extracted from Veatch, Dr. F., personal file.

Meneghini, Domenico, *Chimica Applicata e Industriale* by Casa Editrice Dr. Francesco Vallardi-Societa' Editrice Libraria-S.P.A.; 1965.

R.B. Ladoo and W.M.Myers, 1951. *Nonmetallic Minerals,* New York, NY: McGraw-Hill Book Co,

Von Fisher, 1948. *Paint and Varnish Technology*, New York, NY: Reinhold Publishing Corporation.

William Benton (publisher) 1943–1973. *The New Encyclopedia Britannica, Micropedia*, *Volume II*, City: William Benton (Bridges Construction and History, Super long span bridges and exotic materials, Suspension bridges).

5 *Bridge instrumentation and monitoring*

Chapter 19

Modeling and instrumentation of the Tobin Memorial Bridge in Boston, Massachusetts, USA

B. Brenner
Fay, Spofford & Thorndike, LLC, Burlington, Massachusetts, USA

E. Santini-Bell & W. Durack
The University of New Hampshire, Durham, New Hampshire, USA

M. Sanayei & E. Pheifer
Tufts University, Medford, Massachusetts, USA

ABSTRACT: The Massachusetts Port Authority has contracted consultant services for the structural modeling and instrumentation 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 the fall of 2008. This paper will present those representative analytical structural models, including several special studies focused on the rotational stiffness of the truss connections, boundary conditions, and built-up member properties. An instrumentation plan that included strain gages, accelerometers, tiltmeters, and temperature sensors was developed and deployed for the Little Mystic span of the Tobin Memorial Bridge during the winter of 2009. The collected data will be used to verify the analytical structural models. These models and instrumentation plans will be used as part of the structural health monitoring and condition assessment program for the management of the Tobin Memorial Bridge.

1 INTRODUCTION

The condition assessment of a bridge is traditionally carried out using subjective visual inspections and load ratings using guidelines set forth by the American Association of State Highway and Transportation Officials (AASHTO). Many states use PONTIS to manage the bridge inspection reports data and the regulations in the National Bridge Inventory Standards (NBIS) for inspection protocols. Bridge owners rely on the information obtained from visual inspections and load ratings for their decision-making purposes, both for repair or rehabilitation, as well as complete replacement.

Visual inspections currently serve as the primary means to evaluate the condition of virtually all highway bridges in the United States. The degree of deterioration in the structural components of the bridge is assigned a numerical rating from zero to nine, with zero being failed and nine being in excellent condition. The ratings are subjective to an extent, based in part on the experience and approach of the inspectors. Furthermore, visual inspections are, by definition, limited to what can be seen. Hidden structural members, or conditions that are difficult to directly view, may not receive the same level of inspection treatment (Farhey, 2005). The inspector' expertise is required to evaluate the adequacy of the overall structural condition of the bridge.

In 2000, Brent Phares polled state and county DOTs as well as inspection contractors and found the most common form of nondestructive evaluation is through visual inspection (Phares, Rolander, Graybeal, & Washer, 2000). Visual inspections are typically scheduled every 24 months. Special inspections may be arranged at a more frequent interval to closely monitor any problem areas until issues are resolved by repair or replacement. This approach is adequate for most bridges under normal conditions, but not if there is a damage-causing event such as impact or natural disaster between inspections.

Load ratings are performed using either the AASHTO allowable stress, load factor, or load and resistance factor rating methods. The purpose of load ratings is to determine the load capacity of an existing bridge. Development of a bridge's load capacity indicates is a load limit should be imposed on the bridge, if special freight policies should be implemented, and to develop truck routes around congested urban areas. As with visual inspections, load ratings are another tool that bridge owners have become dependent on for decision-making purposes. Software is available to aid in the load rating process, including the AASHTO Bridge Analysis and Rating System that has been in use throughout the country since the early 1970's. Advances in computing power and availability have provided a platform for consistently improving structural analysis and load rating software, including AASHTO's new bridge load rating system named Virtis/Opis (Thompson *et al*. 2000).

1.1 *Benefits of using modeling and instrumentation*

Computer modeling of an existing bridge is an objective means of evaluating bridge condition. It can be used to supplement more subjective visual inspections. However, while the model seems to be more objective, errors in computer modeling of structural components can be significant. Errors include oversimplification of assumptions, uncertain boundary and continuity conditions, and unknown loading conditions. Instrumentation of the bridge with an array of sensors designed to capture structural responses of the bridge may be used to help validate the computer model.

The combination of a calibrated analytical model along with sensors to measure the structure's response can be used to develop a structural health monitoring (SHM) system. This system can provide supplemental data for reporting on the condition of a structure. Economic benefits of such a system include the reduction of costs associated with routine maintenance and rehabilitation, an increased serviceable lifespan of the bridge, and more effective repair efforts. Structural components that show signs of deficiency in the computer model may not be easily accessed using traditional visual inspection. Components appearing to have damage may still have sufficient capacity to carry design loads. Objective assessment, in combination with subjective assessment, helps to direct limited maintenance funds to components in need of repair. Also, a calibrated structural model can be used for load rating and permitting.

Safety in the operation of the structure is enhanced by modeling and instrumentation. A structural health monitoring system may detect unusual structural behavior at an early stage, thereby reducing the risk of sudden and catastrophic failure. Appropriate monitoring requires the development of an accurate computer model that effectively characterizes the entire structure, including the continuity and boundary conditions.

1.2 *Special studies of connections and boundary conditions*

Special studies of connectivity and conditions are needed to develop a computer model that accurately characterizes a real structure. Examples of factors influential to computed structural responses include connection stiffness and boundary conditions. Appropriate input of these factors into the model can have a significant impact on the overall accuracy of the model. Performing special studies on influential components of the structural model can help to improve the accuracy of global and local structural models.

2 TOBIN MEMORIAL BRIDGE

2.1 *History of the Tobin Bridge*

The Maurice J. Tobin Memorial Bridge carries US Route 1 across the Mystic River to connect the city of Chelsea and the Charlestown section of Boston, Massachusetts. Construction on the bridge began in 1948 and was opened to traffic in 1950. The bridge was originally designed and constructed under the direction of the Mystic River Bridge Authority, a quasi-government group charged with developing a suitable crossing over the Mystic River. The Mystic River Bridge Authority later became part of the Massachusetts Port Authority, who serves as the current owner and maintainer of the bridge. The Tobin Memorial Bridge is considered a critical link in the network of Boston area highways, connecting metropolitan Boston with business and residential areas to the north. The components that compose the approximately 2¼ mile long structure include 32 approach spans on the Chelsea side, 36 approach spans on the Boston side, the Little Mystic Span, the Big Mystic Span, and the Toll Plaza. Figure 1 shows a 3D rendered view of the components of the Tobin Memorial Bridge with Boston in the background.

The Tobin Memorial Bridge consists of three northbound lanes on the lower level of the structure and three southbound lanes on the upper level. The Big Mystic Span, also referred to as the Main Span, is a three-span cantilevered steel through-truss measuring approximately 1,525 feet in length. The Little Mystic Span is a simply supported Warren truss approximately 439 feet in length. Figures 2–3 are photographs of the Big Mystic Span and the Little Mystic Span. Truss

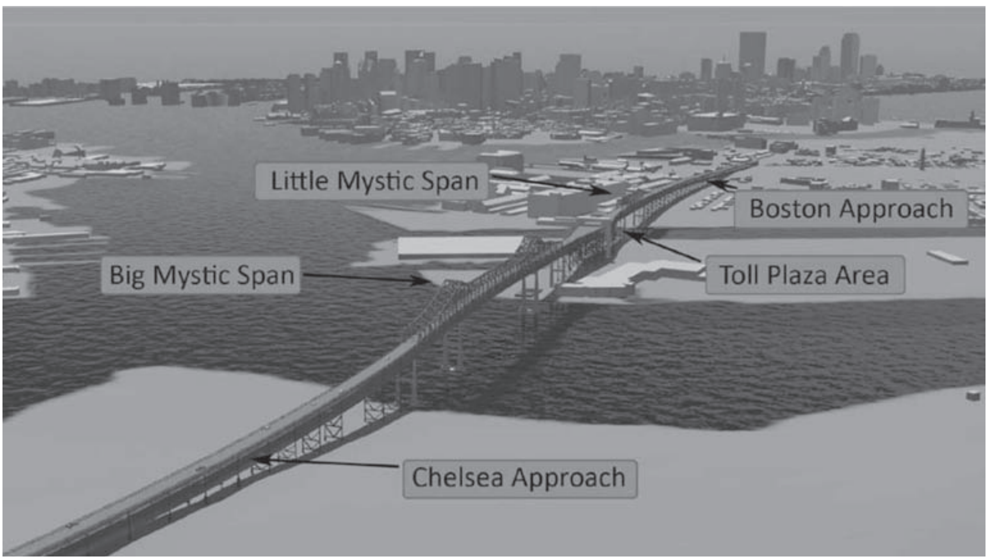


Figure 1. 3D rendering of the components of the Tobin Bridge.

Figure 2. The Big Mystic Span from Chelsea.

Figure 3. The Little Mystic Span from Charlestown.

members in the Big Mystic Span and the Little Mystic Span are mostly built-up steel sections that are bolted and riveted to steel gusset plates.

2.2 *Project for structural modeling*

The Massachusetts Port Authority (Massport) sent out a Request for Qualifications early in 2008 seeking structural modeling and analysis of selected components of the Tobin Memorial Bridge. The winning consultant was to provide finite element (FE) modeling of the selected components, with the goal of helping improve future capital projects. The scope of work included review of existing construction documents, creation of computer models of selected bridge spans and components, development of a cost effective instrumentation plan, and verification and adjustment of the model based on the measured data.

Massport contracted the consultant services of a team led by Fay, Spofford, & Thorndike, LLC, and supported by Tufts University, the University of New Hampshire, and Geocomp, Inc.

2.3 *The complexity of maintaining the Tobin Memorial Bridge*

A bridge as complex as the Tobin Memorial Bridge requires a rigorous maintenance program that could benefit significantly from the implementation of a Structural Health Monitoring system. Maintenance of the bridge is complicated not only by the length of the structure, but also by challenges of access for physical inspection and by the large number of built-up steel sections and rivets in the structure. The bridge will be modeled and instrumented to help improve the maintenance program.

3 DEVELOPMENT OF COMPUTER MODELS

3.1 *More than just "stick" models*

To improve maintenance on the Tobin Memorial Bridge and address various concerns about the performance of the structure, a structural health monitoring system (SHM) is being implemented. The development of a "baseline" finite element (FE) model that accurately represents the geometry and structural parameters of the structure is the first step in the SHM process.

Selected components of the Tobin Bridge have been developed as AutoCAD "stick" models, or three-dimensional line models, using the original construction documents. Figure 4 shows the AutoCAD "stick" model of the Big Mystic Span. Each structural element is represented by at least one line element in the model. Every crossbeam, stringer, truss member, floorbeam, sway frame,

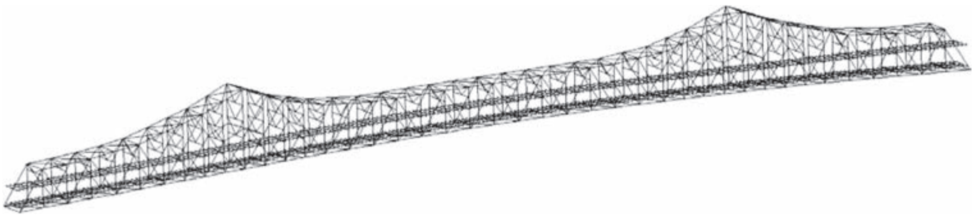


Figure 4. 3D "stick" AutoCAD model of the Big Mystic Span.

Figure 5. Computer model development process.

diaphragm, and bracing member is included in what is called a microscopic-level model. Previous research (Catbas *et al*. 2007) on a bridge similar to the Big Mystic Span demonstrated that a microscopic-level model could predict the structural responses of a non-destructive test with reasonable accuracy. Modeling at a highly detailed level eliminates many assumptions about the effective behavior of structural elements, which are assumptions typically made in a smeared modeling approach. Smeared modeling employs an approach combining a number of structural elements into a few finite elements with effective properties; microscopic-level modeling involves a geometrically complex model but the calculations and assumptions of effective properties do not need to be made.

The commercial finite element analysis programs SAP2000 and GTStrudl were chosen for this project. Both programs were readily available to the authors and can easily generate structural analysis models through their Application Programming Interface (SAP2000) and Text File Input (GTStrudl).

Creation of FE models from AutoCAD models is accomplished using a three step process shown in Figure 5.

The geometric "stick" models are transferred to an Excel spreadsheet using the AutoLISP programming language. Imported geometry is named and assigned important numerical inputs in the spreadsheet including material and section properties, connection stiffness, and loading conditions. Visual Basic routines create the FE models directly from the spreadsheet data.

Standard 3D frame elements compose the steel members of the FE models. Shear deformation is assumed to be negligible (Timoshenko and Young 1935). The concrete deck is represented as a mesh of shell elements with a specified thickness. Figure 6 shows the frame and shell elements that compose the SAP2000 model of the Little Mystic Span.

Frame elements were chosen over truss elements because they have six degrees of freedom at each node. The three rotational degrees of freedom in frame elements allow for proper characterization of the rotational stiffness of the connections.

3.2 *Connections*

Truss design typically assumes that the members are connected by frictionless pins and are free to rotate. Forces are assumed to be present only in the axial direction of connecting members.

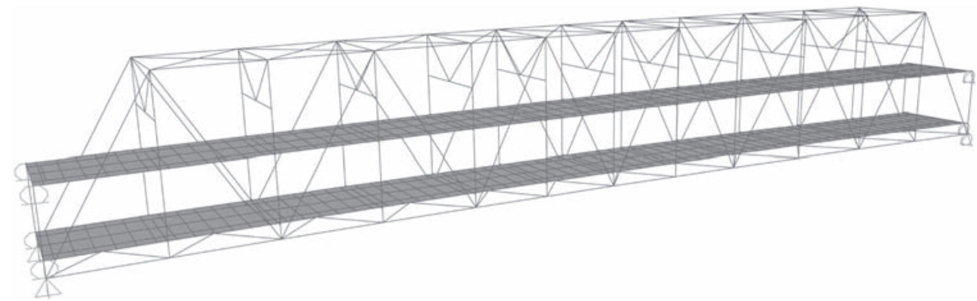


Figure 6. Frame and shell elements in the Little Mystic Span SAP2000 model.

However, the assumption that joints are free to rotate is inaccurate. Friction will always be present at the connections. Secondary stresses, due to shears, moments, and torsion, build up in the members due to the rigidity of the connections. In the case of the Tobin Bridge, truss members are connected by large, riveted gusset plates, probably leading to introduction of secondary stresses in the members.

Secondary stresses have been long considered in the design of truss bridges. In 1877, the polytechnic school in Munich offered a prize for the solution of how to calculate secondary stresses in a riveted truss. Heinrich Manderla proposed a method to calculate secondary stresses that won the prize (Manderla 1880). Manderla's solution, along with other approximate methods proposed by various German professors, provide a way for engineers to calculate secondary stresses. These calculations require an extensive amount of time. It is interesting to note, however, that the study of secondary stresses in riveted trusses all but disappeared from research after the late 1930s.

Advancements in technology and computing power have improved the engineer's ability to calculate secondary stresses and include these effects in the structural model. GTStrudl® and SAP2000® allow connection stiffness to be specified for each member. The connections can be modeled as fully-fixed, fully-pinned, a set of linear translational and rotational spring or a fullly-populated stiffness and mass matrix, which could include off-diagonal terms that account the interaction between degrees of freedom. A study of the Little Mystic Span of the Tobin Memorial Bridge revealed that secondary stresses for a fully-fixed model were significant compared to primary stresses as shown in Table 1. The analysis was performed using two load cases: two HS-20 trucks on each deck at

Table 1. Comparison of primary and secondary stresses and strains for the Little Mystic Span SAP2000 model.

	Member	Axial stress (ksi)	Bending stress (ksi)	Total stress (ksi)	Axial strain (με)	Bending strain (με)	Total strain (με)
Midspan loading	L5L6	0.92	0.78	1.71	31.80	27.00	58.80
	U5U6	–1.19	0.36	–0.83	–41.10	12.50	–28.60
	L0U1	–0.67	0.26	–0.41	–23.20	8.96	–14.24
	L1M1	0.02	–1.26	–1.24	0.69	–43.40	–42.71
	U1L2	0.90	–0.09	0.81	31.20	–3.13	28.07
Support loading	L0L1	0.30	0.05	0.35	10.20	1.67	11.87
	L1L2	0.20	0.29	0.49	6.79	9.80	16.59
	L0U1	–0.32	0.10	–0.22	–11.00	3.52	–7.48
	L0M0	–2.69	2.21	–0.48	–92.80	76.30	–16.50
	L1M1	–0.01	–0.86	–0.87	–0.09	–29.70	–29.79
	N1U1	0.44	–0.19	0.25	15.20	–6.70	8.50

the midspan and at the supports (Member designations are based on the grid shown in Figure 7). Books on the subject of secondary stresses contend that differences in primary stresses between the cases of fully-fixed and fully-pinned can be neglected for practical purposes (Grimm 1908). The SAP2000 analysis confirmed that primary stresses do not vary significantly between the fully-fixed and fully-pinned assumptions.

Characterizing the actual fixity of the connections is a complex process. Analytical studies of connections performed at the University of New Hampshire (UNH) will provide appropriate connection stiffness inputs to the baseline global FE model. The UNH researchers are currently created detailed FE models of the typical connections on the Little Mystic Span of the Tobin Bridge. FE models of the connections are being created in SAP2000® to analyze the rigidity of the connections. Figure 8 shows a SAP2000® model of a typical L1 connection for the Little Mystic Span. Parameter estimation algorithms will be used to populate the stiffness matrix to represent the connection in the overall model of the Little Mystic Span. (Santini-Bell *et al*. 2008).

3.3 *Built-up members*

The computer models are further complicated by the complexity of the steel members. Members are built-up sections consisting of plates and angles riveted together. Hand calculations and

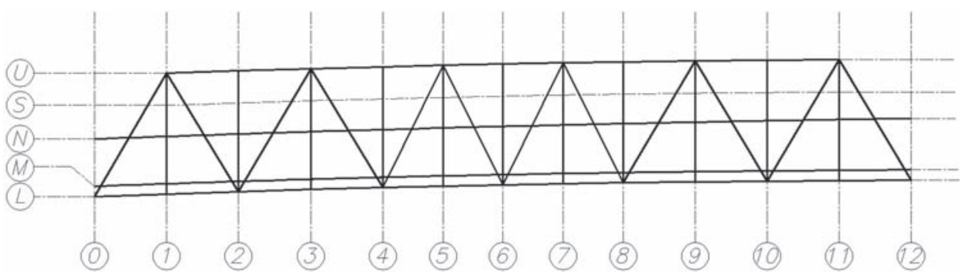


Figure 7. Joint and member naming convention for the Little Mystic Span model.

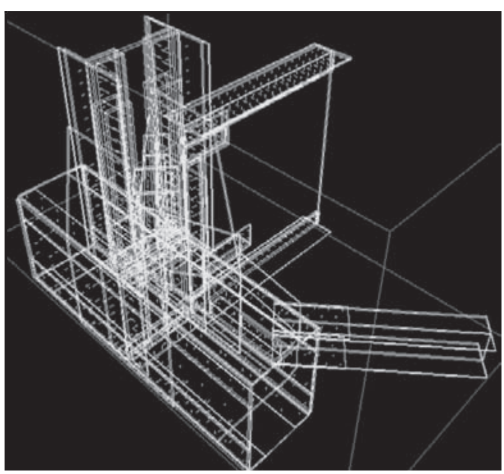


Figure 8. SAP2000 model of typical L1 connection for the Little Mystic Span.

AutoCAD drawings of member sections were used to calculate section properties. Figure 9 shows an example cross-section of a built-up steel member.

Section properties were calculated based on the original design drawings. All of the steel in the model is assumed to have a modulus of elasticity of 29,000 ksi.

3.4 *Piers*

An important consideration in the modeling of any structure is how to represent the boundary conditions. This project will also model the support piers and calculate the support that they provide to the superstructures. This support will be compared with the typical pinned or fixed assumption.

4 INSTRUMENTATION

To calibrate and verify the FE model, selected components of the Tobin Memorial Bridge will be instrumented and non-destructive testing will be performed. Instrumentation used to capture the structural responses of the bridge will include strain gages, tilt meters, temperature sensors, accelerometers, and weather stations. Data will be collected by data loggers at various locations on the bridge and will then be streamed wirelessly to Geocomp, Inc. Post-processing of the data will provide strains and stresses in the actual structure that will be compared to the outputs of the FE model. The model will then to be calibrated to reflect the measured data. Once the FE models is verified that it mimics the actual structural responses of the Tobin Memorial Bridge, the model eventually will be incorporated into the structural health monitoring and condition assessment program for the Tobin Bridge.

4.1 *Little Mystic Span*

An instrumentation plan has been developed, and is beginning to be implemented, for the Little Mystic Span. Table 2 gives types and quantities of instruments that will be utilized on the Little Mystic Span.

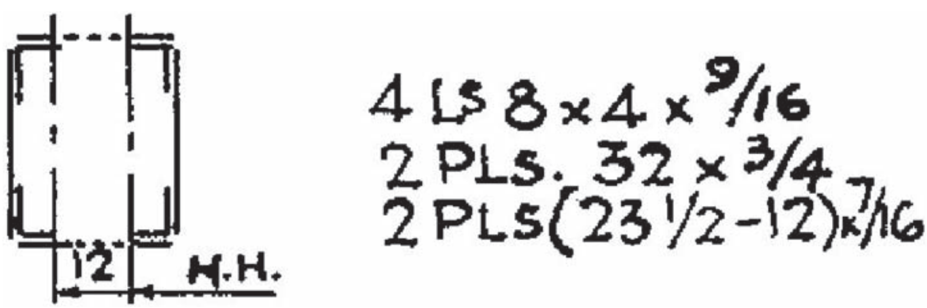


Figure 9. Built-up steel member cross-section.

Table 2. Instrument types and quantities for the instrumentation of the Little Mystic Span.

Instrument type	Number of instruments
Strain Gage—Member	80
Strain Gage Rosettes—Connection	12
Tiltmeters w/Temperature sensor	2
Accelerometers	6
Weather station	1
Temperature sensors	6

Figure 10. Installation of data loggers on the Little Mystic Span at connection L6.

Installation of instrumentation began with setting up the data loggers along the railing of the east side of the Little Mystic Span as shown in Figure 10. Geocomp, Inc. will be able to confirm that the sensors are functioning as they install them on the structure by plugging them into the loggers. Installing the loggers first is advisable to facilitate sensor calibration as they are installed.

5 CONCLUSIONS

A team of engineers and researchers is participating on a unique project to model and help evaluate the Tobin Bridge, the longest bridge in the Commonwealth of Massachusetts. The project has been authorized by the Massachusetts Port Authority to assist in maintenance and rehabilitation of the complex bridge structure. To date, the team has developed a group of three dimensional geometric and structural finite element models representing sections of the bridge. The combination of a global structural model that captures geometry of the truss member, floor beams, deck elements and diagonals with several detailed special studies finite element models will increase the potential use of this model and its ability to reflect the actual behavior of the Tobin Bridge. These special study models will focus on the connections and boundary conditions that are typical assumed to function as a pin or fixed connections in conventional structural modeling.

An instrumentation program has been developed for the Little Mystic Truss span to verify the model and eventually to be used as part of a Structural Health Monitoring System. The instrumentation plan was developed with input from the researchers, bridges designers, instrumentation specialist and bridge managers and owners. The inclusion of all parties related to the Tobin Bridge will help to insure the success of this project. Work proceeds on refining the modeling and developing instrumentation programs for other parts of the bridge. Thanks to this forward-looking project, it is envisioned that the Massachusetts Port Authority, the owner and manger of the Tobin

Bridge, eventually will have a functional Structural Health Monitoring system that will greatly aid in maintenance of the Tobin Bridge.

REFERENCES

Catbas, F.N., Ciloglu, S.K., Hasancebi, O., Grimmelsman, K.A. & Aktan, A.E. (2007). “Limitations in Structural Identification of Large Constructed Structures,” *J. of Structural Engineering*, ASCE, 133(8), 1015–1066.

Farhey, D.N. (2005). Bridge Instrumentation and Monitoring for Structural Diagnostics. *Structural Health Monitoring*, 301–318.

Grimm, Carl Robert. (1908). *Secondary Stresses in Bridge Trusses*, 1st edn. New York, NY: John Wiley & Sons.

Manderla, H. (1880). *Die Berechnung der Sekundärspannungen.* Allgem: Bauzeitung.

Phares, B.M., Rolander, D.D., Graybeal, B.A. & Washer, G.A. (2000). Studying the Reliability of Bridge Inspection. *Public Roads*, 64 (3).

Santini-Bell, E., Sanayei, M., Brenner, B., Sipple, J. & Blanchard, A. (2008). Nondestructive testing for design verification of Boston’s Central Artery underpinning frames and connections. *Bridge Structures—Assessment, Design and Construction*, 4 (2), 87–98.

Thompson, P.D., Duray, J.A., Campbell, J.J., Puckett, J.A. & Wright, B. “VIRTIS: AASHTO’s New Bridge Load Rating System,” Transportation Research Circular 448, Eighth International Bridge Management Conference, Denver, Colorado, June 2000.

Timoshenko, S. & Young, D.H. 1935. *Elements of strength of materials,* 4th edn. New York: D. Van Nostrand.

Chapter 20

Structural health monitoring using wireless acoustic emission sensor network

T. Hay, S. Jayaraman & D.R. Hay
WavesinSolids LLC, State College, Pennsylvania, USA

ABSTRACT: Acoustic emission monitoring of steel fracture-critical members and concrete structures provides valuable input to condition-based maintenance planning and the allocation of maintenance resources. The instrumentation used for AE monitoring, however, is generally power hungry, expensive, bulky, and requires extensive cable installation. Merging acoustic emission technology with wireless technology is required to overcome some of these limitations as they relate to long term bridge monitoring. In this paper, recent work on the development and application of wireless acoustic emission technology for structural health monitoring of aging bridges is discussed. The paper discusses bridging the gap between fracture mechanics and acoustic emission information to assist in predicting crack growth rates and remaining life estimates. The practical issues associated with wireless sensor integration are also discussed in terms of power management, information management, and communication.

1 INTRODUCTION

1.1 *Background*

Acoustic Emission (AE) technique has been used widely in the last few decades for the non-destructive inspection and monitoring of fracture critical members in civil infrastructure such as bridges and pipelines (Gong et al., 1992, Boyd et al., 2001). AE is a passive non-destructive method where sensors are attached to the infrastructure of interest, and the release of strain energy through the formation and/or propagation of fatigue cracks in the form of sound waves are observed. These stress waves are produced by the sudden internal stress redistribution of the materials caused by the changes in the internal structure. Possible causes are crack initiation and growth, crack opening and closure, dislocation movement, twinning, and phase transformation in monolithic materials and fiber breakage and fiber-matrix debonding in composites. Most of the sources of AE are damage-related; thus, the detection and monitoring of these emissions are commonly used to predict material failure (Miller 2006). When load is applied to a fracture critical member with an existing fatigue crack, stress concentration around the crack tip causes extensive plastic deformation. This is followed by work hardening and eventually brittle fracture through this hardened area into a more ductile zone beyond the plastic deformation zone. The principle of the AE technique is shown in Figure 1. The growing fatigue crack generates a stress wave that travels through the member and is detected by an AE sensor that converts the mechanical displacement to an electrical voltage and is displayed using standard data acquisition hardware.

1.2 *The need*

There are an estimated 100,000 railroad bridges in the United States, and about 33,000 of them are steel bridges. According to the federal highway administration (FHWA), approximately 30% of those metal bridges are structurally deficient or functionally obsolete; hence, 10,000 railroad bridges need to be inspected immediately. Currently, the U.S. highway bridges are mostly inspected visually (FHWA 2001a). It is estimated that visual inspection only detects 3.9 percent of existing fatigue cracks (FHWA 2001b). Visual inspection does not provide any feedback on crack growth rates which is an important consideration for bridge load rating, and scheduling and budgeting of maintenance/repair operations. Most bridges that people utilize every day were built at least 50 years ago, and were not designed to withstand today's demanding traffic loads. Public (Federal and state) expenditures on infrastructure have grown by 1.7% per year from 1956 to 2004 and in recent years, have been growing even more rapidly, rising by 2.1% per year (CBO 2008). These rail and highway bridge statistics summarize the need for cost-effective bridge monitoring technologies that supply sound engineering data reliably.

1.3 *Current AE inspection practices*

Current AE measurement systems based on hardwired centralized data collection are expensive, power hungry and heavy. Today it is used for the periodic inspection of selected bridges; a process that takes days depending on bridge traffic and the number of locations to be monitored on the bridge. Figure 2 shows a typical data collection unit during an inspection. The AE system is powered by a 1000-Watt power generator, and requires long cables (30–200 ft) to reach the fracture critical bridge components. Just setting up the system takes several hours. Moreover, the expensive equipment needs to be protected against theft and vandalism.

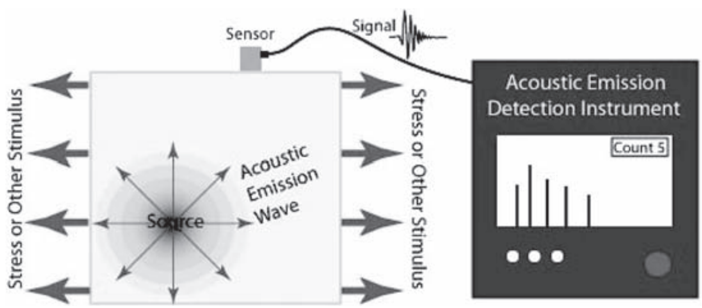


Figure 1. Principle of AE technique (Picture source: NDT resource center, www.ndt-ed.org).

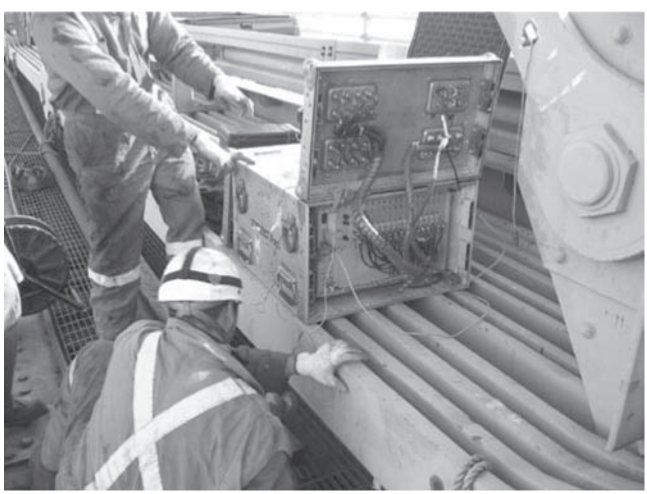


Figure 2. Currently used wired AE bridge testing practice and equipment.

2 WIRELESS ACOUSTIC EMISSION SENSING

2.1 *AE source location*

A fatigue crack generates an acoustic emission event through the rapid release of elastic energy with each step in the crack growth process. To measure such a phenomenon, an array of three or more acoustic emission sensors are placed on the fracture critical bridge member (Figure 3). Measuring the time of arrival of AE events then enables the localization of the active flaw within the array using standard Time Difference of Arrival (TDoA) techniques. Acoustic emission signal features may be used to estimate fatigue crack growth rates. Useful AE signal features include amplitude, rise-time, counts, signal duration, and energy.

2.2 *Wireless sensor node architecture*

Flash-FPGA (Field Programmable Gate Array) based solution for long term digital signal processing of the AE signals observed by the sensors has been adopted. These can operate in sleep modes and do not require huge amount of power for operation. To support the continuous operation of the node for weeks as opposed to hours on a single charge, the device needs to sleep most of the time. A very simple, application specific approach has been taken to "wake" the wireless sensor node from its "hibernation" mode. This has been achieved by including a strain gage channel. When the strain gage channel is activated (beyond a pre-set threshold) due to the weight of the oncoming train, it is then programmed to wake up the entire sensor network so that they could be ready to record and analyze AE signals that might be generated due to the active fatigue cracks. The strain data are checked by a low-power duty-cycled microcontroller and if elevated values are observed, it wakes up the rest of the board including the AE channels and the FPGA.

The prototype design is shown in Figure 4. The four AE channels can be sampled at up to 3 MHz each. However, the resolution of the selected ADC (Analog-to-Digital Converter) is 12 bits, not the traditional 16 bits. At this sampling rate, higher resolution would have meant a much more complex ADC chip making the board more complicated and expensive. Part of this ongoing research project is to determine if this lower resolution will be satisfactory for structural analysis. Data from the strain gage is used to correlate stress on the monitored structure to AE from an active fatigue crack; hence, it is connected to the FPGA. The sampling rate is a fixed 100 Hz and a 16-bit ADC is utilized. The channel has a fixed low-pass filter for anti-aliasing. The channel's

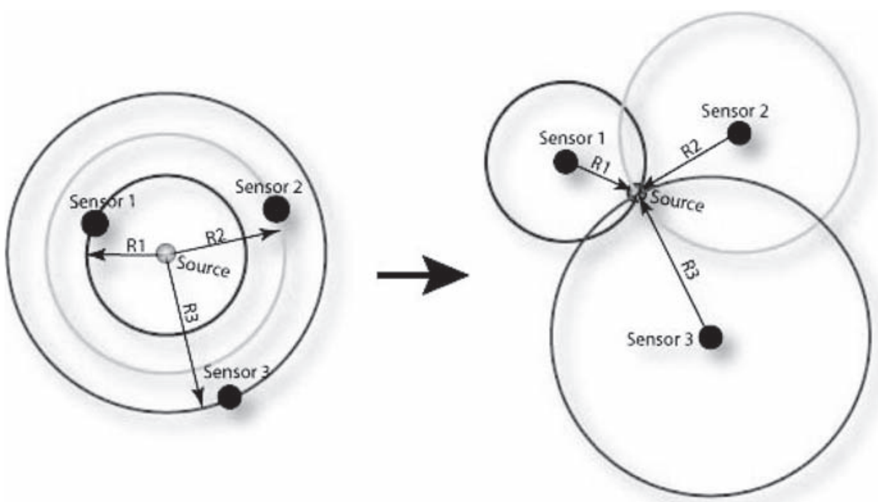


Figure 3. AE source location technique (Picture source: NDT resource center, www.ndt-ed.org).

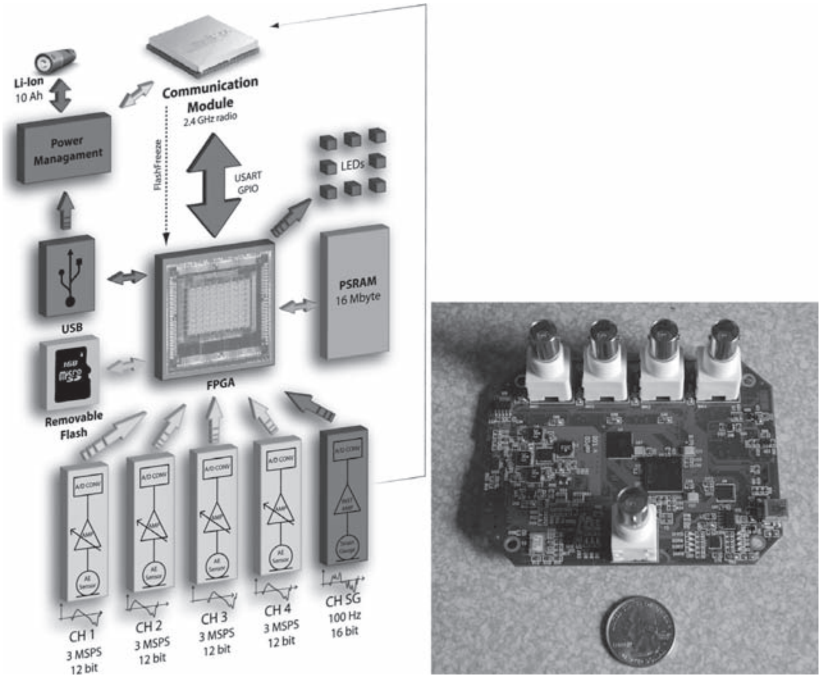


Figure 4. Design (left) and fabricated (right) wireless AE sensor node prototype.

equally important task is to wake up the board as the train approaches, so the analog signal is also connected to one of the input channels of the microcontroller on the wireless communication module. The excitation voltage level is tunable to decrease the necessary power in sleep mode. The on-board SD card and the provided USB connectivity increase the flexibility of the node tremendously. The SD (Secure Digital) card can store the measurements for a long deployment; it can be swapped out without having to download potentially large amounts of data. The USB provides fast data downloads and is used to charge the battery. It is connected through the FPGA and not the microcontroller to support the highest data rates.

The on-board SRAM (Static Random Access Memory) memory can be utilized for short-term waveform storage, for example. Low power and high intensity LEDs are used to display basic status information. A real time clock is included to correlate measurements with train schedule. The battery selected is a 10 Ah lithium ion with a small form factor. A switching regulator with high efficiency is utilized. Remaining battery capacity is monitored with a coulomb counter. A conservative estimate of 5 mA draw (in sleep mode to run strain gage channel) means that the board can sustain 2000 hours of sleep mode. It is estimated that battery life in active mode is 66 hours (at 150 mA current draw). The actual lifetime will depend on the ratio of active to passive mode, i.e. the frequency of trains passing through the bridge. For example, if the node is active 1 hour per day, the expected lifetime of the node on a single charge is about 6 weeks.

3 AE SIGNAL PROCESSING

3.1 *AE source indices*

The objective of interpretation in acoustic emission bridge inspection is to assess the significance of sources of emission. The general approach begins with filtering out non-relevant emissions. A variety of acoustic signals occurs during the monitoring process, such as noise from loose

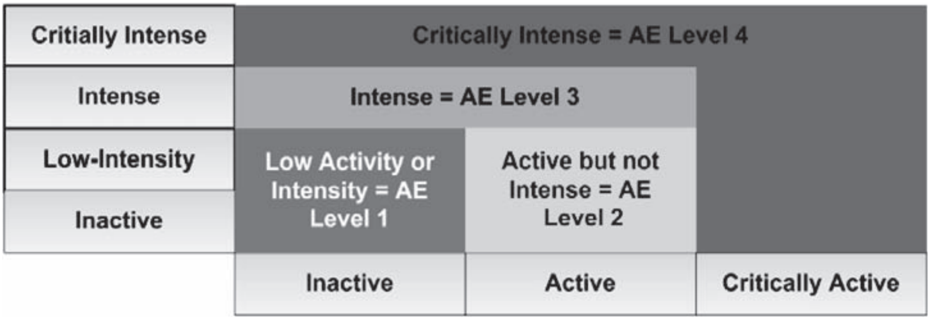


Figure 5. AE source index.

fasteners and working members, mechanical noise transferred from the track and random electrical signals. These may mask the acoustic signals from a growing crack. Since an AE event from a growing crack has a typical, characteristic waveform, signal characteristics (features) are used to identify and filter crack-related acoustic events from noise.

AE data are evaluated in terms of activity and intensity. Activity is defined in terms of acoustic events that are detected inside the sensor array by all four sensors. Intensity is defined as the average signal strength of the acoustic events in dB.

- **Activity:** The number of events that occur within the sensing array. For an acoustic source to be classified as an event, it must be picked up by all four sensors and originate from inside the array. Activity may be classified as ***Critically Active*** if events are observed consistently at peak load, ***Active*** if randomly observed over the load spectrum, and ***Inactive***.
- **Intensity:** The average amplitude, in dB, of the events. Acoustic emission may be classified as ***Low Intensity*** (< 50 dB), ***Intense*** (50–75 dB), and ***Critically Intense*** (> 75 dB).

Based on AE activity and intensity, the AE source index is developed as shown in Figure 5. Activity and intensity metrics are derived from the features of the AE signals and their dependence on the load used to stimulate AE.

3.2 *AE signal features*

Feature extraction and source localization are carried out in the FPGA. The frequency response of the AE sensors themselves provides sufficient filtering to enable a relatively simple time-domain based signal processing approach. This fits the problem well since the relevant AE signal features are all time-domain parameters. The configurable hardware implementation enables signal processing in a streaming manner at the sampling frequency; each sample is fully processed before the next one arrives. That is, there is no need to buffer the signals.

The most important task of the processing core is to detect and measure the time of arrival of the acoustic events and to extract its characteristic features. Lastly, the detection logic needs to work reliably with a few parameters to minimize the calibration requirements of the system. Figure 6 shows the time series of a typical acoustic event along with the current operational parameters (letters) and the extracted features (numbers).

The detection is triggered by a simple threshold crossing condition “A”. The start of the event “1” is the time instant of this threshold crossing. The end of the acoustic emission is identified if the signal does not cross the threshold level for longer than a timeout parameter “B”. Between these two events, the signal is monitored and the following features are calculated:

- The number of threshold crossings “5” – this is a good estimate of the fundamental frequency of the signal,
- Maximum amplitude “4”,

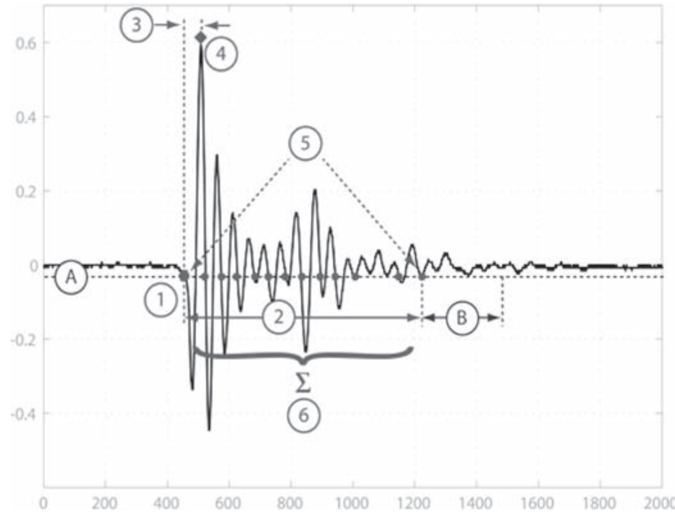


Figure 6. Signal features of an AE event.

- Rise time "3" – the elapsed time between the initial threshold crossing and the time when the signal reaches the maximum amplitude,
- Length of the event "2"– the time span between the initial and last threshold crossings,
- Energy of the signal "6" – sum of the squared sample values.

Events on multiple channels are then evaluated. If at least three channels detect AE, then the TDoA values are checked for consistency. If the difference between any two time stamps is larger than it takes the sound to travel the distance between the corresponding sensors, the event could not have come from the same source. Such inconsistent observations are discarded. If the timestamps are consistent, then the source is localized using the standard TDoA equations. If the AE did not originate from within the sensor array, it is also discarded.

4 BRIDGING THE GAP BETWEEN AE AND FRACTURE MECHANICS

Based on the Paris' Law used in Material Science, Fatigue Crack growth Rate (CGR) could be defined as,

$$CGR = da/dc = C\,(\Delta K)^m \tag{1}$$

where, C and m are experimentally determined material constants, da/dc is the crack extension (crack growth rate) per load cycle due to fatigue and ΔK is the range of stress-intensity factor arising from cyclic loading. The C and m material constants depend on material, environment, temperature and cyclic rate and crack extension increases nonlinearly with increased crack length. It has been shown (Sinclair et al., 1977) that the AE event rate per cycle is proportional to the crack propagation rate per cycle,

$$dN/dc \propto da/dc \tag{2}$$

In experiments and from the proportionality between events rate per cycle and crack propagation rate per cycle this study suggested a relation between the observed event count N over any cyclic interval, and the crack area A created in this interval as,

$$N = \gamma A \tag{3}$$

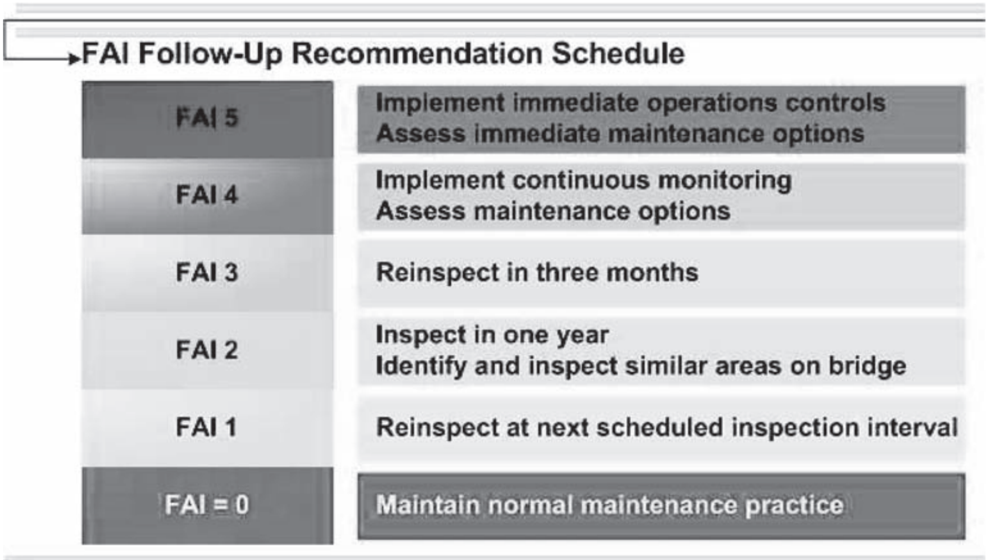


Figure 7. Follow up recommendation schedule based on fatigue assessment index (FAI).

The acoustic activity parameter γ depends on the material and test conditions, such as stress ratio, and cyclic frequency. These findings are mostly empirical and are obtained experimentally. These equations allow the determination of the fatigue life curves based on AE test data. Such curves can be derived for a material or structure and provide an assessment of fatigue damage to material containing a crack. At higher stress intensity levels, the yield of emission per unit of crack extension is higher, a consequence of the larger amounts of stored energy available at higher stress intensities. This provides a basis to link acoustic emission with the fracture mechanism and to establish the relationship between emission and stress intensity factor. A Fatigue Assessment Index (FAI) is then defined based on the AE source activity, intensity and the related fatigue crack. The corresponding recommended actions can be applied to each zone ranging from no action required, through various levels of follow up NDE or analysis, up to taking the structure out of service (Figure 7). These recommendations provide bridge engineers with information to plan, schedule and prioritize maintenance or replacement operations.

5 CONCLUSIONS

This paper discusses the need for a continuous inspection and monitoring system for civil infrastructures. It described the design and development of the various aspects of a wireless AE sensor network prototype for the specific application of railway bridge structural health monitoring. It provides great advantages over wired systems in that they could be employed long-term and provides intelligent interpretation of the data. It is expected that this system will be initially used for inspection before it will be widely adopted for the permanent instrumentation of bridges. The feature extraction aspect of AE signal processing and the relationship between these and the theoretical basis of fracture mechanics to quantify fatigue crack growth in fracture critical members were discussed as well. With more research, testing and concerted efforts to development and deployment of the wireless AE health monitoring product, significant progress could be made to provide continuous monitoring of steel railway bridges for the detection of fatigue cracks.

REFERENCES

Boyd, J.W.R., & Varley, J. The uses of passive measurements of acoustic emissions from chemical engineering processes, *Chemical Engineering Science*, Volume 56, Issue 5, March 2001, Pages 1749–1767.

Congressional Budget Office, "Issues and Options in Infrastructure Investment," May 2008.

Federal Highway Administration, "Reliability of Visual Bridge Inspection," Turner-Fairbank Highway Research Center, March 2001.

Federal Highway Administration, "TechBrief: Reliability of Visual Inspection for Highway Bridges", Turner-Fairbank Highway Research Center, September 2001.

Gong, Z., Nyborg, E.O., & Oommen, G. Acoustic emission monitoring of steel railroad bridges. *Materials Evaluation*, 1992; July: 883–887.

Miller, R.K. "Acoustic Emission Testing", Nondestructive testing handbook, 3rd edition, vol. 6, American Society for Non-destructive Testing, Columbus, OH, 2006.

Sinclair, A.C.E., Connors, D.C., & Formby, C.L., "Acoustic Emission Analysis during Fatigue Crack Growth in Steel", Journal of Material Science Engineering, 1977, vol. 28 p. 263.

Chapter 21

Bridge monitoring to measure corrosion rate and concrete resistivity

C.J. Moretti & S. Jerath
Civil Engineering, University of North Dakota, Grand Forks, North Dakota, USA

ABSTRACT: The purpose of the study is to know whether the permeability of concrete bridge decks will be reduced by adding mineral admixtures. Three bridge decks were monitored, one bridge was constructed with conventional Portland cement concrete, the second bridge was constructed with a concrete containing 38% replacement of Portland cement by fly ash , and the third bridge contained 35% Ground Granulated Blast Furnace Slag (GGBFS) as a partial replacement of Portland cement in its concrete. Corrosion of reinforcement and the concrete resistivity were monitored at the three bridges between 2002 and 2008 by using the Gecor 6 instrument. It is seen that the corrosion rate is the highest in the bridge containing no mineral admixture, the next high values are in the bridge containing GGBFS and the least corrosion rate existed in the bridge whose concrete contained fly ash.

1 INTRODUCTION

A major cause of deterioration of concrete bridges is corrosion of steel reinforcement caused by the penetration of moisture and chloride into the concrete mixture. Corrosion reduces the cross-sectional area of the steel, which decreases the stiffness and strength of the structure. The rust formed as the steel corrodes can also cause the concrete to crack due to expansive forces inside the concrete. In many cases the damaging chloride comes from deicing salt used for controlling snow and ice on the bridge deck. Since use of deicing salts is likely to continue, it is important that new bridges be designed to resist chloride induced corrosion. This can be accomplished by preventing chloride from reaching the reinforcement steel and by making the reinforcement resistant to corrosion. Epoxy coated reinforcing steel is widely used in bridge decks to make reinforcement resistant to corrosion, but it does not prevent penetration of moisture and chloride from the surface to the reinforcement steel. Thus, if the protective layer breaks down, the steel is vulnerable to corrosion. By lowering the permeability of concrete with no loss in strength, moisture and chloride penetration can be reduced. By controlling the flow of moisture into the pores freeze thaw cracking is also reduced.

Reinforcement steel in concrete does not corrode extensively under normal conditions due to the formation of a surface film of iron oxide. In the highly alkaline environment, pH > 12, usually present in concrete the oxide film on the surface of rebars tends to prevent further corrosion as long as the alkaline environment persists, and the rebars are said to be in passive condition. However, carbonation and chloride penetration promote the corrosion of rebars in concrete. Carbonation reduces the pH value of concrete to about 8. At this value, protective ion oxide film on rebars is no longer stable, and if the adequate supply of oxygen and moisture is present the exposed surface of rebars will corrode. Chloride ions also cause corrosion by disrupting the passive iron oxide film on the surface of reinforcing steel. The carbonation and chloride penetration can be controlled by reducing the permeability of concrete.

2 EXPERIMENTAL PROCEDURES

2.1 *Bridge deck construction*

In this study three bridges were constructed on I-29 in Fargo, North Dakota, USA in the summer of 2002. It was intended to know whether the permeability of concrete will be reduced by adding mineral admixtures, fly ash and ground granulated blast furnace slag (GGBFS) as partial replacement of Portland cement. One bridge was constructed with conventional Portland cement concrete, the second bridge was constructed with a modified concrete containing Type C fly ash, and the third bridge contained GGBFS of Grade 100 as a partial replacement of Portland cement in its concrete. Portland cement was replaced with fly ash and GGBFS by 38% and 35% by weight respectively in the two bridges. Corrosion monitoring activities were carried out at the three bridges between 2002 and 2008. The bridges were monitored to evaluate corrosion of reinforcing steel, and concrete resistivity. Instruments were embedded in the concrete bridge decks during construction to measure corrosion rate and concrete resistivity at various locations along the bridge span.

The concrete mix designs for the bridges are given in Table 1.

2.2 *Methods of monitoring corrosion in concrete*

Corrosion rate in concrete bridge decks were monitored by measuring corrosion rate in steel reinforcement and concrete resistivity. A Gecor 6 instrument was used to measure corrosion rate in the rebars and concrete resistivity four times a year. To allow the use of the Gecor 6 corrosion field test instrument, electrical contacts were attached directly to the rebar close to the points where the corrosion rate measurements were to be taken. To monitor the bridge decks, fifteen contacts were attached to each bridge at approximately equidistant intervals along the east edge of the deck. Each contact consisted of a stainless steel rod attached to a small square stainless steel plate. The rod was attached directly to the rebar and the plate was set level with the deck surface so that it could be accessed for the Gecor 6. When the contact was attached to the rebar, a clamp was used to penetrate the epoxy coating and make electrical contact with the steel reinforcing bar. A picture of a contact attached to the deck rebar is shown in Figure 1.

When a Gecor 6 corrosion measurement was taken on a bridge, an electrical lead from the Gecor 6 was attached to the contact plate on the bridge deck and another probe was placed on the concrete over the rebar connected to the contact. All of the contacts were connected to rebar that run transverse to the length of the deck. To take a measurement, the Gecor 6 probe was placed at least four inches away from the contact plate on a line with the plate and perpendicular to the jersey barrier. The probe was placed on the side of the plate away from the jersey barrier. When a measurement was taken with the Gecor 6, the diameter of the rebar was used for the corrosion rate

Table 1. Concrete mix designs for the bridges.

Bridges	17th avenue south	Texas turn	9th Avenue south
Cementitious material (lb./cy)	611	611	611
Portland cement (lb./cy)	397	379	611
Fly ash (lb./cy)	0.0	232	0.0
GGBFS (lb./cy)	214	0.0	0.0
Coarse aggregate size	No. 3	No. 3	No. 3
Max. water/cement (gals./sack)	5.41	5.00	5.00
Max. w/c ratio	0.480	0.443	0.443
Air content (%)	5.0–8.0	5.0–8.0	5.0–8.0
Max. slump (inches)	3	3	3
Design 28-day comp. strength (psi)	4000	4000	4000

Figure 1. Stainless steel contact attachment to the rebar of bridge deck.

calculation. 5/8" diameter rebars were used in the 9th Avenue South and 17th Avenue South bridge decks, and 3/4" diameter rebars were used for the Texas Turn bridge deck.

2.3 *Corrosion rate*

The Gecor 6 instrument measures the corrosion rate in the rebar using the linear polarization resistance (LPR) technique. It has been established with laboratory studies that corrosion current is linearly related to polarization resistance. The LPR measurement is done with a central reference electrode, surrounded by an external counter electrode. A guard ring and external electrode system confines the area of the rebar tested (NDT James). The Gecor 6 gives the corrosion rate (I_{corr}) measurement directly, which is a quantitative measurement of the amount of steel turning into oxide at the time of measurement. I_{corr} is defined by

$$I_{\text{corr}} = \frac{B}{R_p} \tag{1}$$

where R_p = polarization resistance, that is defined as the change in potential, as measured by the Gecor 6, divided by the applied current; and B = constant equal to 26 mV.

In general, the higher the measured I_{corr} values, the higher the corrosion rate. Gecor 6 measurements should be taken at strategic locations. Corrosion rates will vary depending on conditions such as concrete moisture content, chloride concentration, and temperature. Thus measurements should be carried out over time in order to obtain average I_{corr} values.

2.4 *Concrete resistivity*

Since corrosion is an electrochemical process, an ionic current must flow through the concrete for corrosion of the rebar to occur. The electrical resistivity of the concrete affects the ionic flow and the rate at which corrosion can occur. In general, a high concrete resistivity restricts the current flow. Conversely, a low concrete resistivity can cause an increased corrosion rate when steel rebar is not in a passive condition. Concrete resistivity is related to the moisture content, pore structure, and composition of the concrete. Concrete resistivity is also measured with the Gecor 6. Concrete resistivity is defined as

$$\textit{Resistivity} = 2 \times R \times D \tag{2}$$

where R = electric resistance of a pulse between the counter-electrode and the rebar network; and D = diameter of the counter-electrode.

3 RESULTS

I_{corr} data obtained with the Gecor 6 meter are a quantitative measure of the amount of steel turning into oxide at the time of measurement. The following broad criteria have been established to interpret corrosion information provided by the Gecor 6 (NDT James):

I_{corr} less than 0.2 μA/cm^2—passive condition; I_{corr} between 0.2 and 0.5 μA/cm^2—low corrosion rate; I_{corr} between 0.5 and 1.0 μA/cm^2—moderate corrosion rate; and I_{corr} greater than 1.0 μA/cm^2—high corrosion rate.

Measurements were taken at fifteen approximately equal spaced locations on each bridge deck. The locations are numbered 1 through 15, with 1 situated at the north end of the deck and the increasing numbers extending sequentially along the span to the north pier. The locations where the measurements were taken are close to the jersey barrier on the east side of the deck. The sampling plan specified that four I_{corr} measurements be taken at each location between August and June each year. The average I_{corr} measurements taken on the bridge decks over the five years for each location are given in the tabular form in the report (Moretti 2008). Air temperature and relative humidity were also recorded on the bridges when the measurements of I_{corr} were taken. The temperature and humidity data corresponding to the various sampling dates are also listed (Moretti 2008).

The overall corrosion rates for the rebar in the bridge decks were estimated by averaging the I_{corr} values collected from all locations on the deck for each sampling date. The overall corrosion rates are plotted in Figure 2. The trend lines on Figure 2 were generated by linear regression on the overall data from each bridge. These lines indicate how corrosion is progressing with time.

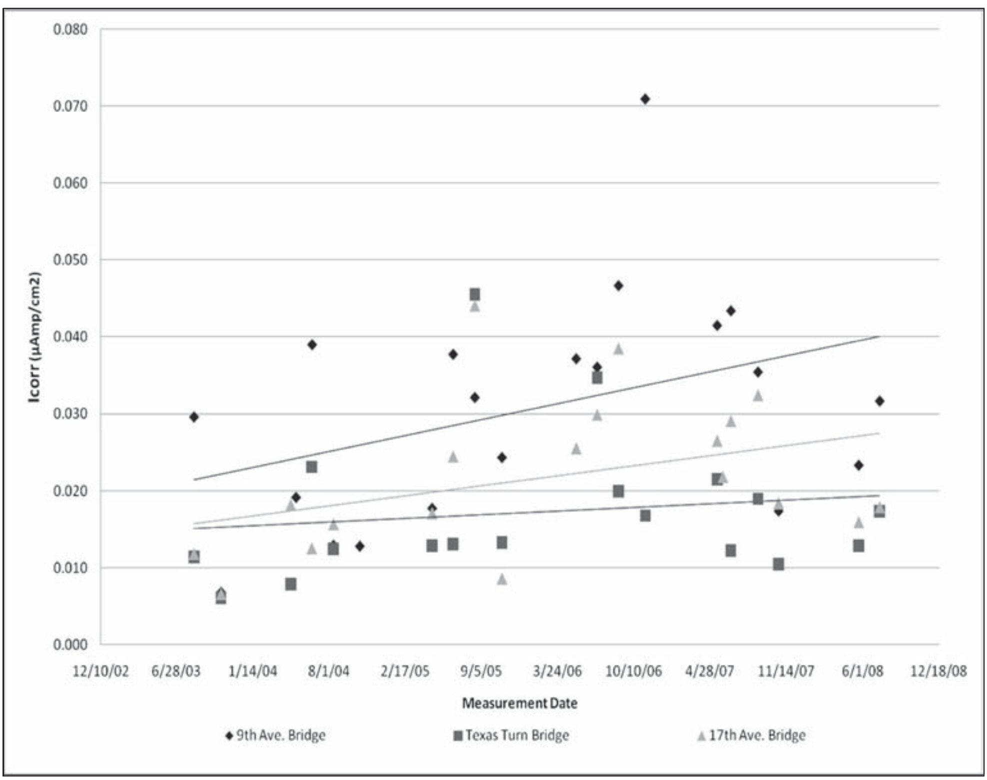


Figure 2. Average I_{corr} values on the bridges at different dates and regression lines.

The trend lines suggest that the corrosion rate is slowly increasing with time. This is expected because environmental factors which support corrosion such as moisture, oxygen, and chloride, tend to diffuse deeper into the concrete over time. All of the I_{corr} values in Figure 2 are below the 0.2 µA/cm^2. These results indicate that the rebar at all measurement locations are in a passive (i.e., non-corroding) state. This is reasonable because the decks were monitored within five years of their construction and the rebars have epoxy coating to protect them from corrosion.

An I_{corr} value can be converted to the rate of metal thickness loss using Faraday's law

$$M = \frac{I\,t\,A_w}{nF} \tag{3}$$

where M = mass of metal dissolved or converted to oxide (grams); I = current (Amps); t = time (seconds); A_w = atomic weight; n = valency; and F = Faraday's constant (96,500 coulombs/equivalent mass).

For iron in reinforcing steel, a factor of 0.492 can be used to convert I_{corr} values to thickness loss in units of mils/yr (ACI Committee 222). Rates of steel loss for the rebar were calculated based on overall I_{corr} values averaged over each year of the project. These values are compared in Figure 3.

The concrete resistivity information is interpreted as follows:

Resistivity > 100 kΩ cm—very low corrosion rate even with high chloride concentration or carbonation; 50 < resistivity < 100 kΩ cm—low corrosion rate; 10 < resistivity < 50 kΩ cm—moderate to high corrosion rate when steel is not in passive condition; resistivity < 10 kΩ cm—resistivity is not the controlling parameter of the corrosion process.

The resistivity measurements were taken at the same time and the same locations as the I_{corr} measurements. The resistivity measured on the three bridge decks varied widely. Some of the

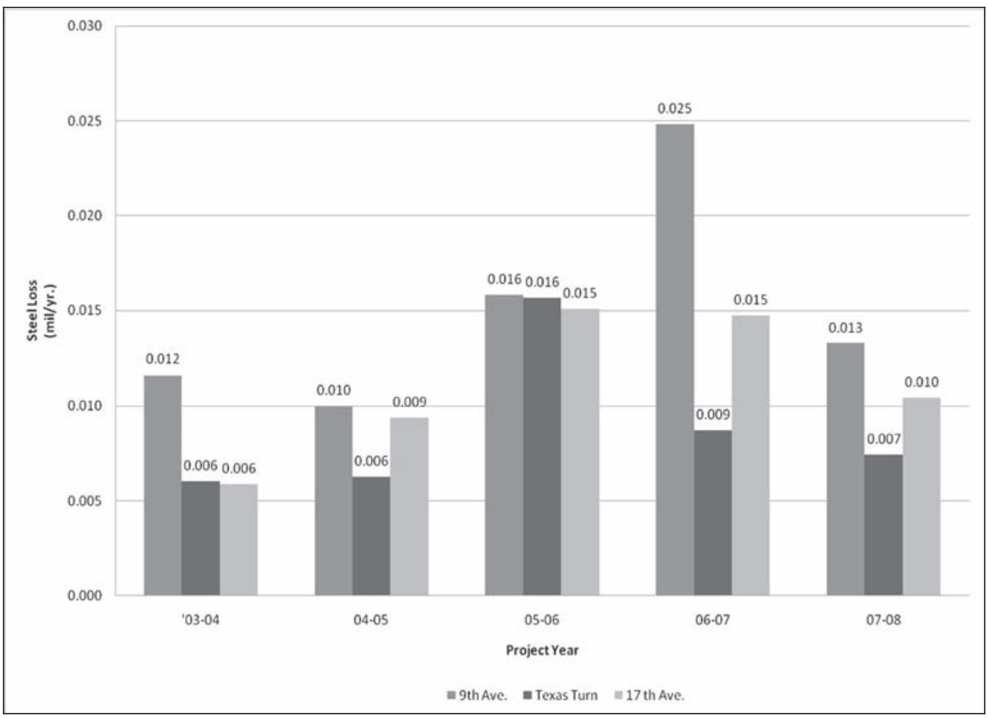


Figure 3. Estimated rate of steel loss in rebars during different years.

values were in the range from 10 to 50 kΩ cm, which indicates moderate to high corrosion rate when the steel is not in a passive condition.

4 CONCLUSIONS

All the three bridges were measured to have I_{corr} values lower than the 0.2 μA/cm^2, indicating that all the rebars were in the passive (non-corroding) condition as of July 2008, six years after their construction at the locations tested on the three bridge decks. The bridge containing no mineral admixture produced the highest I_{corr} readings, the bridge containing GGBFS grade 100 had the next high values, and the bridge containing class C fly ash exhibited the lowest corrosion rate. Some of the concrete resistivity values are in the range from 10 to 50 kΩ cm, but these measurements do not indicate that the rebars are in danger of corroding, because the corresponding I_{corr} measurements showed that all the rebars tested were in passive condition.

Even though the trend lines in Figure 2 indicate a gradual increase in I_{corr} values over the first four years of the project, there is a trend that the I_{corr} values begin to decrease in the fifth year. This reduction may be due to less permeability of concrete with time because it becomes denser. It is recommended to continue the monitoring in future years to know how effective are the mineral admixtures in preventing the corrosion of reinforcing bars in the bridge decks.

REFERENCES

ACI Committee 222. 2001. Protection of metals in concrete against corrosion. *ACI 222R-01*. American Concrete Institute, Farmington Hills, MI.

Moretti, C.J. 2008. Bridge monitoring project—Final report. North Dakota Department of Transportation, Bismarck, ND.

Rebar corrosion and its effects. NDT James Instruments Inc. (www.ndtjames.com).

6 Bridge security and testing

Chapter 22

The security assessment of structural cable assemblies in bridges

T.W. Klein
WireCo Worldgroup, Kansas City, Missouri, USA

ABSTRACT: The ability to provide a secure environment in the bridge community today is a daunting task being undertaken by many engineers worldwide. One of the key structural components identified in long span bridges is the steel cable assembly. The paper focuses on identifying the vulnerability of the cable assembly in the event of an accident, disaster, or attack. The paper describes the response of traditional and experimental cable assemblies to moderate and extreme heat. The results from high temperature testing performed on mock cable assemblies under load in controlled test environments will be discussed in this paper. The paper also looks at new practices being developed in the wire rope industry to increase the security and functionality of cable assemblies currently used in vertical lift bridges. New high performance materials have been shown to drastically increase the functional operating temperature of the cable assemblies without affecting the mechanical properties of the wire rope. The paper will also discuss the theoretical and experimental work regarding the operating mechanisms of wire rope held in conventional tapered sockets.

1 INTRODUCTION

Vehicular accidents involving volatile materials are considered dangerous but when mixed with bridges they may they could become devastating. One such accident that took place on August 15, 2003 had this effect on the surrounding community. The passenger side of a tractor trailer truck traveling over a traditional cable supported arch structure hit a bridge abutment, rupturing the gas tank and ripping open the side of the trailer. The gasoline from the tank caught fire, as did the 6,800 kilograms of garlic powder in the trailer when both spilled onto the bridge deck. It was estimated that the fire burned at temperatures reaching 850°C [1,500°F] for several hours. The effects of the heat took its toll on various areas of the bridges raising the concerns of many people in the bridge engineering community. Fortunately no one was injured in the accident. It was also fortunate that the fire did not occur near the cables supporting the structure.

The ability of a cable supported structure to withstand damage during a high intensity fire or blast contains a multitude of variables. It is estimated that a fuel fire could burn in excess of 1,095°C [2000°F]. This paper focuses on the ability of the structural cable assembly to operate in high-temperature environments. To substantially increase the operating temperature of the cable assembly the socketing medium of the cable needed to be evaluated and tested. Additional options include fire protection methods for the cable assemblies that are capable of resisting and deflecting blast impacts, as well as deflect heat away from critical areas of the assemblies.

2 CABLE ASSEMBLY DESCRIPTION

The cable assemblies on bridges work as a tension member transferring the load of the roadway deck to overhead members. The connection of the cable assembly requires a termination socket be permanently attached to the end of the cables. This is typically done by speltering open cavity sockets on to the cable with socketing mediums.

The speltering process requires the individual wires of the cable be separated into a broom configuration as shown in Figure 1. The broomed wires are cleaned and inserted into the cone of

Figure 1. Individual wires in broom configuration.

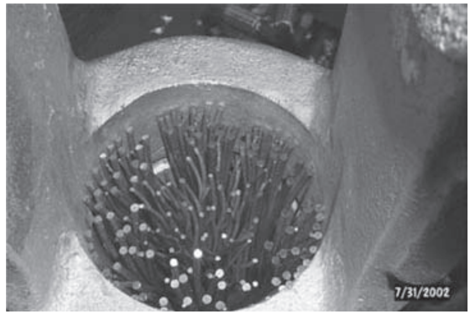


Figure 2. Broomed wires in termination cone.

the termination as shown in Figure 2. The socketing medium is cast in to the cavity and allowed to solidify forming a solid composite of socketing medium and wires. The most widely used socketing medium material is ASTM B6 High Grade Zinc. It is an inexpensive product that provides an efficient end termination while protecting the cable from corrosion. Thermoset resins are also available as socketing mediums. The completed assemblies are proof-loaded to seat the wedge and prove the bond between the wires and the socketing medium.

2.1 *Socketing mediums*

The temperature resistance of cable assemblies produced with traditional methods will be limited by the melting temperatures of the socketing mediums. Zinc has a melting temperature of approximately 315°C [600°F] and Resin will begin to lose mechanical properties at approximately 121°C [250°F]. The need for an alternative socketing medium for cable supported structures is becoming prevalent in the market today due to increased security needs. New socketing mediums have been developed that will increase the working temperature of the socketed connection to above 1,371°C [2,500°F]. At these temperatures the weak link of the assembly will actually be the steel itself. High Temperature socketing mediums ensure the safety of the suspended structure in the event of a moderate fire or accident on the roadway that may directly affect the cable assemblies.

2.2 *Analysis of the socketed connection*

Cables are linked by shape and force to the termination due to the conical shape which the medium forms with the steel wires. The composite material formed from the wires and socketing medium needs to be an efficient connection. An evaluation of the working properties of the conical shape of the termination as well as the medium used for socketing was fully investigated to determine the stresses developed on the socket walls. Socketing mediums are required to adhere to the carbon steel wires as well as provide the compression strength needed for transmittal of the force through the cable end terminations.

The first step in the assessment of an alternative socketing material required an evaluation of the stresses of the socket. A mechanical model of a wire rope termination is shown in Figure 3. In this model the individual wires are not shown dispersed in the cone since the evaluation is focused on the socketing medium.

The equilibrium of forces allow the following Equation 1 to be formulated,

$$W = Nf(\sin a + f \sin a) \tag{1}$$

when $a = 0$, $W = Nf$, representing only the frictional effect of the joint when assembled with sufficient compression between the wire matrix and the socket wall. In reality the external load has to overcome additional adhesion forces.

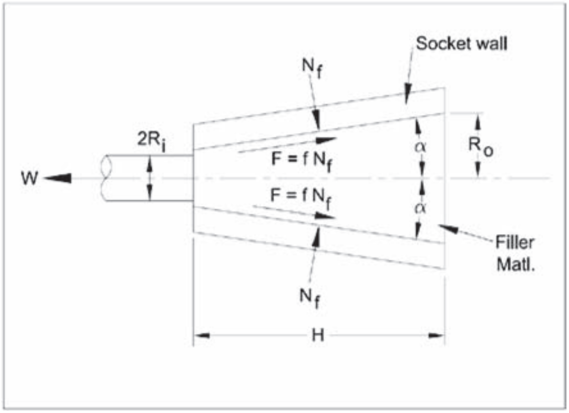


Figure 3. Socket body free diagram.

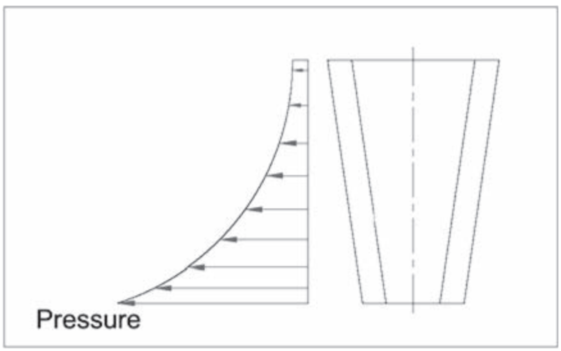


Figure 4. Distribution curve.

The contact pressure, P, from the socket will theoretically be dispersed throughout the socketing along the wall in contact with the socketing medium. The geometry of the conical socket depicted in Figure 1 is shown in Equation 2 below.

$$A = p(Ro + Ri)\ [(Ro + Ri)^2 + H^2]^{(1/2)} \tag{2}$$

Postulating that $P = Nf/A$ the contact pressure may now be used in Equation 3 to estimate the typical stresses applied to the socketing medium.

$$P = W/(p(Ro + Ri)(Ro - Ri + fH)) \tag{3}$$

In the calculations above the cone represents an infinite stiff object applying an equally distributed pressure load on the interior socket wall. Studies performed on sockets have shown the pressure loading, P, in the socket is not equally distributed throughout the socket body. At the base of the socket the internal pressure may be up to four times greater than initially estimated. Figure 4 shows the distribution curve of the forces applied to the interior of the socket.

Using the minimum breaking forces specified for ASTM A603 Structural Bridge Rope the theoretical average pressure the socketing medium will need to withstand is approximately 62,000 kPa [9,000 psi]. This value does not take in to account the extreme pressures at the base of the socket. As in all cases the reliable magnitude of the equations can only be established experimentally.

3 TESTING

3.1 *Alternative material selection*

A Composite Material, CM, was selected for testing as an alternative socketing material. The major components and mechanical properties of the CM material are specified below.

Mechanical Properties at temperatures up to 1372°C [2,500°F]
Compressive Strength – 107,000 kPa [15,500 psi]
Modulus of Rupture – 8,000 kPa [1,160 psi]

The initial destructive testing conducted with the CM socketing medium was successful providing 100% efficient end terminations. Destructive testing was performed on the 10th day after pouring. This ensured all samples had adequate time to cure thoroughly. The samples failed clear of the end terminations. This type of failure is critical to ensure the socketing medium did not affect the efficiency of the test sample. No signs of slippage or disruption were detected in the socketing medium.

3.2 *High temperature testing*

A total of seven tests were performed at two target temperatures to determine the mechanical effects of the three socketing mediums under tension. Table 1 below shows the Sample Number, Socketing Medium, Strand Diameter, and Tension Load during test. Each sample was socketed with the corresponding medium and exposed to an elevated target temperature environment while under a tension load equaling 20% of the cable assembly minimum breaking force.

The test schematic as shown in Figure 5 consisted of a single sample enclosed in an insulated oven. Inside the oven the test socket was pinned to a fixed steel connection plate. The opposite end of the sample was attached to a load cell which connected to a hydraulic ram keeping the specified

Table 1. Test matrix.

	Socketing	Strand diameter		Tension load	
Sample	Medium	mm	inches	kg	lbs
1	CM	38.1	1 1/2	25,402	56,000
2	CM	38.1	1 1/2	25,402	56,000
3	CM	38.1	1 1/2	25,402	56,000
4	Zinc	38.1	1 1/2	25,402	56,000
5	Resin	38.1	1 1/2	25,402	56,000
6	CM	44.5	1 3/4	34,743	76,000
7	CM	44.5	1 3/4	34,743	76,000

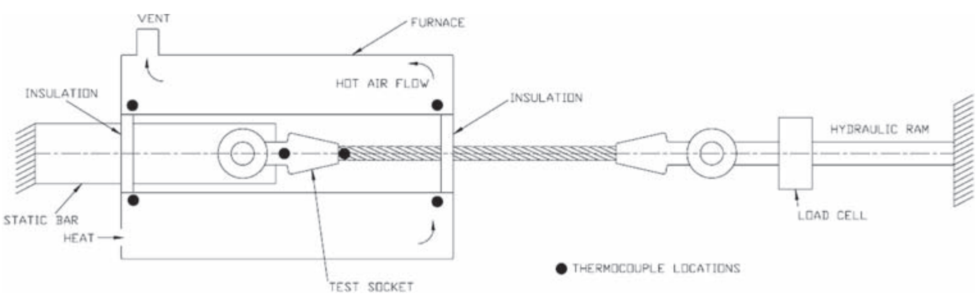


Figure 5. Testing schematic.

load constant during the testing. Propane was used to fuel the burner on the oven. Once the target temperature was reached a smaller nozzle was attached and the control mode was initiated to hold the temperature constant. The nozzle switch point will be evident in the time-temperature curves. Four thermocouples were inside the oven to accurately monitor the oven chamber.

The first samples to be tested were socketed with the CM socketing material. The strand for both samples was 38.1 mm [1–1/2 in] diameter ASTM A586 Grade 1 galvanized strand. The target temperature for samples #1 and #2 was 538°C [1,000°F]. A static tension of 25,400 kg [56,000 lbs] was applied to the samples during the heating.

Sample #1 was prepared with a 3 mm diameter well tube inserted into the socketing material to allow the placement of a thermocouple into the socket basket. The second thermocouple was attached to the inside of the socket ear, see Figures 6 and 7.

The results for test #1 are shown below. The total test time inside the oven was 160 minutes at an average oven temperature of 530°C [987°F]. The socket temperature and well temperature came close to the oven temperature due to the time exposed in the oven. The test was completed at 160 minutes after the strand failed 4″ from the base of the socket as shown in Figure #2. An examination of the socket showed no movement of the wires in the socket basket. The strand failure was due to the annealing of the wires. The socketed assembly was exposed to temperatures in excess of 482°C [900°F] for 150 minutes prior to failure.

Sample #2 had one thermocouple attached to the inside ear of the socket with the second thermocouple attached to the galvanized strand 4″ from the base of the socket. The remaining test samples were monitored using this thermocouple placement procedure. The test time inside the oven for test #2 was 120 minutes at an average oven temperature of 532°C [990°F]. The test was terminated without the strand failing. The sample was removed and examined. No movement had occurred in the socket basket. Test sample #2 was then tensile tested producing an ultimate tensile strength of 108,400 kg [238,970 lbs]. This is approximately 75% of the nominal catalog strength of 144,200 kg [318,000 lbs]. The tensile strength of the structural strand was reduced due to the heat exposure. Most importantly the failure occurred in the middle of the sample indicating the ability of the socketed connection to develop the strength of the structural strand after the exposure at temperatures is excess of 482°C [900°F] for 100 minutes.

Tests #3 and #4 were performed with traditional socketing material of zinc and resin respectively. The strand for both samples was 38.1 mm [1–1/2 in] diameter ASTM A586 Grade 1 galvanized strand. The target temperature for samples #3 and #4 remained at 538°C [1,000°F]. Sample #3 was socketed with zinc material and failed after 47 minutes of exposure never reaching the desired set point temperature. The zinc partially melted from the socket basket losing the mechanical properties required to maintain the load ultimately releasing the strand. Figures 8 and 9 below show the zinc socket sample after failure.

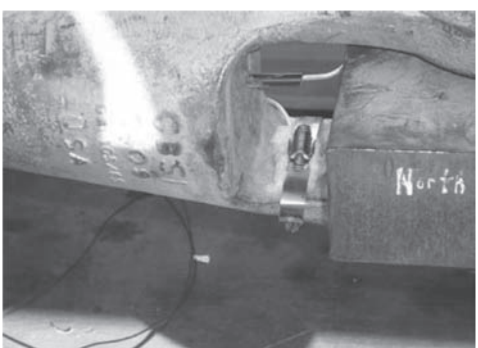


Figure 6. Test #1.

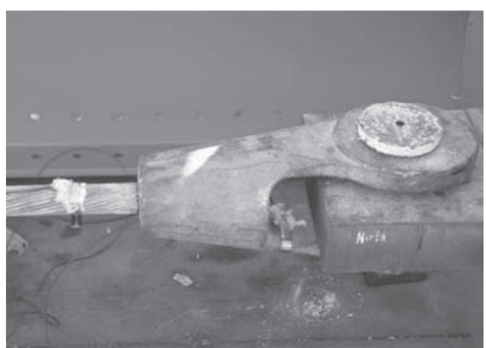


Figure 7. Test #1.

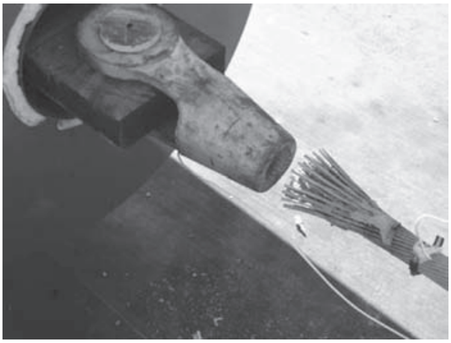


Figure 8. Zinc test #3.

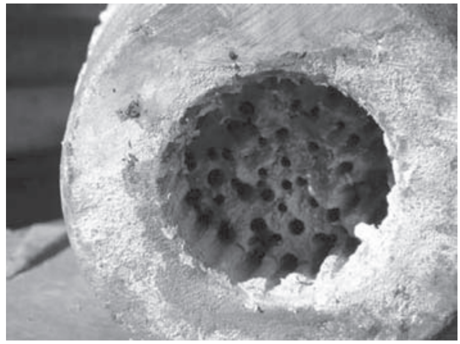


Figure 9. Zinc test #3.

Figure 10. Resin test #4.

Figure 11. Resin test # 4.

Sample #4 was socketed with resin material and failed after 27 minutes of exposure without reaching the 538°C [1,000°F] target. The resin material combusted and began to burn after 15 minutes of exposure in the oven. At the time of failure the socket had only reached 272°C [521.6°F].

Figures 10 and 11 show the resin sample after removal from the oven. The strand had slipped approximately 5" from the socket. The resin lost all mechanical abilities turning the socket medium into a fine particle.

The graph below shows the Time-Temperature curves for test 3 and 4. Neither sample reached the target temperature of 538°C [1000°F] prior to failure. Both tests resulted in failure due to the socketing material weakness. The temperature on the graph below is again the average oven temperature. The temperature of the sockets at the time of failure was significantly less.

Additional testing of the CM socketing material was conducted with tests #5, #6, and #7. The strand for test samples #5 and #6 was increased to 44.5 mm [1–3/4 in] diameter ASTM A586 Grade 1 galvanized strand. The target temperature for samples #5 and #6 was 538°C [1,000°F] and 649°C [1,200°F] respectively. A static load a 34,743 kg [76,000 lbs] was applied to the strand samples during the heating cycle.

Test #5 was heated in the oven for a total of 100 minutes before terminating the test. More importantly the sample was above 482°C [900°F] for more than 60 minutes during the testing. The sample was removed and inspected. No movement or slippage was measured in the socket basket. The sample was tensile tested with an ultimate tensile strength of 147,597 kg [325,395 lbs]. This is approximately 75% of the nominal catalog strength of 195,952 kg [432,000 lbs]. The tensile strength of the structural strand was again reduced due to the heat exposure. The failure occurred approximately 102 mm [4"] from the socket base, see Figure 12. This indicates

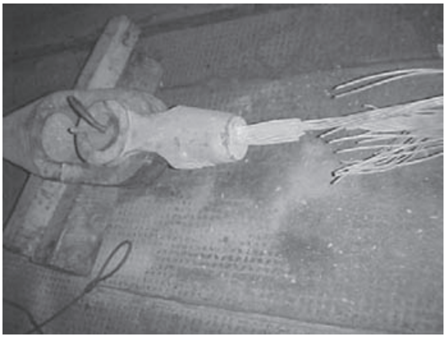


Figure 12. Tensile test of sample #5.

Figure 13. Test #6.

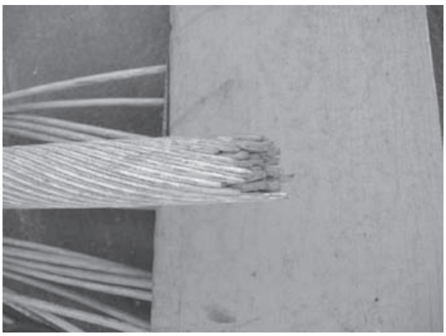


Figure 14. Test #6.

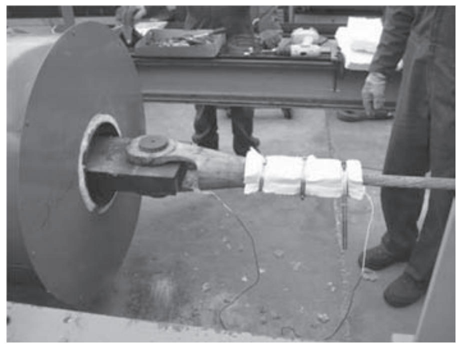


Figure 15. Test #7.

the ability of the socketed connection to develop the strength of the structural strand after the exposure at temperatures is excess of 482°C [900°F] for 60 minutes,

Test #6 increased the target temperature to 649°C [1,200°F] for the sample. After 73 minutes in the oven the strand failed 102 mm [4"] from the base of the socket prior to reaching the target temperature. The oven was not capable of heating the sample quickly causing the strand to anneal prior to reaching the target temperature. Figures 13 and 14 below show the failed sample. The high temperature exposure resulted in necking and ultimate failure of the strand.

Due to the previous results for test #6 the strand in test #7 was insulated to deflect the heat away from the strand. 26 mm [1"] insulation was wrapped around the strand for approximately 305 mm [12"] from the base of the socket as shown in Figure 12. The socket body remained exposed to the heated environment. The strand for test #7 was 38.1 mm [1–1/2 in] diameter ASTM A586 Grade 1 galvanized strand. The target temperature for sample was 649°C [1,200°F]. A static tension of 25,400 kg [56,000 lbs] was applied to the sample during the heating.

The results of test #7 showed the materials ability to withstand temperatures up to 649°C [1,200°F] without losing the mechanical properties required to sustain loading of the sample (Figure 15). The total exposure time of the sample was 135 minutes. The total exposure time of the sample in range of the target temperature was approximately 90 minutes. The temperature of the socket body was 569°C [1,056°F] at the time of failure. At 135 minutes the strand failed at the base of the socket. As before the strand failed due to annealing of the wires. Figures 16 and 17 show the necked condition of the wires at the failed connection. A close examination of the socket showed no movement or distortion in the socketing medium up to the time of failure. An examination of the face of the CM socketing medium showed no cracks or distortion.

Figure 16. Test #7.

Figure 17. Test #7.

Table 2. Test results.

		Tension load		Avg. oven temp.		Total time in oven
Sample number	Description	kg	lbs	°C	°F	Minutes
1	CM	25,402	56,000	530.9	987.6	160
2	CM	25,402	56,000	534.2	993.5	120
3	CM	25,402	56,000	638.4	1181.1	135
4	Zinc	25,402	56,000	406.9	764.4	48
5	Resin	25,402	56,000	395.9	744.7	27
6	CM	34,743	76,000	501.3	934.3	100
7	CM	34,743	76,000	375.9	708.7	135

4 CONCLUSIONS

The testing as a whole was successful in answering many questions regarding the effectiveness of traditional socketing mediums when exposed to elevated temperatures while under a load equaling 20% of the structural strand minimum breaking force. Table 2 below shows the average oven temperatures of the samples and the total exposure time of the samples in the oven. The CM socketing medium was successful in sustaining the test load for an extended period of time while exposed to the target temperatures set forth in the testing parameters. The final test #7 showed the CM socketing medium to be effective at 649°C [1,200°F] for up to 90 minutes. The traditional socketing materials, zinc and resin, were shown to have very poor characteristics in elevated temperatures. Neither zinc nor resin socketing mediums could provide the mechanical properties at temperatures near 538°C [1,000°F]. Both samples failed without reaching the target temperatures specified in the test. Tests #3 and #4 were not successful in reaching equilibrium with the oven temperature nor did they reach the target temperatures of the test due to the short test time.

The tests that have been conducted thus far have shown CM socketing materials to be efficient in high temperature environments. In each of the five tests conducted with CM socketing material no movement was detected in the socketed connection despite failures occurring in the component body of the structural strand. The material remained solid and retained all mechanical properties despite the exposure time to the target temperatures. The material has also be through several testing cycles at ambient temperature and proven to be effective in terminating the cable with 100% efficiency. CM socketing materials have shown to be an efficient and future replacement for the traditional materials currently being used in the market place today.

REFERENCES

ASTM A586-04a: Zinc Coated Parallel and Helical Steel Wire Structural Strand.
ASTM A603-98(2003): Zinc Coated Steel Structural Wire Rope.
ASTM B6-08: Standard Specification for Zinc.

Chapter 23

Overview of available detection technologies with applications to bridge security

J. Godoy
eClic International Advisory Board, New York, New York, USA

J. Chang
Columbia University, New York, New York, USA

ABSTRACT: Perimeter Intrusion Detection Systems (PIDS) can be deployed as a physical security tool at bridges. If PIDS is deployed as part of an overall multi-stage physical security system, the holistic approach to security will position owners and operators to pro-actively detect unauthorized intrusions, deter attacks, deny access and defend the facility.

1 INTRODUCTION

Protection of the nation's civil infrastructure has been a principal concern since the events of September 11, 2001. Strategies to achieve this end range from deflection and dispersal techniques to the use of physical barriers and structural hardening. Since detection technologies can be installed at lower cost and with relative ease when compared to some alternatives, they have a useful role to play in critical infrastructure protection. These technologies have been deployed as part of a broader physical security system at facilities such as ports, chemical facilities, power plants, and un-manned border crossings. This paper provides an overview of available detection technologies, including video surveillance, video analytics, and fence sensors. Issues to consider include life-cycle cost, maintenance needs, and installation techniques. One foresees potential applications of these technologies to bridge security; one example includes protection of the anchorages of suspension bridge structures. While not a cure-all, detection technologies are an important part of any comprehensive strategy to protect the nation's infrastructure.

2 PUBLIC POLICY ON CRITICAL INFRASTRUCTURE

2.1 *Directives, orders, acts and laws*

In the 1990s the United States was the target to a series of terrorist attacks both domestically and abroad. These included the World Trade Center bombing in 1993, numerous attacks on US Embassies abroad, the bombing of the Federal Building in Oklahoma City in 1995 and the attack on the USS Cole in 2000. As a direct consequence to the rising terrorist activities and threats, the tone and tenor on "infrastructure" in public policy shifted to concerns of terrorist attacks and how these can affect critical support functions vital to the interest of the nation. Public policy focused on the physical security of critical infrastructure started in the Clinton Administration and flourished under the Bush-43 Administration fueled by the attacks of 9/11.

The George W. Bush Administration issued over 10 policy documents, Presidential Directives, Acts and Strategy Plans that defined and dealt with different aspects of critical infrastructure and key resources. These policies have evolved over time with regards to what is considered critical infrastructure and what agencies have lead responsibility for their oversight and protection.

National policies are implemented and enforced at the state, local and private levels via funding vehicles provided by the federal government. The state of flux associated with the refinement and evolution of national policy posed a non-trivial challenge to stakeholders at all levels involved in the protection of critical infrastructure. (Moteff & Parfomak, 2004).

Over the 2 terms period, as the Bush Administration refined the national policy on critical infrastructure protection, federal grant funds were earmarked on a yearly basis for those projects that closely aligned with the prevailing public policy of the day. The dynamic public policy environment presented critical infrastructure operators and owners with a challenging dilemma. They were chasing a target that readjusted itself when requesting federal and state grant funds to comply with national mandates. It also poses the question of how cost effective and prudent the expenditures made on critical infrastructure protection were, based on the constant changes of national goals and objectives.

2.2 *Blue ribbon panel*

In the aftermath of 9/11 and the on-going threat of terrorists attacking assets with substantial economic consequences, the American Association of State Highway and Transportation Officials (AASHTP) Transportation Security Task Force and the Federal Highway Administration (FHWA) convened a Blue Ribbon Panel (The Panel). The Panel included bridge and tunnel experts from academia, public and private sectors. The objectives of the Panel were "to examine bridge and tunnel security and develop strategies and practices for deterring, disrupting and mitigating potential attacks." (AASHTO, 2003). The Panel issued the "Recommendations for Bridge and Tunnel Security" in September of 2003. In the Report it acknowledged the vulnerability of the nation's bridges and tunnels to terrorist attacks and made recommendations "to reduce the probability of catastrophic structural damage that could result in substantial human casualties, economic losses and socio-political damage."

One of the many recommendations issued by the Panel was a process for prioritizing bridges and tunnels with respect to their vulnerability and criticality. They suggested the development of a framework for evaluating alternatives for mitigating attacks. The Panel stated that if a given critical asset was at risk, it must be made more secure by implementing countermeasures "to deter attacks, deny access, detect presence and defend the facility."

The Panel issued the "Guidance to Highway Infrastructure owners and operators" where it stated that threat of an attack must be mitigated when the consequences were deemed unacceptable. Threat mitigation guidelines included: "the establishment of secure perimeter using physical barriers; Inspection surveillance, detection and enforcement, CCTV; Visible security presence; Minimize time on target".

3 STRATEGIC SECURITY PLANNING

The first step in deploying a physical security system is to conduct an assessment and planning exercise. There are many factors to consider when conducting a threat assessment on a critical infrastructure asset.

A sound physical security strategy should embrace a holistic approach. The strategy should include an exhaustive threat assessment followed by an extensive risk mitigation planning phase. The most comprehensive physical security deployments are the results of careful planning where a collection of multimodal technologies, systems, processes and procedures are integrated to form an end-to-end system of systems. A technique to physically secure a location or asset is to deter intruders from entering areas that immediately surround the location of interest. This would include protecting the perimeter surrounding the location or asset of interest with physical barriers to prevent unauthorized intrusions. A security system should also provide alerts of attempts or successful intrusions; these types of systems are known as perimeter intrusion detection systems (PIDS). In addition, physical barriers can be deployed in concentric levels and stages to increase the deterrence and/or delay advancement of intruders. It follows that intrusion detection systems can also be deployed in stages as well. The deployment of detection system can provide valuable information to security operatives so these can react in a reasonable time frame and if required respond appropriately to counter-act intrusions or other security threats.

When planning the installation of a PIDS careful consideration should be given to the environmental conditions of the surrounding areas and how these can affect the performance of the different sensors. These environmental factors should include lighting, climate, temperature, terrain, line of sight, seasonal changes, wind, humidity, access to power sources and network facilities.

The performance parameters of interest in the design of a PIDS are the minimum probability of detection (P_d) and a specified level of confidence. The target goal of these two parameters should be as close to 100% as possible for optimal performance. The other performance parameters to be considered for the PIDS are the false alarm rate (FAR) and Nuisance Alarm Rate (NAR). The target goals of these two parameters should be as close to zero as possible for well-designed systems. These parameters measure the overall performance of a PIDS and the calculations are influenced by many factors such as system components, installation issues, environmental conditions, configuration parameters, vegetation, animal life, surrounding structures and operating procedures to name a few.

No one sensor will detect the different kinds of intrusions with a high level of reliability and a low level of false alarms rates. Sensors are typically designed to detect certain types of intrusions. All well performing PIDS will have the adequate type of sensors installed appropriately given the threats, environment, surroundings, operational infrastructure and response capabilities.

4 TECHNOLOGIES USED IN THE PRACTICE OF PHYSICAL SECURITY

As mentioned before sensors are designed to detect specific types of intrusions or intruders. This section will explore at a cursory level the underlying technologies and systems employed in some of the sensors used in a perimeter intrusion detection system.

4.1 *Fence sensors*

There are a variety of sensors that can be used in conjunction with barrier, fences and walls. Fence sensors are popular because they offer the opportunity to detect an intruder at the outer perimeter of a facility, most likely in the early stages of an intrusion. Fence sensors while employing different technologies are designed to detect disturbance, motion or vibration on or surrounding the fence. The disturbances are picked up by the sensors employing a variety of techniques such as the opening and closing of contact switches, measurable displacements or sensing differences in energy fields. Some of these sensor controls can be calibrated to distinguish between environmental disturbances and actual intrusions by monitoring the frequency, intensity and duration of the detectable disturbances. The performance of the fence sensors can be improved by deploying additional complimentary equipment such as local weather stations and other optics based sensors such as cameras to verify the nature of the disturbance.

4.2 *Infrared sensors*

4.2.1 *Passive Infrared (PIR) sensors*

PIR sensors are designed to detect infrared heat emissions from living things. These sensors are used to detect motion in its field of view. These sensors are considered passive because they don’t emit any energy sources during the detection process. The infrared emissions are detected by a lens or mirror and the information is then transferred to electronic circuitry that records changes. As the intruder moves across the field of view the sensor tracks the changes in the detected “hot spot” and will trigger an alarm if certain thresholds are exceeded.

4.2.2 *Active Infrared (AI) sensors*

AI sensors consist of transmitter and receivers or photo detectors and specific lenses. Transmitters are aligned with photo detectors or receivers. The receivers are programmed to generate alerts if they stop sensing the beams or signals generated by the transmitters. In more sophisticated systems multiple transmitters and receivers are arranged to form an "infrared fence". Optical techniques are employed to reduced interference from other light sources. Processing each interrupted beam can generate a profile of the intruder being detected as it crosses the "infrared fence".

4.3 *RADAR*

Radar stands for Radio Detection and Ranging and is categorized as monostatic microwave sensors since the transmitting and receiving antenna of microwave signals are co-located. Radar works by transmitting microwave signals that are reflected by the target and detected by the receiver. Radar can detect intruders based on the frequency shift between the transmitted and received signals. A phenomenon called the Doppler shift occurs in electromagnetic waves when the intruder moves towards or away from the antenna. Radar can only pick up the radial vector of the intruder's motion in relation of the radar signal. Radar antennas sweep in a circular motion to collect the reflected radial components across a 360-degree arc that allows the display of a more comprehensive picture of its surroundings. These types of radars can detect the shape, speed, range and direction of an intruder. Radar can be a very effective detection tool in certain environments but could be less productive in other surroundings where it could generate alarms that are of no interest to the security operator. Examples of clutter can be the reflection of water surfaces, trees in a forest, rain or snow and other atmospheric or man-made disturbances.

4.4 *Electro-optical sensors*

These sensors convert light rays into electrical signals such as the charge-coupled devices (CCD) used in digital video cameras. Other devices in this category include thermal image sensors that can operate in low light conditions and detect heat emissions from objects to create images of thermal signatures in its field of view.

4.5 *Video based sensors*

4.5.1 *Video cameras*

There are many types of cameras available today in the marketplace. Recent advancements in technology have created a trend in the proliferation of digital cameras over its analog counterpart.

Figure 1. Picture of fence sensor courtesy of RBtec.

Figure 2. FLIR Voyager II infrared camera.

Other advancements have taken place in the adjustable cameras often referred to "PTZ" for pan, tilt and zoom. These cameras can be controlled and adjusted remotely from the comfort of a central location. These provide flexibility to security operators by allowing one camera to monitor a large area that traditionally would have required multiple cameras. There are tradeoffs associated with PTZ cameras compared to fixed ones, such as the chance of missing the occurrence of an event due to the field of view of the camera being pointed in a different direction. Since PTZ cameras have movable parts they have a lower mean time between failures (MTBF) figures when compared to their fixed counterparts. PTZ cameras are often used with other sensors when the latter generate alarms. The PTZ camera's field of view can be directed to the coordinates of the generated alarm of the other sensor for verification purposes. This technique when automated in security systems is coined "target correlation" or "sensor fusion". This integration of sensors can be used to reduce false alarms.

4.5.2 *Video Motion Detection*

These sensors are typically associated with the analysis of digitized images captured by video cameras. Movement is detected as a change in the video signal in relation to a reference image of a specific location, video scene or part of a video scene. The detection algorithm can be embedded in digital signal processing (DSP) micro chips inside cameras or as a software application running in computer servers. The advantages of video motion detection (VMD) are many. For instance it can generate alarms in real time for immediate verification by a security operator, allowing for a quick and appropriate response to an incident. Another capability afforded with VMD is the tracking of moving objects. Tracking takes place at two levels. Within the image frame, the software traces the movement of an object in its field of view. The other kind of tracking takes place when the VMD software is integrated with the operational controls of PTZ cameras. The result is an automated tracking of movable objects not only within an image frame but throughout the field of view of a movable camera. This is a very powerful tool for responders and forensics analysts. For example, he VMD can also be used as a forensics tool to analyze video that has been stored for a period of time. The VMD system can be programmed to search the recorded video file captured over a 24 hour period, pause on the first frame it detects video motion and generate an alarm. This procedure can be conducted while fast forwarding the video file, saving time and alleviating the dedication of a human resource to review a long monotonous video.

4.5.3 *Video analytics*

These technologies are also applied to video images typically captured by digital video cameras. These algorithms are programmed to generate alerts when certain patterns or behaviors are recognized and detected in video images. Some of these algorithms are a derivative of video motion detection (VMD) systems. One behavior is known as "objects left behind" such as the detection of a small suitcase left by a car at the entrance of a tunnel. Another behavior is "objects removed", such as the removal of a toll collection box from the field of view of a camera. These algorithms also allow the creation of virtual fences and trip wires in the cameras field of view. When an intruder virtually trips the wire or go through the virtual fence the system will generate an alarm. This tool facilitates automating the monitoring of large areas by video surveillance systems.

4.5.4 *License Plate Recognition (LPR)*

License Plate Recognition is a video analytics that is widely used in the transportation sector. The LPR system is comprised of many technologies that include specialized cameras, video analytics, optical character recognition (OCR) and database access. Cameras are strategically placed to capture images of license plates as vehicles travel along the highway or road. The system is programmed to recognize license plates as the vehicles move in the camera's field of view. The system then captures the license plate image and converts the content into text using OCR technology. Once the license plate information has been converted to text data, it can be used to query different databases for verification and/or lookup. This has application in access control systems for the operation of entrance gates or automated toll systems in highways and bridges. LPR has applicability in law enforcement as well, for example, in verifying if a license plate is associated with a stolen vehicle.

4.5.5 *Video Management Systems (VMS)*

These systems are end-to-end integration of systems and technologies that allow an enterprise to manage their entire video system, from the controlling of the cameras to the capture, storage and retrieval of video. The configuration, operations and maintenance of a video system is performed on the VMS. Video analytics and alarms are configured and responded to via the use of this platform. This system will also provide the platform to conduct forensics in an automated and effective manner. Certain VMS are designed with the necessary facilities to safeguard video from the moment of its capture through its retention lifecycle, providing chain of custody so that captured video and/or images can be used as reliable evidence in the courts of law.

4.6 *Sensor fusion*

As the number and types of sensors deployed in any physical security system grow the level of complexity to manage and operate the different subcomponents will increase due to the sheer number of alerts and alarms generated. Systems that properly combine or fuse the outputs of the different sensors automatically could minimize false/nuisance alarms and facilitate the integration of the different sensor subsystems. One of the objectives of these types of systems is to provide situational awareness more efficiently by reducing duplicate targets counts generated when the different sensors types detect the same object. For example if a fence sensor and radar pick up the same intruder each of these two systems will generate an individual alarm. The sensor fusion system will correlate the two targets by comparing temporal-spatial parameters and fuse the output of the sensors to display only one target. These systems can provide the platform for a Common Operating Picture (COP) enhancing operational awareness and response agility.

4.7 *Global Information Systems (GIS)*

Global information systems are tools that allow users to archive, display and analyze data spatially. A typical GIS system will display the map of a facility. Pertinent data represented as icons will be displayed spatially based on coordinates on the facility map.

5 TECHNOLOGIES APPLICABLE TO PHYSICALLY SECURING BRIDGES

The Blue Ribbon Panel on Bridge and Tunnel Security identified structural components of suspension bridges that are critical. These were Suspender ropes and stay cables, Tower leg, Main cable, Orthotropic steel deck, Reinforced and prestressed bridge decks, Cable saddle, Approach structures, Connections, Anchorage and Piers. In addition the Panel highlighted threat mitigation guidelines that included: the establishment of secure perimeter using physical barriers; Inspection surveillance, detection and enforcement, CCTV; Visible security presence; Minimize time on target (AASHTO, 2003).

The Blue Ribbon Panel recommended counter-measuring potential terrorist activities if it was assessed that specific bridges or tunnels were not secure enough. This paper takes the position that deployments of perimeter intrusion detection technologies, at a minimum, will meet the counter-measure of detection by sensing the presence of unauthorized intruders at bridges. This paper also claims that if PIDS are deployed as part of an overall multi-stage physical security system which includes early alerting and response support, the holistic security approach will position owners and operators to also meet the counter-measurements of attack deterrence, access denial and facility defense recommended by The Panel. The deployment of overt sensors can provide a level of deterrence by reducing the asset's attractiveness to attack. The strategic placement of covert PIDS sensors can provide early intrusion warnings, facilitating a timely response and therefore aiding in the defense of a facility. Lastly, a successful response aided by a PIDS early warning alert can deny facility access to potential intruders.

Let's assume that during a prioritization analysis it is concluded that a certain bridge asset is vital to the interest of a major urban center. Structural damage to the bridge would cause dire economic losses and flow disruption to a multi-modal regional transportation corridor. Given the importance of the asset a comprehensive all-hazards threat assessment exercise is conducted resulting in unacceptable levels of risk associated with certain bridge structural components. The threat analysis indicated that the structural components that pose the most risks are the bridge anchorages, suspender ropes, stay cables and tower legs.

As noted previously, a holistic and comprehensive approach to physical security design, inherently integrated within the operational life cycle of the asset can be more effective than current security practices.

Please refer to Figure 3 below for an illustration of a multi-tier security system applied to a toll bridge as described in this section.

A multi-tiered security system approach is appropriate for the bridge in question. Some of the identified structural components will be hardened such as the suspension cables and ropes. Perimeter security will be enhanced by the deployment of barriers and high fences in a concentric arrangement. Concrete barriers will be placed to create an outer perimeter border around the bridge access ways and employee parking area. Fencing will be installed around the anchorages area of the bridge to deter and obstruct the progress of unauthorized intruders.

Furthermore, access control systems will be installed using proximity cards to access the employee parking and main building. The access control system for the law enforcement parking area will employ proximity cards and License Plate Recognition (LPR) to operate the access gate. The two levels of vehicle and personal verification will increase security in an area where mandatory control access has historically not been enforced.

In conjunction with the installation of the concentric perimeter establishment devices and access control systems, a complementary perimeter intrusion detection system (PIDS) will be installed to provide an efficient early warning system against unauthorized intruders. The selected edge sensors, a key element of the early warning system, will be a combination of seismic barrier and fence sensors designed to generate alarms based on configured vibration parameters customized for the local environment. In addition, passive infrared sensors will be placed covertly in strategic areas between the barriers and fence to provide an additional level of alerting. The placement of the infrared sensors has been closely coordinated with the first responder unit's operating procedures.

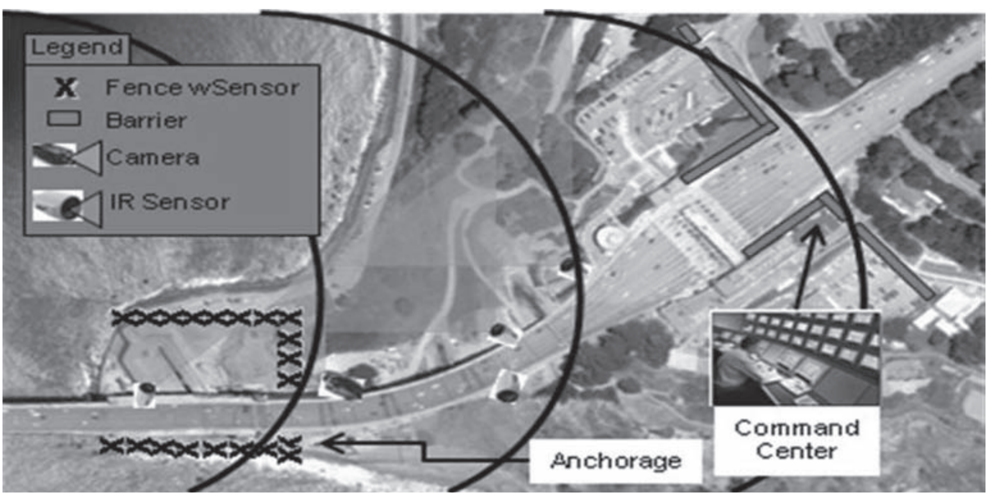


Figure 3. PIDS sensors deployed concentrically from bridge anchorage.

The sensors are designed to generate alarms with enough time to allow an adequate response to unauthorized intrusion into areas considered highly sensitive.

Additionally, different types of video surveillance cameras have been installed in strategic locations to provide security operators monitoring capabilities throughout the facility. PTZ cameras have been installed in areas covered by fence and infrared sensors. Fixed cameras have been placed in locations where the installation of other sensors was impractical. The fixed cameras have built in video analytics where virtual trip wires and virtual fences have been designed to generate alarms close to the shore areas of the facilities. The virtual trip wires have been configured to monitor specific areas where facility vulnerabilities have been identified sourcing from shallow water areas. Since these areas are not well lit at night the fixed cameras are capable of day/low light operations. Two of the cameras are thermal in nature with the ability to detect infrared emissions from live objects.

Megapixel cameras with License Plate Recognition (LPR) capabilities have been installed on trusses beyond the outer perimeters of the facility focusing on incoming vehicles. The LPR system is tied to external databases such as Motor Vehicle Department and two federal terrorist watch-lists. Additionally, the cameras deployed at the toll booth area have been integrated with the newly installed LPR camera system as well as the external databases. If the new LPR system matches an incoming vehicle's license plate with an entry in the terrorist watch-list it will generate a message to the first responders and the legacy camera system at the toll booth. The LPR system at the toll booth will now be on the lookout for the license plate of interest. Operational counter-measures can be taken by closing down toll lanes and funneling incoming traffic and slowing it down. By the time the vehicle of interest reaches the toll area the gate will not open and the response unit will be in place waiting to interdict the vehicle, passengers and cargo of suspected interest.

At the owner/operator Operations Center a GIS and a sensor fusion system have been installed. These two systems form the core of the command and control function for the facility's security operations. PTZ cameras have been programmed to sweep across a wide area including the locations of the fence, barrier and infrared sensors. The cameras, barriers and sensors have been integrated with the fusion system. This allows the programming of PTZ cameras to focus on specific geographical areas where sensors have triggered alarms. This will facilitate security operators to quickly verify the nature of the alarm and, if required, initiate the appropriate response. The flexibility of remotely controlling, pointing and focusing PTZ cameras will provide heightened situational awareness at all stages of security operations; whether during the early stages of an intrusion detection or the later stages of an incident response. The integration of the GIS system with the sensor fusion system provides an improved level of situational awareness, where alarming sensors are highlighted on a facility map in flashing manner while the path of intruders as well as blue forces are accurately tracked geospatially. Specific aspects of the situational awareness information can be communicated to field personnel responding to incidents. For example, video feeds of the PTZ cameras auto-tracking the intruders can be re-directed to field agents PDAs and handheld devices facilitating the search and interdiction.

6 CARING AND FEEDING OF THE PIDS

Like most technology and capital projects PIDS has a lifecycle that includes planning, funding, installation, operations, maintenance, technology refresh and finally, retirement. Key phases of any PIDS are the design and planning stages. Understanding and foreseeing the operations of the system and needs of the operator not only soon after installation but also well into the future is crucial to developing any design specifications. Since most of the sensors and technologies associated with PIDS are electronic devices that generate data they require electrical power and network connectivity. These requirements pose logistical installation challenges since sensor placements will be outside in inclement environments exposed to extreme climatic conditions throughout the seasonal changes. In addition to power and data connectivity, ruggedized or hardened containers are required to house sensitive electronic components in unfriendly environmental conditions.

The same ruggedized shelters must allow sensors to capture video and detect low levels of disturbances and energy fluctuations emanating from the environment they're designed to hermetically hinder. Some of the most corrosive elements known to electronics are water and saline. Most bridges are geographically located over bodies of water with high levels of humidity and salinity. For electro-optical devices, these environments impose an added maintenance burden as lenses and glasses need to be kept clean and clear since optimal performance requires unobstructed field of view. Scheduled seasonal changes, lighting and shadow conditions at different times of the day as well as times of the year will impose yet other technical and logistical challenges to a PIDS installation. Since line of sight with adequate lighting is required for the optimal performance of some sensors, foliage and shadow changes must be well understood. Traffic flows of pedestrians, vehicular and migratory animals should be taken into consideration. Mounting assets for optical sensors should be structurally sturdy to withstand strong winds as well as the housing, mounting hardware and sensors weights. The mounting assets should also facilitate the provisioning of power and network connectivity. Other considerations that should be taken into account when positioning of sensors, such as camera, are the logistics of maintenance and repair. Oftentimes the location offering the best visual vantage point provides logistical challenges from a maintenance and environmental perspective. Maintenance and repair issues that should be taken into consideration when selecting mounting locations include, level of manual effort, climbing challenges, obstruction of traffic flow and unacceptable levels for risks to field personnel.

Fence sensors offer different sets of installation and operational challenges. The high winds and roaming wildlife can cause fence sensors to trigger false alarms. Some fence sensor systems have the capability of adjusting the alarm threshold parameters to take into account frequency, intensity and pervasiveness of vibrations. The fusion of PTZ cameras with fence sensors will aid in the reduction of false alarms by allowing visual verification of a triggered alert.

7 ORDER OF MAGNITUDE COST

When preparing a financial budget for a physical security system all costs associated with every phase of its life cycle should be identified and estimated. For the purposes of estimating the cost of a security system this paper will focus on the planning, installation and on-going operations and maintenance for a period of 3 years.

The planning of a project should take into account requirements gathering including the user's operational expectation as well as local terrain and environmental surroundings. The output of this phase will be an overall system detailed design and estimated equipment, systems, installation and testing costs including the project timeline and contingencies. Systems maintenance and operational training costs should be identified in this phase as well.

Operational, maintenance and support costs should include inventory sparing, recurring expenses, warranty expiration period, out of warranty repairs, upgrades, technology refresh, frequency and complexity of preventive and scheduled maintenance activities, in-house or outsourced maintenance and local labor costs.

The figures provided in this section are based on previous experience designing, costing and implementing multiple physical security projects. These deployments included mostly video surveillance cameras installed in an urban environment with a substantial portion of the connectivity being provided with wireless technologies and the operational infrastructure housed at the command and data centers. The loaded cost per camera has been estimated to be between \$15,000 and \$30,000. These figures typically include maintenance and support cost for approximately 3 yrs. The costs per camera figures cover a wide range and there are many factors that affect the overall price of a system. Some of these factors include the quality (frame rate and resolution) of the video being transmitted and recorded, the amount of time video is stored before being discarded, dependence on whether permitting is necessary for equipment mounting, level of systems backup and redundancy required, systems performance specifications, extremeness of climate external devices are exposed to, natural and man-made environmental surroundings and terrain, local labor costs, distance between sensors and command center, number and types of client end users consuming situational awareness information, ratio of camera count to central or "head-end" infrastructure. Typical maintenance cost for video surveillance systems can be estimated to be 25% to 30% of the non-recurring costs per year.

The installation of a PIDS within a bridge facility might have a different cost structure dependent on factors such as the length of the structure, the type of sensors deployed, the sensor density, the use of wired or wireless technologies, the availability or ease of installing power and network connectivity in areas of interest and others.

8 CONCLUSIONS

Since the 1990s infrastructure public policy has focused on the concern of protecting critical assets from terrorist attacks. The federal government has employed grant funds as a vehicle to implement these national policies at the state and local level. Operators and owners of bridges that are classified as critical infrastructure assets have a national mandate to protect them from terrorist attacks.

This paper takes the position that perimeter intrusion detection systems (PIDS), when deployed as part of a comprehensive security operations environment, can be an effective tool to deter attacks, deny access, detect intrusions and defend facilities.

Security of infrastructure can best be achieved by addressing it in a holistic manner. Security should be an integral part of the assets' design, day to day operations and maintenance activities. Security should be a comprehensive set of integrated systems, technologies, processes, procedures and manpower efforts tailored to the unique environment, threats and operational needs of the specific asset.

As noted at the White House web site, President Obama has articulated as an issue in his agenda the modernization of America's aging infrastructure, including upgrading highways rail, ports, water, and aviation infrastructure. It is still too early in Obama's term to assess how and if critical infrastructure protection policies will deviate from those of the past 12 years. The Obama administration has a unique opportunity to incorporate security and resiliency in the design and construction of new critical infrastructure resulting from the announced modernization initiative.

REFERENCES

American Association of State Highway and Transportation Officials (AASHTO). Blue Ribbon Panel on Bridge and Tunnel Security. 2003. *Recommendations for Bridge and Tunnel Security.* Washington, D.C.: Author.

Moteff, J. and Parfomak, P. 2004. *CRS Report for Congress RL32631 Critical Infrastructure and Key Assets: Definitions and Identification. Washington, D.C.*

Chapter 24

Development of a baseline model for an arch using diagnostic tests

J. Liu
WSP SELLS INC., New York, USA

H.W. Shenton
University Of Delaware, Delaware, USA

ABSTRACT: Cracks were found in several floorbeam-to-tie girder connection angles for a tied arch in a biannual inspection. After replacing those damaged angles and frozen stringer bearings, the cracks appeared soon. A series of diagnostic tests were performed in order to identify the crack causes. The objective of the study presented was to develop global and local finite element models and calibrate those models to obtain a baseline model using testing data to find the crack causes.

1 INTRODUCTION

The massive transportation network that is made up of bridges, highways, railways, waterways, and airways is vital to national and global economies. The condition of the transportation infrastructure is therefore a critical concern for governments throughout the world. Many of the U.S. bridges were built after World War II, and are now approaching the end of their design life. These bridges are deteriorating due to environmental degradation, increasing volume and weight of truck traffic, and lack of effective repair and maintenance actions.

Performance studies of bridges can use static or dynamic data or both static characteristics include displacement, slope, and strain, while dynamic characteristics include natural frequencies, mode shapes, mode shape curvature, damping, etc. Changes of one or more of those characteristics may be an indicator for performance changes in a bridge (Doebling etc. 1998, Salawu, 1997, Carden etc. 2004). Today, the finite element model (FEM) is a mature and efficient tool to simulate construction stages and condition assessment in bridge engineering (Liu, 2006). The typical procedure consists of developing an FEM, studying parametric sensitivity, and obtaining the baseline model matching experimental data.

2 DESCRIPTION OF THE BRIDGE

The Chesapeake City Bridge is a public crossing over the Chesapeake and Delaware Canal in Maryland and Delaware. Carrying two lanes of traffic, this bridge includes multiple span approaches and a steel tied arch. It was built between 1947 and 1948 and opened to traffic in 1949. The length of the bridge is 1200 m (3,955 ft), and the length of the main tied arch span is 165 m (539.5 ft). The arch consists of arch ribs, tie girders connected through hangers, and a floor system (see Figure 1). The floorbeams connect the tie girders, and stringer beams are placed on the floorbeams. In the original design, the main bearings at the north end were fixed and rotated freely, while the cast rocker bearings at the south end could slide freely. The floor system of the arch consists of floorbeams, stringers, and a fully composite deck on the stringers, diaphragms, and lateral bracing (see Figure 2). The floorbeam-to-tie-girder connections are south and north angles attached along the full length of the floorbeam webs. The stringer bearings were designed to be

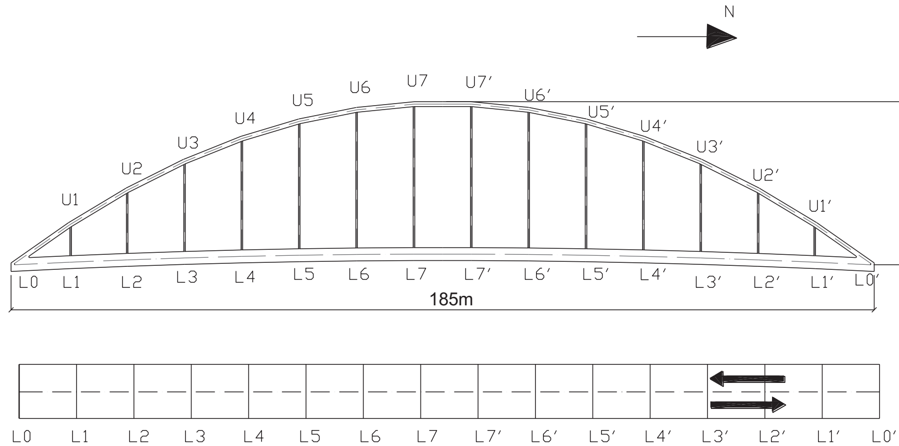


Figure 1. Elevation and plan view.

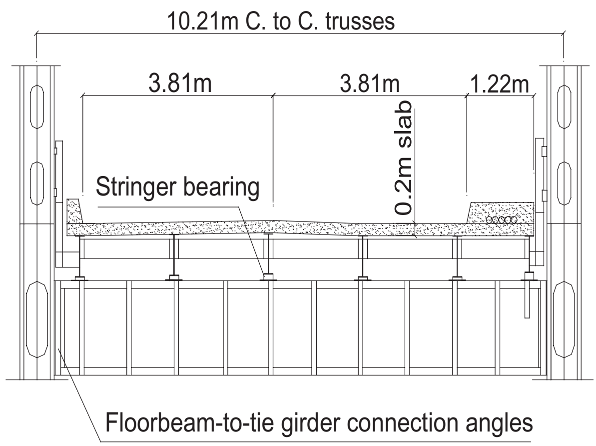


Figure 2. Typical floorbeam (looking north).

fixed at one end and expansion at the other end, except for the middle panel, where both stringer bearings are fixed.

The objectives of this study are to identify the parameters, and build an updated baseline model to explain the experimental data in order to determine the causes of the cracks in floorbeam-to-tie-girder connections.

3 HISTORY OF DISCOVERED CRACKS

Cracks were found because of biennial inspections conducted from 1995 to 2001 on the several floorbeam-to-tie girder connection angles. The inspection results were summarized as follows:

1. In the 1995 inspection, cracks were found in the floorbeam connection angles at panel point L0' (south and north connection angles), and panel point L1' (south and north connection angles). To arrest the five cracks, 19 mm (3/4 in) diameter holes were drilled;
2. In the 1997 biennial inspection, the crack found in the 1995 inspection at floorbeam L1' south connection angle of east tie-girder had extended beyond the previous hole. An additional

19 mm (3/4 in) hole was drilled to arrest the extended crack and another new crack on the other side of the angle fillet;

3. In the 1999 biennial inspection, a new crack was found in the southwest connection angle of floorbeam L0’. This was the first time that a crack had been discovered on the west connection angle of the floorbeams. The crack found in the floorbeam southeast web connection angle in the previous inspection had extended beyond the second hole, and
4. In the 2001 biennial inspection, the cracks in the southeast and northeast angles at floorbeam L1’ had lengthened.

4 THREE FIELD TESTS

To discover the causes of the cracks and provide input to numerical modeling, a series of field tests were conducted. In those tests, relative displacement between the stringer bearings and stringers, as well as strains at the floorbeam connection angles and top and bottom flanges, were measured. Because the floorbeams are the secondary elements in the arch system, the test results will be sensitive to the complex boundary conditions of the stringer bearings. It is difficult to determine global behavior from the strain tests. So a third test to measure the bridge displacement under the test truck was performed. Test trucks and total loads were almost similar for the three tests.

4.1 *2001 field test*

In March 2001, a field test was conducted to discover the likely cause of the cracks in the floorbeam connection angles. Strains in the floorbeam web connection angles, flexural strains on the floorbeam flanges, and longitudinal displacement of the stringers at the stringer bearings were measured on floorbeams L1, L1’ and L2’. Data from the test truck and ambient traffic were recorded. Because floorbeams L1 and floorbeams L1’ and L2’ are at different ends of the arch, the tests had to be conducted individually. Eleven truck passes were used to test floorbeams L1’ and L2’, and thirteen truck passes were used for floorbeam L1.

Stresses around the cracks were the focus of the test. Gauges were positioned to measure the horizontal and vertical strains in the outstanding legs of the connection angles. Cable potentiometers were used to measure the relative displacements between the stringer bearings and the stringers. Three important conclusions were made based on the measurement results:

1. The strains and displacement were predominantly due to the global behavior of the arch; while a much smaller contribution could be attributed to local effects when the test vehicle moved over the floorbeam;
2. For all twelve bearing displacements, only one potentiometer presented a significant displacement, which meant that most stringer bearings that had been free in the original design were now frozen, and
3. The high stresses in the floorbeam connection angles were from out-of-plane deformation, i.e., torsion of the floorbeams, which was attributed to frozen stringer bearings.

4.2 *2004 field test*

Extensive analyses using numerical modeling have been performed by the Baker, and the 2001 test verified the analysis. The most important conclusion is that frozen stringer bearings induced higher stresses in the connection angles. A repair was conducted to decrease the higher stresses. All cracked angles and all stringer bearings on floorbeams L1, L2, L3, L3’, L2’, and L1’ were replaced by new angles and elastomeric bearings, respectively. To verify the effectiveness of the remediation, a new field test was conducted in 2004 to measure strains in connection angles, strains on floorbeam flanges, stringer bearing displacements, and relative displacement between the top of the floorbeam and the tie girder on floorbeams L0’, L1’, L2’ and L3’. The typical instrumentation setup for the four floorbeams is presented in Figure 4. The north end floorbeam L0' is unique. In 2002, the connection angles on floorbeam L0' were replaced, but after a few months, new cracks appeared. The east connection was coped out and stiffeners were added below the adjacent stringer bearing. Also during the testing, it was found that one of two anchor bolts at the fixed end of the eastern- and westernmost stringer bearings over floorbeam L1' were sheared off, and one bolt at the fixed end of the easternmost stringer bearing and two bolts at the fixed end of the westernmost stringer bearing over floorbeam L1' were sheared off. The new bolts were sheared off only about two years after the old stringer bearings had been replaced, and the reason was unknown at that time.

A total of twenty truck passes combined with ambient traffic response were recorded for the 2004 test. Here, only some representative data have been plotted and explained. One can see that the SGX 12 strain time history curve is a superposition of a gradual sinusoidal shape (global behavior) and a short pulse (local effect) in Figure 4 and All other gages installed at the same place for other three floorbeams show the same phenomena. As soon as the test vehicles moved onto the opposite end, forces immediately were transferred to those floorbeams, but the design drawing shows the deck is discontinuous. Figure 5 shows that all four stringer bearings were functioning at floorbeam L0' and moved in the same direction. The displacements for the outer bearings were much larger than those for the inner bearings, which meant that the outer bearings experienced larger forces than the inner bearings and all four bearings experienced shear forces in the same direction. The signs of strains and the relative positions of these four gauges (SGX 12, SGX 13, SGX 14 and SGX 15, installed in the same plane) allowed the assumption that torsion was playing a large role (see Figure 4). The two top strain gauges, SGX 12 and SGX 13, were opposite in sign, as were the two bottom strains, SGX 14 and SGX 15, for the first half of the sine curve. For the second half of the sine curve, the torsion effect was not prominent when the vehicle moved near floorbeam L1'. The magnitude of the top strains was higher than that of the bottom strains. There are two possible explanations for this phenomenon. First, torsion from the bearing shear was not enough to turn the entire section. Second, the gusset plate on the bottom connecting the floorbeam, tie girder, and cross bracing strengthened the bottom flange and distributed the torsion.

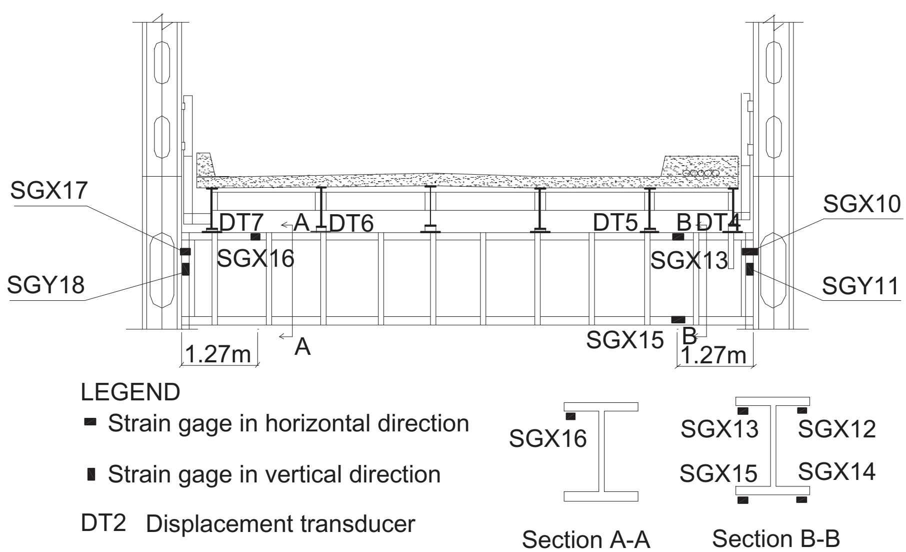


Figure 3. Instrumentation setup on floorbeam L1' (2004 Test).

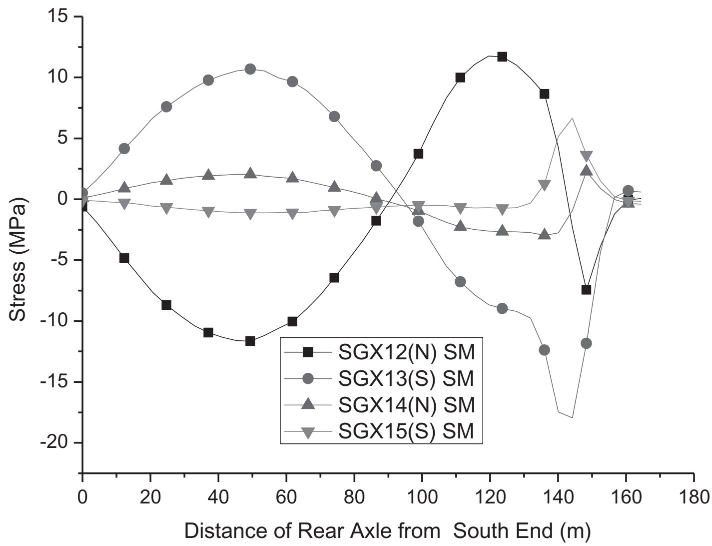


Figure 4. Strain time history for floorbeam L1' (northbound).

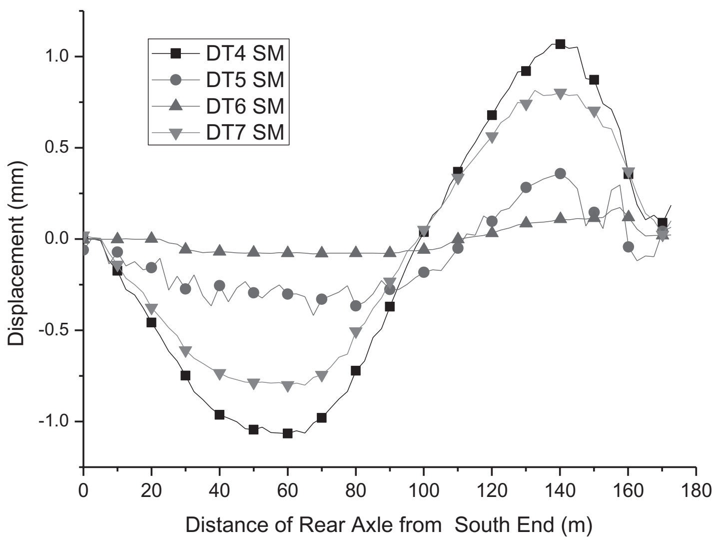


Figure 5. Stringer bearings displacement at floorbeam L1' (northbound).

4.3 *2005 field test*

In the 2001 and 2004 tests, only floorbeams (secondary elements in the arch) were instrumented. As a result, it was difficult to predict the global state of the arch. A new static test was conducted to measure the global deflection under the test vehicle in 2005. Four cases were measured (see Figure 6), and the test results are shown in Table 1, deflections when the dump truck was at L4' were larger than those when the dump truck was at L7'. The reason is that the arch ribs act like a fixed beam between the middle and end of the arch rib.

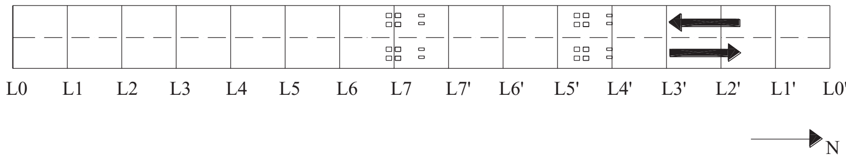


Figure 6. Four test vehicle locations.

Table 1. Deflection survey data (unit: mm).

	Front axle at mid. span		Front axle at L4’	
	East point (L/2)	West point (L/2)	East point (L4’)	West point (L4’)
Northbound lane (Traveling North)	12.2	9.1	15.2	15.2
Southbound lane (Traveling North)	6.1	12.2	15.2	24.4

5 DEVELOPMENT OF FEMS

Understanding the stress distributions under the test vehicles is the focus of the modeling. The complexity of the bridge makes it difficult to model it precisely, so two models were developed using STAAD Pro 2004. One model is a three-dimensional global model using shell elements for the deck and beam elements for other components. Rigid links are used to connect the concrete deck and the stringers. The other is a refined local model for floorbeams using shell elements. The distribution of live loads on the floorbeams, obtained from the global model, was applied to the local model.

5.1 *Global model*

The three-dimensional global model is developed based on the dimensions from the design drawing (see Figure 7). Because there is a distance between the stringer bearings and the centroid of the stringers, rigid elements are used to connect the stringer bearings with the floorbeams (see Figure 8).

In the original design, the main bearings of the south end were allowed to move along the length of the bridge, while the main bearings of the north end were designed only for rotation about the transverse axis. In a visual inspection, movement of the south main bearing was observed, but there was no indication of rotation of the north bearing, because the paint on the north bearings showed no apparent cracking. Based on these observations, the north main bearings are fixed in the nominal model, while the south main bearing moves along the length of the bridge. The actual performance of the main bearings will be identified through the test results.

The old stringer bearings at both ends of each panel were fixed based on the 2001 test, while the new stringer bearings provide free longitudinal movement at one end and constraints at the other end of the panel based on the 2004 test. The floorbeams are connected to the tie girders using rigid elements. This modeling technique provides flexibility to model the connection cracks by adjusting the stiffness of the rigid elements. The end nodes of the top and bottom lateral bracings are offset from the centroids of the tie arches and girders, respectively. Stringer bearings on floorbeams L1, L2, L3, L3’, L2’ and L1’ were replaced by two kinds of new elastomeric bearings. Considering that rotational stiffness may affect the final results, theoretical values should be obtained. The average rotational stiffness for type 1 and type 2 of both bearings is 5.63 KN-m/degrees and 3.9 KN-m/degrees, respectively.

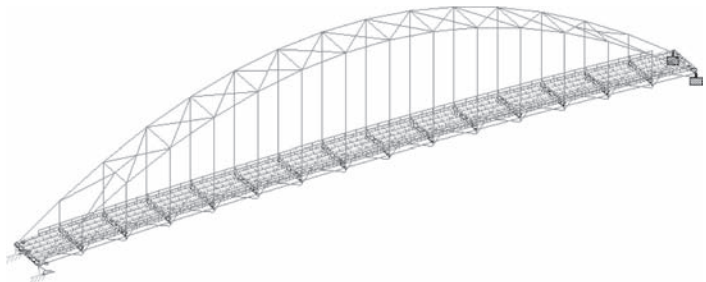


Figure 7. Global model.

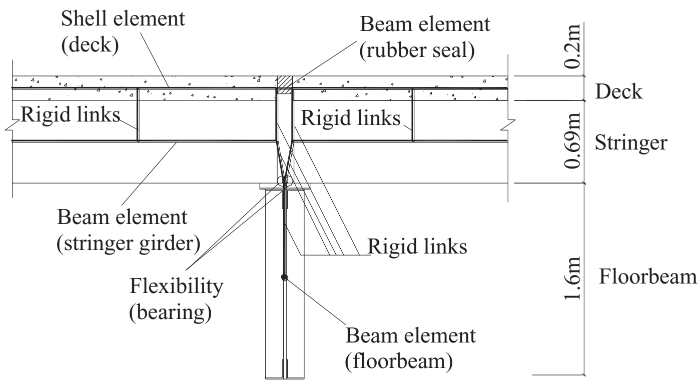


Figure 8. Schematic for connections between stringers and the floorbeam.

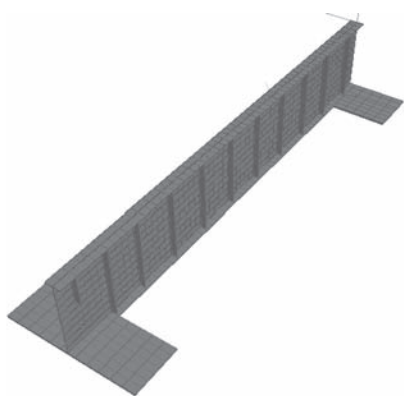


Figure 9. Local model for floorbeams.

5.2 *Local model*

A refined local model of the interior floorbeams is developed to study the localized effects on the top and bottom flanges (see Figure 9), where the strain gauges were installed. Because cracking occurred in the connection angles, the stresses there should be the focus of the study. However, the complexity of the boundary conditions in the connection angles makes exact modeling difficult. Angles at the top and bottom flanges are not modeled separately but are incorporated into the web and as the top and bottom flanges. Gusset plates connecting the bottoms of the tie girders and the floorbeams are also incorporated into the bottom flange. Pinned constraints are used for the sections that connect the floorbeams and the tie girders.

6 PARAMETRIC SENSITIVE STUDY

6.1 *Torsional stiffness for floorbeams*

The torsion stiffness of floorbeams is a critical parameter that affects the shear forces in stringer bearings. The St. Venant Constant, $C = \alpha bt^3$, is used to represent the torsional rigidity of the components, where α, b, and t are the constant for torsion of a rectangular section and the height and thickness of the components, respectively. There are two methods to calculate theoretical constants for the built-up sections. One is to calculate the components separately and then add them together. For the other, components having the same interfaces are first integrated as a single component, and then the constants are calculated by the first method. The actual floorbeams have stiffer torsional rigidity because of the transverse stiffeners, but there are no theoretical methods to take that into account. A simple method to consider the effect from the transverse stiffeners is to develop a model for the floorbeam. First, a local floorbeam without stiffeners is built, with one end free and the other end fixed. Then, a unit torque is applied at the free end. One can get a torsional angle α_1 for the free end. According to the formula $C = TL/G\alpha$, where T = torque, L = length of the element, G = shear modulus and α = rotational angle, a rotational stiffness of the model can be obtained. Then stiffeners are added to the model, and another rotational stiffness is obtained. The final results and comparisons are presented in Table 2. One can see that there is a significant difference between the separated and integrated theoretical results. Furthermore, the stiffeners increase the rotational stiffness by 60%.

6.2 *Parametric sensitive study*

Although it is desirable to have exact parameters for a bridge prior to modeling, it is not practical due to time and cost constraints as well as lack of appropriate techniques to obtain that information, especially for complex structures. Engineering experience can be used to determine reasonable upper and lower bounds that affect the model results. Then, based on the test data, the FEM can be calibrated considering the most likely parameters. All parameters can be classified as boundary conditions, material properties, or geometric properties. Seven parameters here may play the largest role in affecting the final results and need to be identified (see Table 3): (1) North main bearing conditions (transverse rotation); (2) Rotational stiffness of new and old stringer bearing (transverse rotation); (3) Deck continuity (elastic modulus of rubber seal); (4) Elastic modulus of the concrete slabs; (5) Connection angles from the floorbeams to the tie girders; (6) Sheared-off bolts on floorbeams L1 and L1', and (7) Railing.

There was plenty of data from the three field tests, but it is impossible to compare the FEM results to all field tests in the parametric study. The basic procedure is to choose the stresses at Gauge 21 for floorbeam L2', the same position to SGX 13 (see Figure 4), as a reference point to study the parametric effects. Then other parameters can be selected and studied for additional

Table 2. Comparisons to calculate torsional constants of the floorbeam.

	Rotational constant (cm^4)
Theoretical (separated)	221
Theoretical (integrated)	1422
Numerical (w/o stiffener)	1188
Numerical (w/stiffener)	1901

Table 3. Parameter bounds and combination.

Parameter		Nominal value	Bounds	M1	M2	M3	M4	M5	M6
North main bearings		Fixed (F)	Released (R)	F	F	F	F	F	F
Str. bearing	New	5.63 or 3.9	0.5, 1.5, 2 times	1	1	1	1	1	1
(KN-m/deg.)	Old	1000	0.5, 2, 4 times	1	1	1	2	1	1
Deck continuity (KN/m^2)		537	10, 50, 500, 1000 times and infinite (Inf.)	500	500	Inf.	500	500	Inf.
Slab concrete elastic modulus (KN/m^2)		2.1E7	2, 5, 10 times	1	1	1	1	1	1
Floorbeam to tie girder connection		Fixed (F)	Partially constrained (P)	F	P	F	F	F	F
Sheared-off Bolts		W/O	W	W/O	W/O	W/O	W/O	W	W
Railing		W/O	W	W	W	W	W/O	W	W

reference points to verify the accuracy of the calibrated model. The procedure is that unit loads are applied at individual bearing locations in the local model; the stresses are obtained at the positions installed gages, and then the interior forces from the global model are incorporated. It is important to note that any method that incrementally changes one parameter at one time neglects cross-sensitive effects. This means that the effects may not be the sum of the results of several parameters.

Figure 10 to Figure 12 present the stresses for the nominal model and are totally different from the test results. Figure 10 and Figure 11 do not show the superposition of the global and local effects, while Figure 12 is somewhat similar to the test results. The reason may be that floorbeam L3' is adjacent to floorbeam L4' with old stringer bearings. In Figure 13, N.M.E. denotes that the dump truck is in the northbound lane (N), the front axle is at the mid-span (M), and the recorded displacement is at the eastern point (E). The figure illustrates comparison of the measured displacement, and the model predicted displacement for fixed and released north end main bearings. For the four values at mid-span, the difference is small, while it is larger at the 1/4 span, especially when the north main bearings are released. There are two factors that affect the global test results. One is the main bearing constraints, and the other is the geometric properties of the arches and tie girders. The gross geometry properties, which consider the most likely of section properties, are used in the FEM. So the main bearings are only one factor affecting global behavior. Considering that there is no indication of rotation of the north bearings, it is safe to conclude from the experimental results that the rotation degree of the north bearings is fully constrained, while the south main bearings are partially constrained, although the constraint degree is not known. Whether or not the main bearings are con-strained does not affect the behavior of the floorbeams. Figure 14 presents the effects from the rotational stiffness of the new bearings. This parameter has a limited effect on the global behavior. There are no effects from rotational stiffness for the old bearings according to Figure 15. Effects of deck joint rubber seals are shown in Figure 16. Global behavior is affected by an increase in the elastic modulus of the rubber seals. The elastic modulus of the slab concrete has no effects on the global behavior, and only minor effects on local behavior as shown in Figure 17. In Figure 18, one can see that railings produce apparent effects, while connection constraints have little effect on global behavior.

Based on the above parametric discussions, only railings and the deck joint rubber seals affect global behavior. There are two possible explanations for increasing the elastic modulus of rubber seals in the parametric study. First, the rubber seal can age and harden over time. Second, the details under rubber seals are not known. It is possible that adjacent stringer girders were connected during the construction, which has happened in other bridges. For an element of area 377 cm^2, and 500 times the normal elastic modulus of rubber seals, 0.4 KN, the effect is equal to a 0.38 cm^2

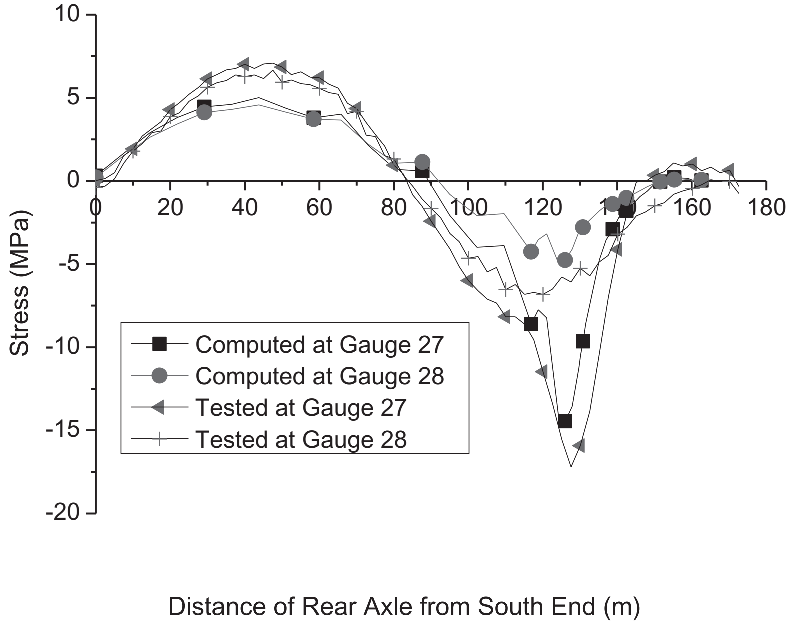


Figure 10. Stresses for the nominal model and test (Floorbeam L3’).

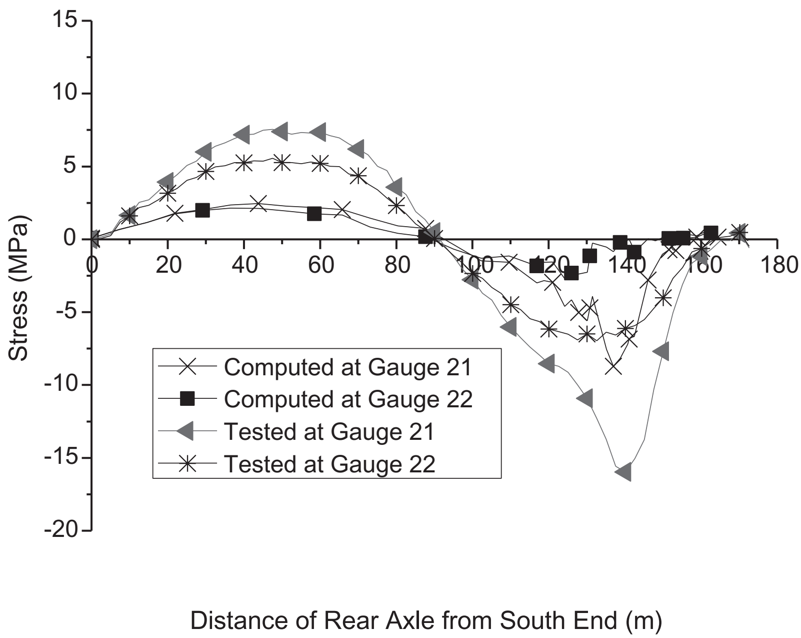


Figure 11. Stresses for the nominal model and test (Floorbeam L2’).

steel bar, which means that the global behavior is very sensitive to the type of connection. Third, the largest shear at the exterior bearings on Floorbeam L1’ is 0.1 KN for the baseline model, 5.9 KN when the elastic modulus of rubber is increased by 500 times. Sheared-off bolts on Floorbeam L1 and L1’ showed that there were large shear forces there, but the nominal model does not

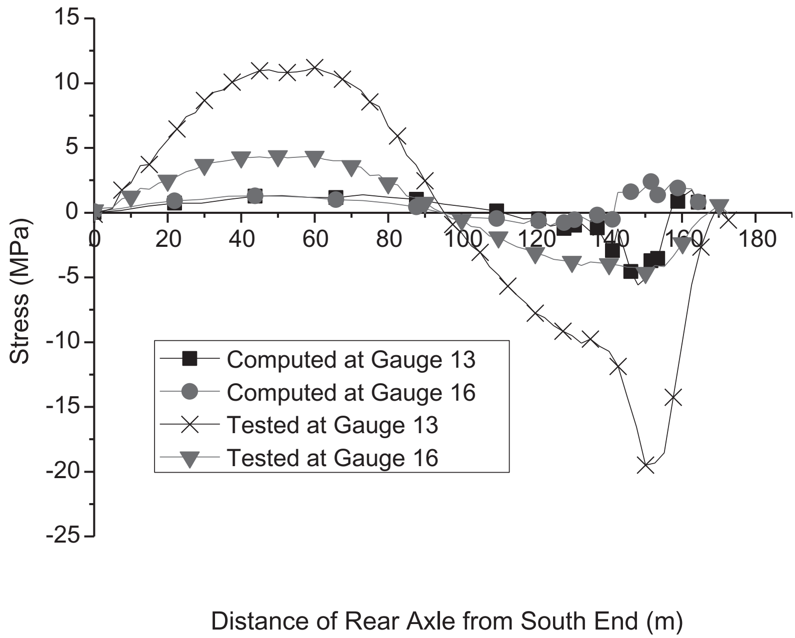


Figure 12. Stresses for the nominal model and test (Floorbeam F1').

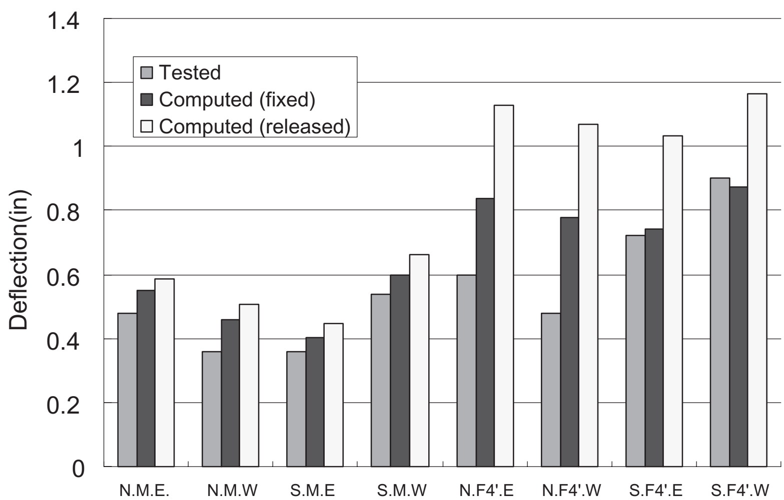


Figure 13. Comparison of displacements at eight tested locations.

indicate such large shear. Stress history in Figure 12 shows a large difference around the peak for the two gauges, while the other two tests did not show this phenomenon. The primary assumption is that the bolts are sheared off at the stringer bearings on Floorbeam L1', but this needs to be verified in the models.

Figure 19 to Figure 23 compare the computed stress history at six gauge positions for the six models. In the M1, the elastic modulus of the rubber seals and railings is 500 times that of the

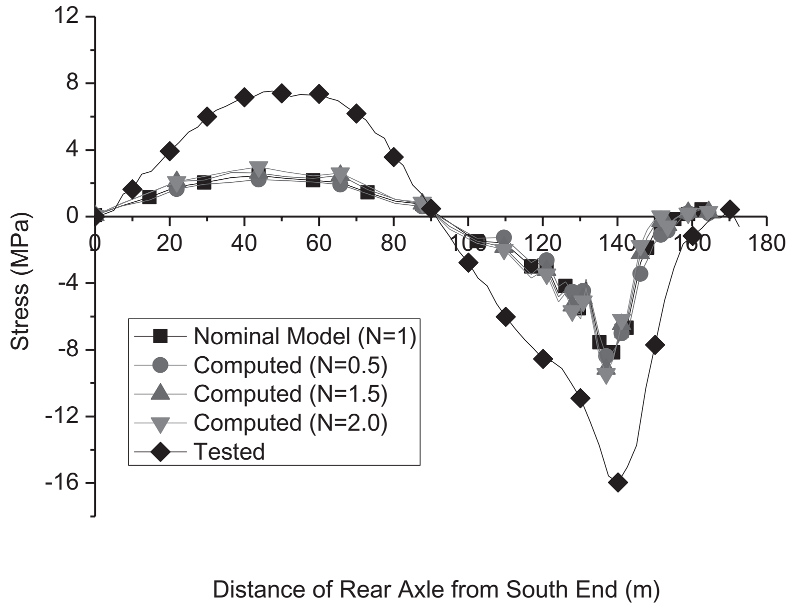


Figure 14. Effects of rotational stiffness of new stringer bearings at Gauge 21.

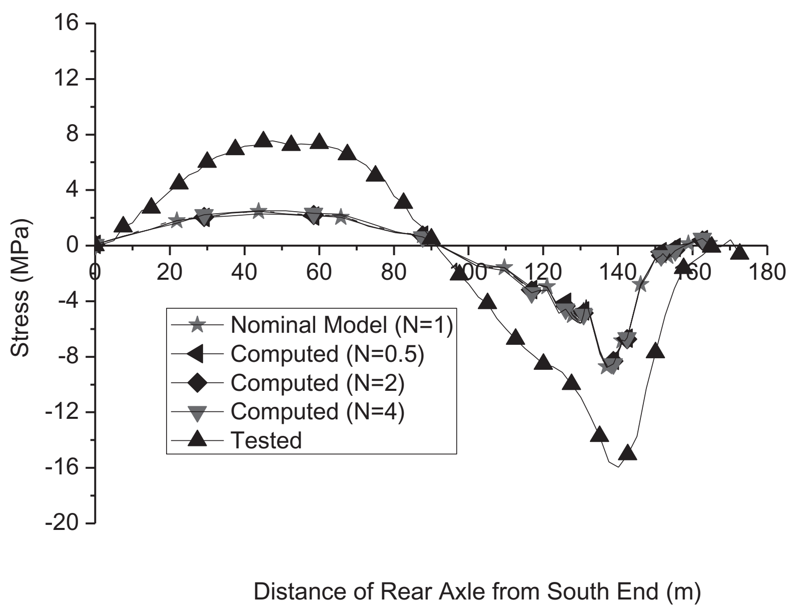


Figure 15. Effects of rotational stiffness of old stringer bearings at Gauge 21.

baseline model. But the peaks do not show the sum of the effect of the two parameters, which means that the effects from the two parameters are cross-sensitive. The M5 and M6 include effects from the sheared-off bolts. The stress history shows a difference around the peak between Gauges 13 and 16, while the other models do not show this effect. In the M3 and M6, adjacent deck slabs

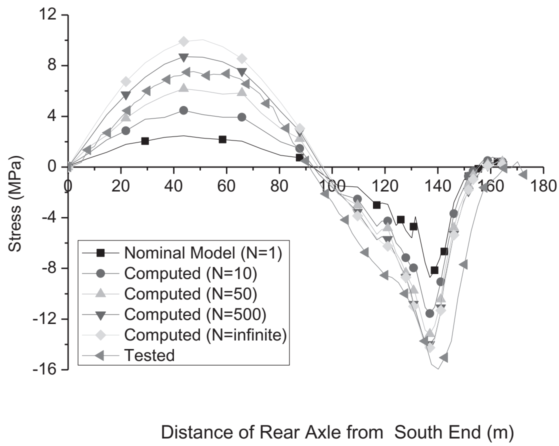


Figure 16. Effects of elastic modulus of deck joint rubber seals at Gauge 21.

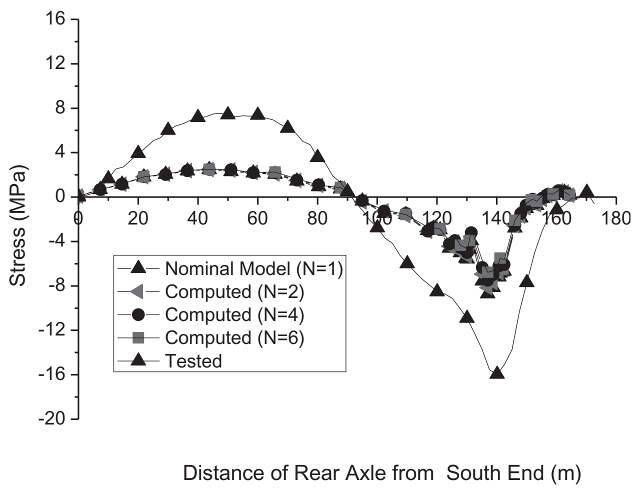


Figure 17. Effects of elastic modulus of slab concrete at Gauge 21.

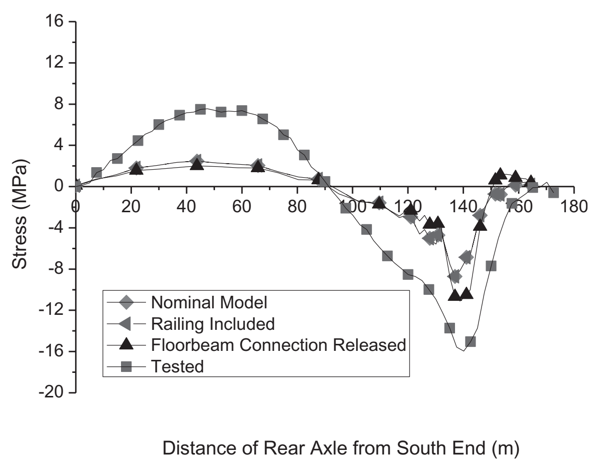


Figure 18. Effects of railings and floorbeam at Gauge 21.

are connected by rigid links along the longitudinal bridge. The stress history curves are similar to the testing results. The M5, which is similar to the experimental results and has reasonable explanations for the parameters, is the recommended baseline model for the bridge. Although there is some difference around the minimum values, this model does not consider the effects from

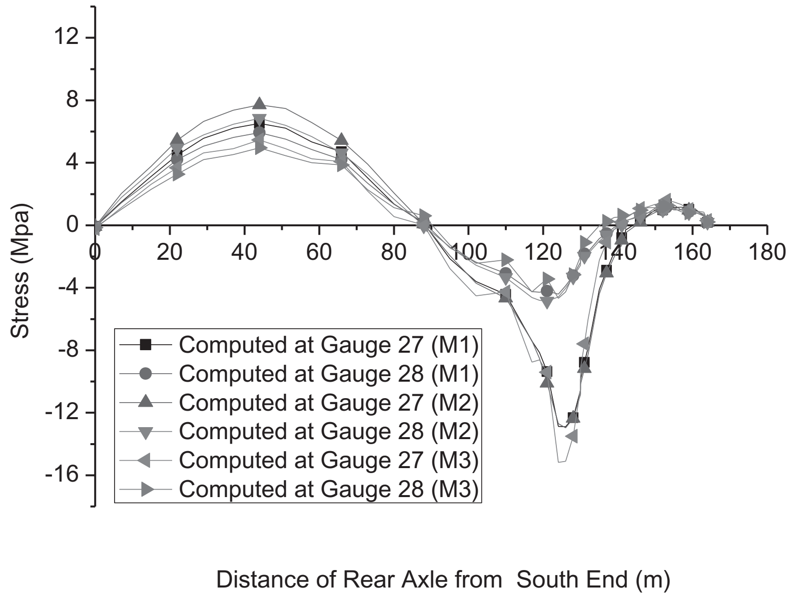


Figure 19. Comparison of computed stress history.

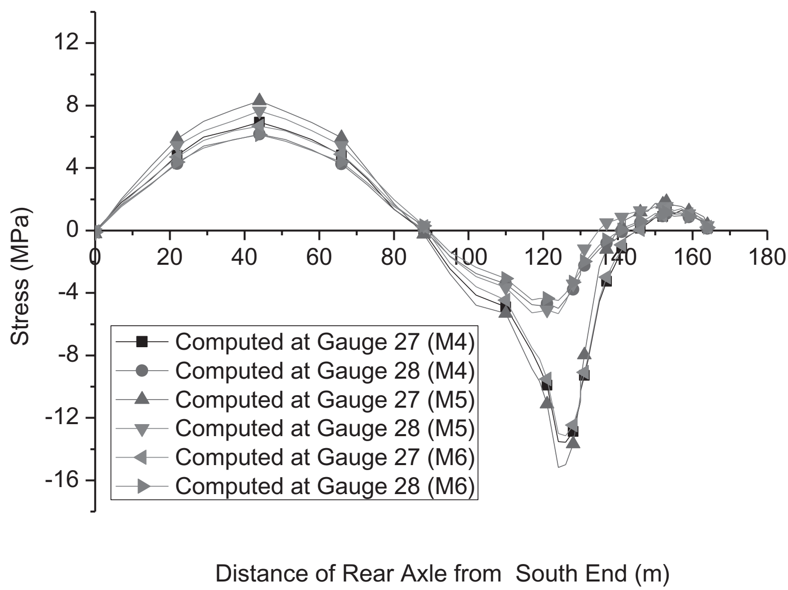


Figure 20. Comparison of computed stress history.

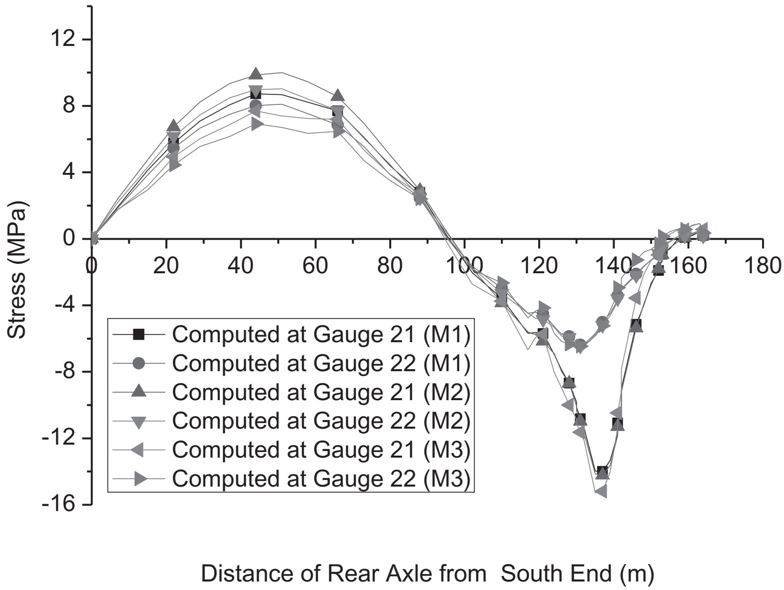


Figure 21. Comparison of computed stress history.

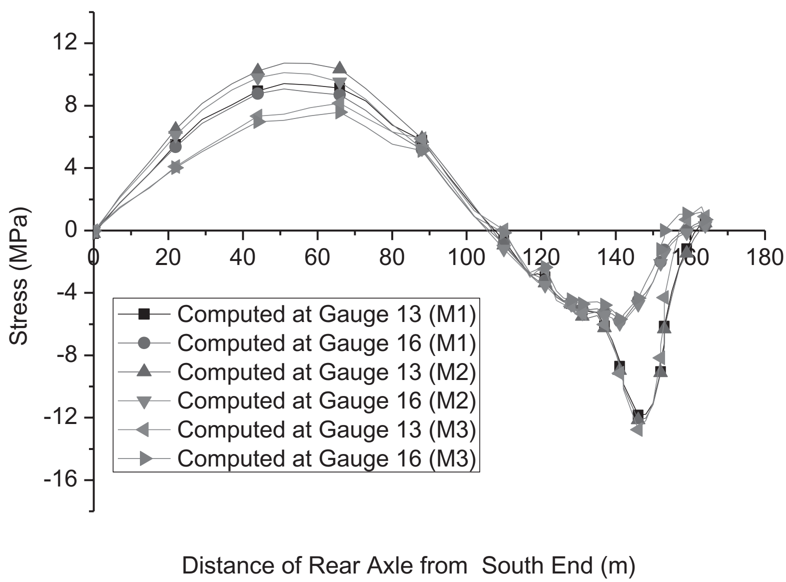


Figure 22. Comparison of computed stress history.

stress concentration because of the complexity of the connections between the stringer girders and the floorbeams. In all models, all of the old stringer bearings are assumed to be in the same state, but the parametric studies show that the effect on computed stresses is limited and can be neglected if some of those bearings have different constraints.

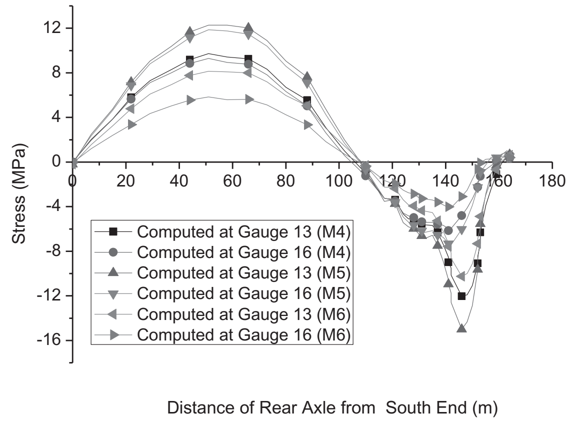


Figure 23. Comparison of computed stress history.

7 CONCLUSIONS

To determine the causes of cracks at connection angles, three field tests were conducted on the Chesapeake City Bridge to test its global and local behaviors. Global and local models were developed, and a parameter sensitivity analysis was carried out. Calibration of the analytical model and parameter identification is conducted based on the experimental results. The following conclusions can be drawn from the work:

1. The M5 is the best model to represent the bridge state, which considers effects from railings, deck connections and sheared-off bolts;
2. The north main bearings are fully constrained at all rotation degrees, while the south main bearings may be partially fixed in the longitudinal direction;
3. The global behavior stems from shear forces applied on stringer bearings, which may be caused by a combination of the railing, hardened rubber seals, and invisible connections under those rubber seals;
4. Experimental results show that there is not much difference before and after replacing stringer bearings. The numerical model results verify this conclusion;
5. The stress difference around the peak tension strain at Gauges 13 and 16 on Floorbeam L1' in the 2004 test is from sheared-off bolts, and
6. The floorbeams of the arch were not designed to withstand torque. Most longitudinal shear forces that cause torque are absorbed by the exterior stringer bearings. Numerical models confirm that the new exterior bearings on Floorbeams L1 and L1' withstand the largest shear forces, which is the reason the bolts were sheared off.

REFERENCES

Carden, E.P. & Fanning, P. 2004. Vibration based condition monitoring: a review. *Structural Health Monitoring,* Vol. 3(4): 355–377.

Doebling, S.W., Farrar, C.R. & Prime, M.B. 1998. A summary review of vibration-based damage identification methods. *The Shock and vibration digest*, 30(2): 91–105.

Liu, J. 2006. Understanding bridge performance through integrated modeling and monitoring. *Dissertation at the University of Delaware*.

Salawu, O.S. 1997. Detection of structural damage through changes in frequency: a review. *Engineering Structures*, Vol. 19, No. 9, pp. 718–723.

7 Bridge inspection, management and assessment

Chapter 25

Developing a business process model for bridge management in Europe

T. Browne
Collins Engineering Inc., Milwaukee, Wisconsin, USA

J. Harvey
Waterways Ireland, Enniskillen, Ireland

P. Owens
CEI Collins Engineering Ltd., Dublin, Ireland

ABSTRACT: Waterways Ireland is responsible for managing and maintaining Ireland's navigable inland waterways, as established under the British-Irish Agreement in December 1999. As part of this cross-border role, Waterways Ireland manages and maintains bridges and an array of other structures on its seven major waterways in order to fulfil their mission of providing a high quality recreational environment on the waterways in their care, for the benefit of all.

Currently, there are 580 structures spanning over the 1,000 kilometres of waterways and their associated branches. Of these 580 structures, 360 are owned, managed and maintained by Waterways Ireland.

Recently, Waterways Ireland initiated a programme to develop a Business Process Model for managing their bridges. This paper describes that Business Process Model, which is comprised of a set of logical diagrams, textual descriptions and data elements. These components provide both an overview and an in-depth guide to the procedures involved with managing Waterways Ireland's bridge stock, in a format understood by managers and personnel within Waterways Ireland.
The following components of the business process model are discussed within the paper:

- The set of hierarchical procedures and activities associated with managing bridge structures.
- Textual descriptions of each activity contained within the process procedures.
- The workflow diagrams illustrating the interrelating procedures and activities.
- The range of inputs and outputs associated with each activity together with a description of the personnel responsible for the undertaking of those activities.
- Key Performance indicators, which afford quantifiable measurements that reflect the operational efficiency of the various procedures within the Business Process Model.

1 INTRODUCTION

Waterways Ireland, as established under the British-Irish Agreement in December 1999, is responsible for managing and preserving Ireland's inland navigable waterways. As part of this cross-border role, Waterways Ireland manages and preserves bridges and an array of other structures on its seven major waterways in order to fulfil their mission of providing a high quality recreational environment on the waterways in their care, for the benefit of all.

Currently, there are 580 bridges spanning over the 1,000 kilometres of waterways and their associated branches (*1*). Of these 580 structures, 360 are owned, managed and maintained by Waterways Ireland. Figure 1 shows an overview of the Irish Inland Navigable Waterways.

Recently, Waterways Ireland initiated a program to develop a Business Process Model for its Bridge Preservation program. The context for the creation of the Business Process Model is set

Figure 1. Overview of Ireland's inland navigable waterways inventory and condition survey.

in Section 2.0 of this paper by giving an overview of the work undertaken by and on behalf of Waterways Ireland during the initial stages of the operation of the Bridge Management System. The Business Process Model, which is comprised of a set of logical diagrams, textual descriptions and various data elements, is detailed in Section 3.0 along with typical examples. The components of the Bridge Process Model provide both an overview and an in-depth guide to the procedures involved with managing Waterways Ireland's bridge stock, in a format understood by managers and personnel within Waterways Ireland.

2 WATERWAYS IRELAND'S BRIDGE MANAGEMENT SYSTEM (BMS)

Bridge management is the practice of caring for a bridge network from its conception to the end of its useful life cycle (*2*). The deterioration of bridges must be effectively managed in order to maintain functionality, value for money and to meet health and safety responsibilities.

The Waterways Ireland BMS has been introduced to harmonise and coordinate the multitude of activities involved in the management of bridges. The primary tasks associated with management of Waterways Ireland's bridge network include the collection of inventory data, the regular determination of structure condition and strength via inspection and assessment, the repair, preservation, rehabilitation and replacement of structures where necessary, the prioritisation and allocation of resources and the management of Waterways Ireland's health and safety responsibilities (*2*). The BMS aims to coordinate and assist in managing this abundance of tasks by way of a single Software System. This integrated and versatile BMS Software solution assists in the operation and preservation of bridges by providing access to the tools and guidelines required by all levels of Waterways Ireland staff, from Executive to Operational, in order to carry out their duties and to maintain the stock of bridges.

Before choosing to undertake the design of the BMS, Waterways Ireland researched a number of existing Bridge Management systems in Ireland, USA, UK and continental Europe. It was decided that a tailor made solution would suit best for a number of reasons. A tailor made BMS allows the flexibility to design the BMS around the Existing Waterways Ireland infrastructure and internal procedures, thus minimising disruption to the organisation. In addition, a customised BMS offers the ability to easily adapt and tweak the system once installed to take account of unforeseen tasks, or circumstances that may arise. Since many structures over the waterways are maintained by third party authorities, care was also taken to design the BMS so that structure information would be comparable with information from other Bridge Management Systems in Ireland.

The first step towards the implementation of Waterways Ireland's BMS involved definitively ascertaining the makeup of their entire bridge stock and undertaking an outline condition assessment of each structure. Although Waterways Ireland had previously retained details of many structures over the waterways, no complete inventory had been maintained. In addition, previous inspections of Waterways Ireland structures had been undertaken independently within each region, with no centralised record of structure conditions.

CEI Collins Engineers Ltd (Collins) was commissioned in autumn 2006 to commence the gathering of Inventory data and to undertake the initial visual condition assessment of each structure owned by Waterways Ireland. This site work was required to be completed in a timeframe of just three months. Collins also produced and maintains an interim Bridge Database to record and manage all collected data until the completion of Waterways Ireland's BMS Software in 2008.

Of the 580 structures spanning over the various inland waterways, 360 bridges are owned by Waterways Ireland, while 220 bridges are owned by a variety of other authorities. These 580 structures spanning over canals, rivers and roads are comprised of a range of bridge types including single and multi-span masonry arch bridges and aqueducts, steel and concrete beam bridges, steel truss bridges, passenger and freight railway bridges, masonry and concrete pipe culverts and many types of moveable and lifting bridges. Many of these structures are maintained by third party authorities such as the Department for Regional Development—Roads Service in Northern Ireland, the Irish National Roads Authority, Local Road Authorities, Irish Rail and

Bord na Móna (peat producers). The most common structure type over the navigable waterways is the single span masonry arch bridge, many of which were constructed circa 1800, and are most prevalent along the Royal and Grand Canals. All 580 structures underwent a general structure inventory. This inventory entailed the gathering of basic structure information such as the bridge name, bridge type, bridge location, route priority rating and the collection of standard bridge and bridge element photographs. The 360 structures owned and maintained by Waterways Ireland called for the collection of significantly more inventory data encompassing substructure and superstructure construction details, element material types and various geometric dimensions (span lengths, element dimensions, arch details, vertical clearances, channel depths, carriageway details, GPS information etc). In addition to inventory gathering, an initial condition inspection was undertaken at each structure. These inspections comprised of brief 'from the ground' visual inspections which aimed to provide Waterways Ireland with an appraisal of the condition of their bridge network. As part of the initial condition inspections, Collins adopted a bridge condition rating system ranging from 0 (Excellent condition) to 5 (Unsafe/Failed Condition). The creation of this rating system was influenced by a number of existing rating systems utilised in the USA, Ireland and the UK, as used by inspections undertaken by Collins throughout the world. Throughout the inspections, each structure element (abutments, piers, carriageway etc) was rated. Defect notes and photographs for each element were also recorded. These individual element ratings resulted in the production of an overall condition rating for each bridge. In addition to this overall condition rating, a route priority rating was also assigned to each structure (including structures owned by third party authorities). The route priority rating refers to the relative importance of a particular structure, taking account of how heavily trafficked the structure is, its location and its contribution to the functionality of the navigable waterways. The route priority rating system ranges from 1 to 5, with lower ratings typically being assigned to structures of high importance, such as heavily trafficked landmark structures and multi-span aqueducts. Higher ratings are generally assigned to structures of less importance, such as land drainage culverts and utility structures. The overall structure condition ratings and the route priority ratings are then inserted into a matrix to produce a Bridge Maintenance Rating (*3*). The Maintenance Rating Matrix can be seen in Table 1. The Bridge Maintenance Ratings allow Waterways Ireland to prioritise their resources in order to most efficiently carry out further bridge tasks including maintenance operations and Principal Inspections. Since the completion of this initial population of the interim database, Waterways Ireland have been managing the undertaking of Principal inspections, Load Assessments and repair works on many of their structures based on the initial Visual Condition Inspections.

Table 1. Maintenance rating matrix.

Bridge Priority / Condition Rating	1	2	3	4	5
5	1	2	3	4	5
4	2	3	4	5	6
3	3	4	5	6	7
2	4	5	6	7	8
1	5	6	7	8	9
0	10	10	10	10	10

2.1 *Principal inspections*

Principal Inspections involve a thorough visual examination of all elements of a bridge, including adjacent earthworks and waterways that may be relevant to the behaviour or stability of the structure. It is a close examination conducted with ladders and other access equipment, as necessary to typically be within touching distance of all bridge parts (*4*). The purpose of a Principal Inspection is to continually monitor the performance and condition of bridges. Detailed descriptions of the condition of all elements together with descriptions of all defects are recorded. A photographic record of all elements, typical conditions and structure defects is maintained. In addition, recommendations are made as to the need for repairs, additional investigation, and the interval until the next Principal Inspection. Cost estimates for recommended repair works are also included in Principal Inspection deliverables. Principal Inspections are typically carried out once each 6 years; however, this interval may be reduced depending on a number of factors.

Since 2007, Collins has been fulfilling Waterways Ireland's aim of completing sixty Principal Inspections per year. This program allows for all Waterways Ireland owned bridges to undergo a Principal Inspection in any given 6-year period, as required, so as not to exceed the maximum inspection interval.

2.2 *General inspections*

A General Inspection comprises of a visual inspection conducted at least once each two years by Waterways Ireland engineering personnel or sufficiently approved and trained operations personnel. The inspection includes all visible parts of the structure, including adjacent earthworks and waterways that may be relevant to the behaviour or stability of the structure. A General Inspection does not require access equipment since the inspection should be undertaken at ground level. Entry into confined spaces should not be undertaken during a General Inspection. Special testing equipment is not required; however simple inspection tools or aids such as measuring tapes or binoculars may be used. The general condition of the structure and the condition of individual components are determined by visual examination and recorded by means of a rating from "0" to "5," where "0" is Excellent and "5" is Failed or Unsafe. The type and extent of significant damage or deterioration is also noted and documented. As part of the inspection, recommendations are made as to the need for repairs, additional investigation in the form of a Special Inspection, and the interval until the next General Inspection is recommended.

The initial condition inspections as undertaken by Collins were similar in nature to a General Inspection and thus, the first round of General Inspections took place in late 2006. Currently, General Inspection training seminars are being implemented to facilitate the ongoing General Inspections by Waterways Ireland Staff. These ongoing inspections will begin following the implementation of the BMS software.

2.3 *Load assessments*

Every bridge capable of carrying motorised vehicles is assessed to determine its load carrying capacity. This assessment considers the capacity of the structure in its current condition as compared to the capacity of the structure as originally constructed. Consideration is given to the fact that many older canal type structures, often built in the late eighteenth century, were never designed to take the HA or HB loading specified today in the Irish Design Manual for Roads and Bridges. Structural assessments are generally conducted in conjunction with the initial Principal Inspection of a structure since accurate condition information is required to carry out a structural assessment. Should a subsequent Principal Inspection indicate significant changes in the condition of the structure, the bridge shall be re-evaluated considering those changes.

Many load Assessments have been carried out for Waterways Ireland structures in conjunction with the Principal Inspections. The compilation of detailed Principal Inspection Results and Load Assessment details together with the previously gathered condition assessment data and comprehensive inventory data results in the basis for Waterways Ireland's Bridge Management System.

2.4 *Special inspections*

Special Inspections involve an in-depth examination of a particular area of a bridge which may be causing concern and therefore, are not carried out on a scheduled basis. Special Inspections often arise as a result of a Principal Inspection recommendation for further investigation or due to a need for specialised inspection equipment or expertise not typically available during a Principal Inspection. Special Inspections may include specialist inspection methods, equipment, expertise, destructive or non-destructive testing. Typical examples of Special Inspections include underwater inspections of substructure components, underwater scour inspections, ultrasonic thickness surveys of steel defect areas or inspections of concrete post-tensioning systems.

Waterways Ireland maintains a number of aqueduct structures spanning roadways, rivers and railway lines across the country. Collins has undertaken a number of Special Inspections of these aqueducts investigating leakage through the arch barrels from the canal above and also undertaking underwater inspection of substructure units to document any potential scouring of the channel bottom surrounding structure piers and abutments.

3 WATERWAYS IRELAND'S BUSINESS PROCESS MODEL

In order for a Bridge Management System to function correctly and efficiently, it must harmonise with the existing organisational structure of the authority adopting it. Clear guidelines must exist as to what activities are required to be carried out, who should undertake those tasks, when they should take place and what resources are to be utilised. A collection of processes, textual descriptions and data constituents that describe the hierarchy and organisation of tasks and activities to be undertaken is called a Business Process Model (*5*). In essence, a Business Process Model provides a detailed blueprint to show the order of activities required to complete a Procedure. In turn, that procedure may often be a small step in another overall process.

A Business Process Model for a Bridge Management System organises and describes each of the activities that are undertaken by an authority responsible for a bridge network, in a format that can be understood by all personnel within the authority. Collins Engineers, in partnership with Waterways Ireland, have developed such a Business Process Model to enable Waterways Ireland to successfully implement their BMS.

3.1 *Organization of process procedures*

Waterways Ireland's Business Process Model for their BMS is partitioned into two interlinking sections; one portion deals with the administration of inspection procedures, while the other relates to the management of bridge works. Two overall procedures organise the primary activities involved with these two headings and are named, 'The Overall Inspection Procedure' and 'The Overall Work Management Procedure'. While these overall procedures are distinct from one another, they are closely related, with numerous interlinking tasks and activities. The Overall Inspection Procedure Chart is shown in Figure 2.

Within each of these primary partitions, several sub-processes exist which deal with the principal BMS tasks, such as the 'New Structure Procedure', 'Principal Inspection Procedure', 'Emergency Works Procedure' and the 'Repair/Maintenance Procedure'. Again, many of these process procedures are closely inter-related, with one procedure often arising out of another. For example, the 'Repair/Maintenance Procedure' may often be undertaken as a result of a previously conducted 'Principal inspection Procedure' since repairs are often recommended following a Principal Inspection. These sub-processes are then further sub-divided, where necessary, down to the required level of detail.

The procedures are represented graphically by logical process charts, a commonly used managerial tool. A logical flow from one activity to the next is illustrated, with the overall procedure tending towards completion as each task is carried out. The individual process activities typically fall into one of five categories—Process Activities, Pre-defined Process Activities, Process

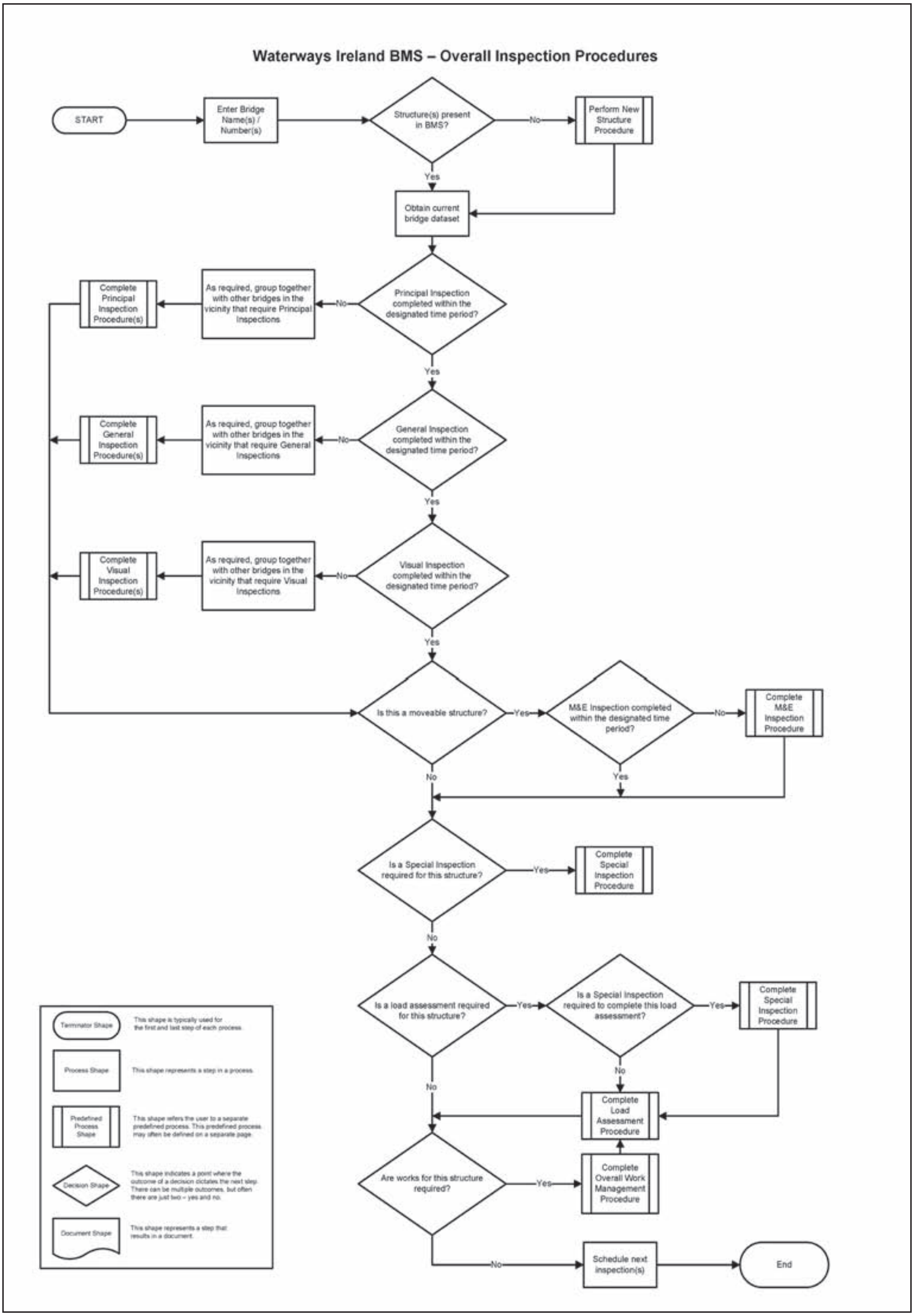


Figure 2. Overall inspection procedure chart.

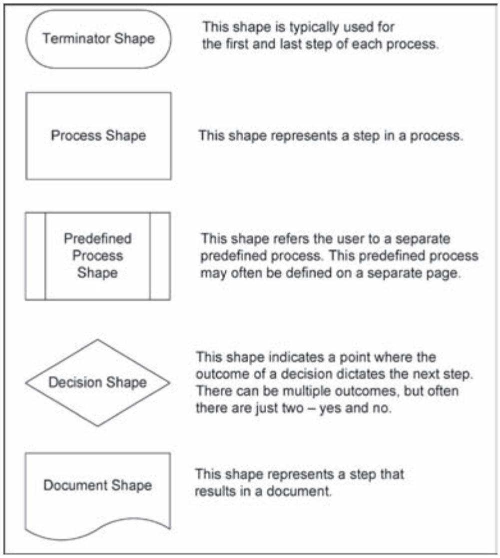


Figure 3. Process charts activity shapes (*5*).

Decisions, Document Activities and Terminators (*5*). These five procedural activities are each illustrated differently on the logical process charts using specific activity shapes. Examples and descriptions of these activity shapes are outlined in Figure 3.

3.2 *Activity textual descriptions*

Each chart is accompanied by a manual of textual descriptions. A textual description of each individual process activity explains that activity in detail. These textual elements of the Process Model typically conform to a standard template in order to present data in a complete and consistent form. The consistency of presentation encourages an unambiguous comprehension of all activities associated with the process model. The textual descriptions are generally one page long and list activity data such as a general description of the activity, the purpose of the task, the inputs and outputs associated with that activity, typical duration of the activity and also details of any record that should be maintained concerning that task. Refer to section 3.5 for an example of a textual description.

3.3 *Inputs and outputs*

This essential component of the textual descriptions details the information flowing in and out of each activity. Since each activity is part of a chain of tasks, information resulting from the previous activity is often required to undertake the task and/or the current activity results in the production of new or updated information to facilitate the next activity in the procedure. For example, a step in the 'Principal Inspection Procedure' may direct the reader to determine if a required inspection should be undertaken in-house or if the works should be carried out by an external consulting engineering firm. The inputs for this decision activity may include a multitude of items such as structure name, structure number, structure type, structure location, current structure condition information, available funding, availability of inspection staff etc. The outputs are either 'Yes, the inspection can be undertaken by Waterways Ireland Staff' or 'No, the inspection cannot be undertaken by Waterways Ireland Staff and must be carried out by an external consulting engineering firm'. By carrying out this decision step or activity, the procedure can progress to the next step to allow the inspection to be carried out. The provision of potential inputs and outputs for each activity in the textual descriptions manual enables the user of the procedure to be more aware of what information should be available to carry out the activity in question and also what information is required from the current step to progress to the next task within the procedure.

3.4 *Key performance indicators*

Key performance indicators (KPI's) are management tools used to measure the effectiveness of processes and procedures within the Business Process Model (*5*). In order to be effective, KPI's, as the name suggests, must measure key, meaningful parameters. They must relate to a performance that is easily measurable and quantifiable and they must give a meaningful indication of current performance. They should provide staff with realistic achievable goals towards which they can work.

The 'Principal Inspection Completion Efficiency' KPI is an example used in the Waterways Ireland Process Models. Details of this KPI, as set out in the Business Process Model are described in Figure 4.

This performance indicator gives a measure of how many Principal Inspections were completed in a given time period as a percentage of the total number of Principal Inspections, which were scheduled to take place in that same time period. It is a gauge of how well Waterways Ireland is meeting its targets in terms of completed Principal Inspections. With respect to this KPI, the aim may be to achieve a KPI value of at least 85%. A KPI value of less than 85% may indicate that an insufficient amount of Principal Inspections have been carried out in the given time period, however, it may also indicate that the initial target for the time period was set too high. Conversely, a KPI value of greater than 110% may indicate that a more than sufficient number of Principal Inspections have been carried out in the given time period, however, it may also indicate that the initial target for the time period was set too low.

In order to improve the Principal Inspection Completion Efficiency KPI, it is necessary to initially forecast a realistic goal of Principal Inspections to be completed. This goal may call for a slightly greater number of inspections to be carried out in this time period compared with the last time period (assuming equal period lengths), thus encouraging a development in productivity.

3.5 *An example—Principal inspection procedure*

Principal Inspections are a critical portion of the Waterways Ireland Bridge Management System. As such, the Principal Inspection Procedure, as part of this Waterways Ireland Business Process Model is vital to the management of the bridge stock. The Principal Inspection Procedure Chart can be seen in Figure 5.

The Principal Inspection Procedure begins by instructing the user to identify the structure to be inspected and to subsequently obtain the existing dataset for that structure. Note that this procedure will typically be carried out for a tranche of structures; however for the purposes of this example we shall assume that only one bridge is undergoing a Principal Inspection. This required dataset is maintained by the administrator of the BMS and typically comprises of previous inspection reports, drawings or other structure information.

The user must then determine if any particular equipment or expertise is required to carry out the inspection and that any associated risk assessments have been carried out for the inspection. For example confined space training may be required to complete the inspection. This may then affect the next step where the user must decide whether the inspection can be undertaken by Waterways Ireland staff or an external engineering consultancy.

The outcome of this decision step significantly affects the progression of the procedure, as can be seen from the chart. However, many of the activities involved with undertaking the inspection are still the same. Should it be decided that an external consulting engineer will be utilised, the user must establish whether there is an existing consulting engineer already under contract for similar works, and if they wish to use the same firm for the current inspections. Depending on that decision, the Consulting Engineer Tender Procedure may be required.

Waterways Ireland
Uiscebhealaí Éireann Watterweys Airlann

Bridge Management System

KEY PERFORMANCE INDICATORS

COLLINS ENGINEERS

Title:	**Principal Inspection Completion Efficiency**
KPI Hierarchy:	Top Level KPI
KPI Formula:	$\dfrac{\textit{No. of Principal Inspections Completed in any Given Period} \times 100}{\textit{No. of Principal Inspections Scheduled in any Given Period}}$
KPI Description:	This performance indicator gives a measure of how many Principal Inspections were completed in a given time period as a percentage of the total number of Principal Inspections which were scheduled to take place in that same time period. It is a gauge of how well the Structures Department is meeting its goals in terms of completed Principal Inspections.
Inputs:	There are 2 inputs required in order to complete this KPI. They are the total number of fully completed Principal Inspections in the given time period and the total number of scheduled Principal Inspections in that same given time period.
Outputs:	A measure of how many Principal Inspections have been completed as scheduled, in the form of a percentage.
Analysing KPI Results:	This KPI gives the Structures Department an overview of how well they are attaining their targets with respect to the completion of Principal Inspections. The aim of the Structures Department should be to achieve a KPI value of at least 85%. A KPI value of less than 85% may indicate that an insufficient amount of Principal Inspections have been carried out in the given time period, however, it may also indicate that the initial target for the time period was set too high. Conversely, a KPI value of greater than 110% may indicate that a more than sufficient number of Principal Inspections have been carried out in the given time period, however, it may also indicate that the initial target for the time period was set too low.
Aims:	The aim of the Structures Department should be to achieve a Principal Inspection Completion Efficiency KPI value of at least 85%.
Improvement Methods:	In order to improve the Principal Inspection Completion Efficiency KPI, it is necessary to initially forecast a realistic goal of Principal Inspections to be completed. Typically, this goal should call for a slightly greater number of inspections to be carried out in this time period compared with the last time period (assuming equal time periods). Waterways Ireland should always endeavour to carry out Principal Inspections as efficiently as possible within the given time and financial constraints.

Figure 4. KPI information sheet.

The Principal Inspection Procedure then progresses through a number of steps involved with the undertaking of the Principal Inspection and the submittal of the agreed deliverables in the prescribed format, including a quality control process associated with those submissions. Upon completion of the inspection and submission of the deliverables, the inspection data is added to the Bridge Management System by the BMS Administrator. Note from the chart that this activity,

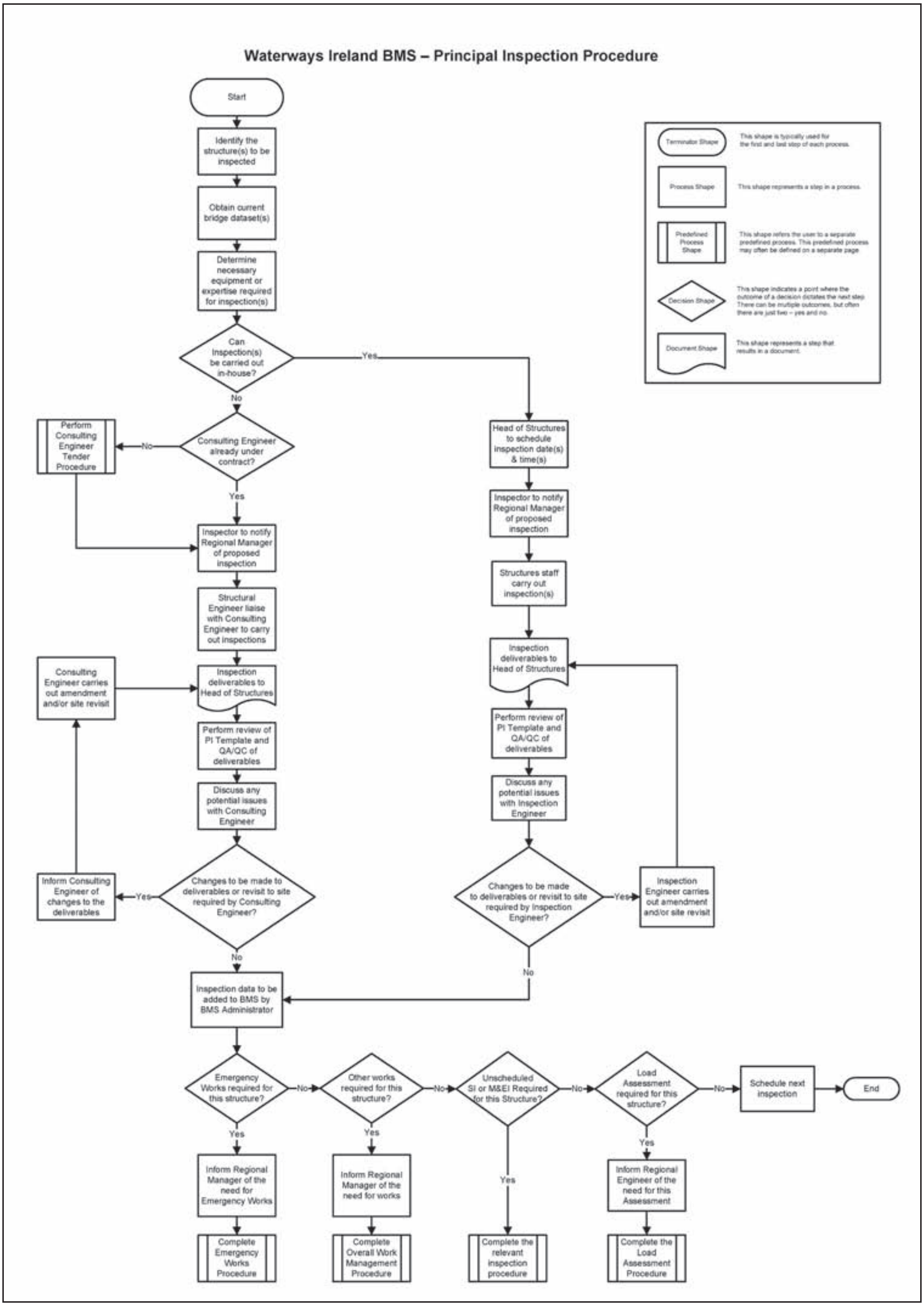


Figure 5. Principal inspection procedure.

Bridge Management System

Waterways Ireland
Uiscebhealaí Éireann Watterweys Airlann

COLLINS ENGINEERS

Title:	**Determine necessary equipment or expertise required for inspection(s)**
Hierarchical Location:	Low Level Activity.
General Description:	Deciding what, if any, equipment is required to carry out the Principal Inspection.
Purpose:	Before going on site, the Inspection Engineer must determine if any particular equipment or expertise is required to carry out the inspection. From previous experience, this information will typically be available from the BMS, however, should conditions have changed since the last inspection or should the structure be new, then the Inspector will have to ascertain any equipment requirements. The Inspection Engineer must also satisfy himself that any associated risk assessments have been carried out for the inspection and that all risk assessments incorporate the required inspection equipment. All risk assessments must be up to date and be applicable to the inspection due to take place.
Duration:	Minimal.
Inputs:	Bridge name(s) or identification number(s).
Outputs:	Any required inspection equipment and confirmation that all Risk Assessments are applicable and up to date.
Required Resources:	Any member of the Waterways Ireland Structures Department Staff who is familiar with the Bridge Management System and Principal Inspection requirements.
Record Keeping:	Any required inspection equipment should be recorded in the Principal Inspection Template for entry into the BMS. This will allow future Inspection Engineers to be aware of any required inspection equipment. Any updated Risk Assessments should also be recorded in the BMS for future use.

Figure 6. Sample textual description.

which comprises of the final addition of the inspection data to the BMS, is arrived at regardless of whether the Principal Inspection was carried out in-house or not.

Following the completion of the inspection and the entry of the inspection data into the BMS, a number of resulting actions may be required. The first possible outcome of the inspection may necessitate the need for emergency work to be carried out at the structure. If this is the case, the applicable Regional Manager will be informed and the Emergency Works Procedure will be carried out. Similarly, the Principal Inspection may require other structure works, further inspection in the form of a Special or Mechanical and Electrical Inspection, or a Load Assessment to be undertaken. Again, should these be required the corresponding procedure will be undertaken.

Finally, the time interval until the next inspection must be assigned by the engineer. This interval is typically 6 years, although this may reduce if necessary but never intentionally increase beyond 6 years.

For each of the 35 activities shown in this Principal Inspection Procedure, there is a corresponding set of textual descriptions, which sets out the attributes of that activity. An example of a Business Process Model textual description can be seen in Figure 6.

4 CONCLUSIONS

The development of this Business Process Model allows Waterways Ireland to organise the multitude of tasks and activities associated with a Bridge Management system in order to undertake the effective management of their bridge stock.

The Business Process Model provides an at-a-glance overview of what activities Waterways Ireland has undertaken, is currently carrying out and is planning to do as part of their BMS. It also acts as a reference manual for all levels of Waterways Ireland staff involved with bridge preservation, to allow them to identify how their individual roles can facilitate the business as a whole.

Currently, there are a total of 2 overall process procedures and 12 sub-process procedures associated with the Waterways Ireland Business Process Model, each containing between 4 and 71 activities. In turn, each of these activities typically has a one A4 page textual description, similar to Figure 3,5, resulting in a document of over 320 pages.

Waterways Ireland is continuing with the implementation of its Business Process Model for Bridge Management. The BMS software will be fully implemented in 2009 ready for the inclusion of all existing inventory, inspection and assessment data from the interim database. Following the implementation of the Bridge Management Software, Waterways Ireland will undertake the regular Visual and General Inspections of their structures in addition to the ongoing necessary structure repairs, carrying out structure load assessments and continuing to see all Principal Inspections carried out within the 6 year cycle.

While Waterways Ireland has been in the advantageous position of being able to design and implement the Business Process Model and Bridge Management System software simultaneously, this is by no means a necessity. Many authorities of varying magnitudes, in both Europe and the US, currently have bridge management software in place to aid in bridge preservation. The implementation of a Business Process Model to clearly describe and organise their existing bridge management tasks can provide the capacity to streamline bridge stock preservation and increase efficiency. In addition, the process of creating a Business Process Model for an existing Bridge Preservation Program places a focus on all aspects of the active program. This then highlights inefficiencies and encourages positive change which can be implemented with the Bridge Preservation Program Process Model.

REFERENCES

Alec Sharp, Patrick McDermott. *Workflow Modeling—Tools for Process Improvement and Application Development.* Artech House, Inc. Massachusetts, 2001.

Highways Agency. *Inspection Manual for Highway Structures.* TSO (The Stationary Office), London, May 2007.

Waterways Ireland. *Invitation to Tender for a Bridge Management System, Version 1.0.* December 2006.

Waterways Ireland. *Invitation to Tender for Survey Services in support of Waterways Ireland Bridge Management System, Section 4.0.* August 2006.

Waterways Ireland. *Navigating Ireland's Inland Waterways.* Received February 2008 from: http://www.waterwaysireland.org/index.cfm/section/publications/article/52/page/NavigationInformation

Chapter 26

Use of public-private partnerships in bridge infrastructure delivery

J. Chang
Columbia University & MTA Bridges & Tunnels, New York, New York, USA

ABSTRACT: With federal budget deficits on the order of $1 trillion, it is unrealistic to expect government financing to appreciably close the infrastructure funding gap. Although private capital is available to this end, there remains considerable resistance in the U.S. to the use of private investment to finance public infrastructure projects despite its general acceptance in other parts of the world. This is unfortunate, since there is no overriding rationale for exclusive government involvement in infrastructure provision. Properly applied and structured, Public-Private Partnerships (PPP) can offer significant advantages to a traditional approach, particularly in the delivery of large-scale bridge projects, as illustrated in the case of the Confederation Bridge. This paper concludes by discussing the contemplated use of PPP to replace a number of major bridge crossings in New York. While no panacea, public-private partnerships have an important role to play in any comprehensive strategy to renew the nation's infrastructure.

1 INTRODUCTION

There is a generally acknowledged funding gap for civil infrastructure projects in the United States that will be only partially filled by the much-touted federal stimulus package. While the stimulus allocates about $27.5 B to roads and bridges (Ichniowski and Hunter, 2009) this represents just a fraction of the $122.2 B annual gap for roads and bridges determined by ASCE in its most recent *Report Card for America's Infrastructure* (ASCE, 2009). One way to narrow this gap is through the use of private investment in so-called public-private partnerships (PPP). The idea is not a new one, and examples of this approach abound throughout both World as well as U.S. history. The use of private financing has regained general acceptance in other parts of the world, with some 1,100 public-private deals world-wide in the last 20 years (Malanga, 2007). Nevertheless, there is considerable resistance in the U.S. to the use of PPP that typically centers around two arguments. Opponents first argue that as a public good infrastructure belongs in the public realm. The second argument is that the government can finance infrastructure projects more cheaply than the private sector since it alone can issue tax-exempt debt. However, as this paper will argue, such arguments fail to consider a number of critical points, particularly in the case of bridge infrastructure. First, bridges have the characteristics of a private rather than a public good, so in economic terms there is no overriding rationale for exclusive government control. More importantly, the real potential offered by public-private partnerships lies in its power to innovate, as this paper illustrates using the Confederation Bridge as an example. Potential PPP applications include the planned replacement for the Tappan-Zee Bridge, a project that currently lacks a sufficient direct funding source. While no panacea, public-private partnerships have an important role to play in any comprehensive strategy to renew the nation's infrastructure.

1.1 *Infrastructure & public-private partnerships*

Infrastructure and *public-private partnerships* are two terms in wide use of late, particularly in the wake of the current economic and financial crisis. Indeed, one of the central features of the Obama Administration's plan to restore a functional financial system involves a public-private partnership; under this plan, both public and private funds would be used to purchase the toxic assets that are crippling the nation's banks. While widely employed, the terms in fact defy easy characterization; Garvin (2007) offers six definitions of the term infrastructure, each with different connotations. It is therefore advantageous to propose at the outset working definitions that will be useful in the discussion that follows. Accordingly, Garvin (2007) offers a broad definition of infrastructure as "the physical assets that facilitate the delivery of both social and economic services." Next, Garvin and Bosso (2008) define PPP with respect to infrastructure as a "long-term contractual arrangement between the public and private sectors where mutual benefits are sought and where ultimately (a) the private sector provides management and operating services and/or (b) puts private finance at risk." Underlying this approach is the idea that each sector has its inherent strengths with infrastructure as an area of mutual interest and overlap. The public sector can for example identify project needs and regional interests while the private sector can contribute efficiencies, innovation and an alternative source of financing (Garvin, 2007).

PPP's can be used both for the maintenance & operations of existing ("brownfield") facilities and for the development of new ("greenfield") infrastructure projects. One PPP approach that has gained some traction in the U.S. concerns the long-term lease of existing infrastructure assets to private operators. In 2005 the city of Chicago finalized a deal to lease the 7.8 mile long Chicago Skyway to the joint venture team of Macquarie & Cintra for a period of 99 years. In return for an upfront payment of $1.83 billion the joint venture assumed responsibility for maintenance and operations and the right to collect tolls over this time period. This deal was followed in 2006 by the 75 year lease of the 157 mile long Indiana Toll Road for $3.8 billion to the same joint venture. More recently, attempts to secure long–term leases for the New Jersey Turnpike, Garden State Parkway, and Pennsylvania Turnpike have failed, primarily due to concerns over foreign ownership and excessively higher tolls.

There are two general PPP approaches to the development of greenfield projects that differ primarily in the financing source. Under design/build/operate (DBO), the owner selects a single contractor to design, construct, and operate a facility for a given length of time. In this case the owner is responsible for financing the project. The design/build/finance/operate method is similar to a DBO except that here the contractor is responsible for financing the project based on the expectation of future user fees.

1.2 *Historical examples*

There is a long history of private development of public infrastructure. One interesting example cited by Samuel (2005) is the Old London Bridge (Figure 1), which first opened in 1209. The bridge was built under a toll concession granted by King Henry II to Peter of Colechurch, who was

Figure 1. Old London Bridge.

an officer in the Church of St. Mary Colechurch. As Samuel (2005) notes, the arrangement would today be categorized as a public-private partnership between the King and this church to finance and build the bridge in return for right to collect tolls on it. The bridge also proved to be a significant source of non-toll revenues. Stores & residences were built atop it, the bridge sponsored jousts & entertainment, and even collected fees from fishermen who used it.

In addition, there is a long history in the U.S. of successful private investment in roads and bridges. In the period from 1789 to 1933 about 62% of all American infrastructure was provided with the aid of private investment. An example from this period is the postal and road system, built and operated almost exclusively through private sector initiative in what would now be referred to as a PPP (Miller, 2000).

1.3 *Bridging the gap*

A great deal has been written on the insufficiency of current government spending to pay for current and future infrastructure needs (Ortiz & Buxbaum, 2008), (Page et al., 2008). While the attention that President Obama has given to infrastructure investment is welcome news to many, it is questionable whether the new focus will lead to markedly increased federal spending on infrastructure given the current financial and economic crisis. The estimates that U.S. public debt-to-GDP could reach 60% over the next few years to its highest level since the aftermath of World War II is indeed sobering (Wall Street Journal, 2009). States are also hampered by budget woes and a constitutional ceiling on how much they can borrow. So the potential clearly exists for the investment of private capital to finance infrastructure projects. Page et al. (2008) cite estimates of anywhere from $80 billion to $130 billion in private equity investment funds. Certainly in the near term the picture is clouded by the ongoing crisis. This has created borrowing difficulties on the one hand, but the collapse of securitized bond market also means there are fewer investment opportunities, perhaps enhancing infrastructure's appeal. Pension funds and insurance companies in particular invest a portion of their funds in these so-called alternative investments in order to diversify their portfolios; the major California public teachers and public employees' pension funds allocate 6% and 10%, respectively, into private equity investments (Page et al., 2008).

2 BRIDGE INFRASTRUCTURE AS A PUBLIC-PRIVATE GOOD

2.1 *PPP concerns*

There is considerable resistance in the United States to the use of private investment in financing public infrastructure projects that has only stiffened in the wake of the perceived market failures that led to the current financial crisis. One only has to read columnists like Thomas Frank and his derision of PPP as "privatopia" (Frank, 2009) to appreciate the degree of opposition. Clearly the principal concerns are that private operation will lead to significantly higher tolls coupled with poorer service, since the expectation is that the private operator would shortchange maintenance and repair in order to maximize profits. Opponents can also argue that these higher tolls are inequitable, disproportionately affecting those who are least able to pay them. There is also the belief that public tolls would be lower since the public has lower borrowing costs through its issuance of tax-exempt debt. Another issue concerns the status of the existing work force under the newly privatized operation.

Malanga (2007) and Samuel (2005) offer some specific responses to these arguments. To briefly summarize, the counterargument would be that private operation first of all leads to efficiencies that will improve service and hold down costs. For example, one of the first changes made by the private operators of the Chicago Skyway was the installation of an electronic toll collection system. Toll increases can be regulated by tying them to indices such as the Consumer Price Index. Also, concession agreements contain strict operating and maintenance contracts that can be cancelled in the event of poor performance, allowing the governments to take back the road or bridge.

Rather than revisiting familiar ground, the objective here is to first of all challenge the notion that infrastructure is somehow exclusively public by pointing out that infrastructure, in economic terms, is a public-private good. Next, integrated public-private approaches provide ample room for innovation, particularly in the delivery of major bridge projects. The Confederation Bridge illustrates this point.

2.2 *Public vs. private good*

In economic terms, a public good is one that exhibits two attributes. First, once the good is available it is impossible to deny its use to anyone who has not paid for it (non-excludable). Second, a public good is non-rivalrous, that is, any number of people can consume the good at the same time without diminishing the amount of that good available for anyone else to consume. A classic example is a system of national defense, and it is clear why government provision is necessary in such cases, since there would be no market for these goods in the private sector. A private good by contrast is both excludable as well as rivalrous.

It is also clear that roads and bridges do not fit neatly within the bounds of a public good. Instead, as show in Table 1, they have the attributes of a tolled good; they are certainly excludable, as evidenced by the numerous government-controlled toll facilities. In periods of growing congestion they are also rivalrous, as each additional car reduces the available amount of roadway for the car alongside it and take on the attributes of a private good. Since roads and bridges share these characteristics of a private good it is difficult to see how they are inherently public and hence require exclusive government involvement.

2.3 *Innovation potential*

Ultimately, one of the real benefits of private investment lies in the capacity for innovation. In principle, there are clear advantages to the DBFO approach. Of course there is a financial incentive to innovate since the developer has "skin in the game" and will seek to increase her return on equity. Additionally, the designer is working from the outset in concert with the contractor and the operator. As a result, the design can among other things incorporate new materials and utilize efficient construction techniques. Moreover, given the firm's long-term responsibility, the design will necessarily account for future operational and maintenance needs. There is the potential as well as the incentive to introduce new technologies, as demonstrated in the case of the SR 91 Express Lanes, delivered under the DBFO approach (Garvin, 2007). The project involved the construction of four express lanes in the existing median strip of a 10 mile section of SR-91 in Southern California. As the first privately financed toll road in US in the 20th century, this project also has the distinction of being the first toll road in the world without toll booths; making full use of then-new electronic toll collection. It was also the first toll road in world to use "value pricing" to manage traffic flow.

Tawiah & Russell (2008) conducted research on infrastructure projects seeking a link between the delivery method and innovation potential. They found that a complete integration of all project functions (i.e. DBFO) is a high driver for innovation. Other characteristics that promote innovation include large-scale heavy civil infrastructure projects with large resource requirements and costs in excess of $100 M. One project that entails all these features and supports the key findings is the Confederation Bridge.

Table 1. Public vs. private good.

	Rivalrous	Non-Rivalrous
Excludable	Private	Tolled
NonExcludable	Common Pool	Public

3 CONFEDERATION BRIDGE

3.1 *Background*

Prince Edward Island (PEI) is the smallest of the ten provinces of Canada in both land area and population, with about 135,000 residents. When Canada was created in 1867, PEI initially balked at joining, electing to remain part of Great Britain and Ireland. At the same time, PEI was being courted by the U.S., and Canada was anxious to thwart U.S. expansionism. So when PEI did become part of the Canadian confederation in 1873 it was able to secure several concessions from the federal government. One of them was for the federal government to provide a continuous and year round transportation facility for goods, services and people between the island and the mainland. The government fulfilled this responsibility by operating subsidized ferry services. However, ferry service had proven unreliable during the tourist season as well as in winter, when the ferry could be forced out of service due to floating ice or high winds in the straits. Moreover, the ferry was losing money and required increasing government subsidies; by 1992 the government was spending approximately C$42 M a year subsidizing the ferry service. Of greater concern was the fact that the subsidies were rising at a rate approximately 15–20% higher than the Consumer Price Index (CPI) (Miller, 2002). Given the increasing unreliability and escalating costs associated with the ferry service, the government settled on plans for a fixed crossing between PEI and the mainland.

3.2 *Project delivery*

After a three-stage procurement process the consortium Strait Crossing Development, Inc. (SCDI) was ultimately selected to design, build and operate a fixed link crossing for 35 years. In return, the consortium was granted the right to collect user tolls as well as an annual subsidy equivalent to C$41.9 M (in 1992 dollars) from the Government of Canada, starting at the end of May 1997, whether the fixed link was completed or not. If the bridge was not complete by that time, ferry services would continue to operate at SCDI's expense. The project was financed by a combination of debt and equity, and the government limited the toll increase in any year to 75% of the CPI.

3.3 *Project description*

The Confederation Bridge (Figure 2) is a two-lane, 8-mile fixed link over the Northumberland Strait, connecting Prince Edward Island with mainland New Brunswick, Canada. It is the longest span ever built over ice-covered water. There are basically two bridge types, the main spans and the approach spans. The bridge includes precast, concrete, balanced cantilever spans alternating with drop-in spans for the 11-km main bridge. The drop-in span contains a continuous connection between cantilevered tips; creating a two-pier portal frame. The two approaches, totaling 1,880 m in shallow water, are designed as precast segmental balanced cantilevers. The main bridge is comprised of a precast, reinforced post-tensioned concrete box girder superstructure sitting on precast reinforced concrete piers that are taken to bedrock. The typical span length is 820 ft (250 m) and the typical elevation above water is 131 ft (40 m).

3.4 *Project innovations*

PEI has a reputation as a province of great natural beauty, bountiful fishing and frigid, icy winters. Environmental considerations consequently played a large role in project planning and delivery. Because of the harsh northern climate, the designers were challenged to maximize the time available for component fabrication and to minimize the time required for marine erection. Both design and construction methods were developed with an environmentally friendly bridge in mind. For example, the design specifications eliminated causeways as an option due to their negative environmental effects (Miller, 2002). Given these sensitive conditions there was a need for extremely

Figure 2. Confederation Bridge.

durable high-grade concrete and reinforcing steel. Also, due to tight scheduling constraints, construction was only possible for 34 weeks of the year.

The project is notable for a series of innovative design & construction solutions. First, the bridge was designed for an unusual 100 year service life. Second, a specific concern was the effect of ice on the structure, so a principal design problem was how to make sure the bridge could withstand mats of broken ice that flow down the strait. Environmentalists and fisherman alike were concerned that delays in the breaking up and flowing away of winter ice could significantly alter the wildlife in the strait. In response, conical ice shields were developed on all the piers at water level that lift the ice and cause it to break. Piers were also spaced at 820-ft (250-meter) intervals in order to minimize ice blockage of the strait. The extreme weather and long service life led to the development of new glues that could be used in sub-freezing temperatures during fabrication of main girders, extending the time in which casting could take place. New techniques were employed for placing and grouting the main columns on the sea floor, and extensive use was made of Global Positioning Systems, allowing for very precise placement and aligning of all components of bridge, including at the sea bottom (Miller, 2002). Innovations were not limited to products and processes. Tawiah and Russell (2008) report on organizational innovations that in their view led to transparency and capital market confidence resulting in the issuance of the largest real-rate project bonds in Canada. With numerous awards for engineering excellence and design innovation and an opening that was six months ahead of schedule the project may in many ways be judged a success.

4 POTENTIAL APPLICATIONS

4.1 *Bridges in New York*

The preceding discussion grows increasingly relevant when the focus shifts to the future plans for major bridges in the New York Region that are being considered for replacement. Both New York State as well as the Port Authority (PA) are contemplating the use of PPP. Last October Governor David Paterson formed the Commission on State Asset Maximization to evaluate variants of the PPP model for a wide range of asset classes. The preliminary report (2009) accepts in principle the use of public-private partnerships while the final report will analyze its use on specific projects. One project mentioned in the report concerns the planned replacement of the Tappan-Zee Bridge. The full Tappan Zee Bridge Corridor would include a replacement bridge and bus rapid transit and commuter rapid transit services and is currently estimated at \$16.2B. In addition, the PA is studying PPP as an option for replacing the three major crossings between New York's Staten Island and New Jersey (Phillips, 2009). The Outerbridge Crossing, Goethals, and Bayonne Bridges, with combined annual traffic of 33.5 million vehicles in 2006, were built between 1928 and 1931 and are all nearing the end of their useful lives.

4.1.1 *Some considerations*

Certainly a decision to pursue PPP in these cases will rest on a host of considerations, politics not least among them. Nevertheless, three factors worth highlighting at this point are: prior institutional PPP experience, institutional borrowing capacity, and market demand risks. First, current New York State law does not allow the pursuit of alternate delivery methods like PPP; so enabling legislation must be the first step. However, the PA is not bound by the same set of laws and regulations and in fact has successfully used PPP on a few large scale infrastructure projects. One project is Terminal 4, an airport terminal at JFK International Airport that opened for service in 2001. The project, delivered under DBFO, represents the largest privatization for an airport terminal and is the first airport terminal in US to be managed by a foreign airport operator. A second example is the JFK Airtrain, an 8.1 mile rail system that operates between the airport's terminals and its parking areas while connecting with existing transit systems at Jamaica and Howard Beach in Queens. The system began service in 2003 and was delivered using a design-build-operate strategy with an initial five-year operating period followed by renewable one-year options.

Limits on borrowing capacity are a second factor. The PA in particular has a number of large-scale capital commitments, including the Hudson River Rail Tunnel that will channel freight traffic between New York and New Jersey. It also has numerous responsibilities at the World Trade Center site that only seem to increase over time-the PA now plans (Rivera, 2009) to guarantee debt used to finance one of the three office towers under development. These commitments may limit further PA borrowing, a development that would favor the use of PPP to replace the Staten Island Bridges.

A third factor concerns project economics. A principal risk in toll road development is whether the expected traffic volumes will in fact materialize. For example, the Dulles Greenway is a tolled extension from Dulles International Airport to Leesburg, Virginia and was one of the first PPP projects developed in the twentieth century in the U.S. The greenfield project was in financial distress from the outset, as it drew only 10,500 vehicles a day on average, well below the estimated 34,000 vehicles per day (Garvin, 2007). Yet there are notable differences between development roads like the Dulles Greenway and the projects being discussed that involve the replacement of existing structures. These types of projects tend to have strong economics due to established toll revenues and traffic patterns and volumes (Fisher & Babbar, 1996). This predictability is sure to attract private sector interest.

5 CONCLUSIONS

Increasingly evident is the idea that different circumstances call for different delivery methods. There are however a number of hurdles to overcome before PPP truly secures a foothold as a viable option in the U.S., not least of which is an aversion to "foreign" ownership. Another concern is windfall profits; although as shown in the case of the Confederation Bridge, this is a problem that can be mitigated by tying toll increases to indices such as the CPI. Owner sophistication is crucial to the process in order to draft a properly structured concession agreement where more than ever the devil is in the details. For example, prior tangles over a non-compete clause hampered both the Dulles Greenway and SR-91 Express Lane projects (Garvin, 2007).

Given its long-history and successful use elsewhere around the world, there is a compelling case to be made for the selective use of public-private partnerships in the delivery of infrastructure projects here in the United States. With numerous awards for engineering excellence and design innovation and an opening that was six months ahead of schedule, the Confederation Bridge offers a successful example of a public-private partnership used in the procurement of a major bridge structure. Research and prior experience suggest that large-scale bridge projects may be particularly amenable to the PPP approach. Consequently, the preceding discussion should among other things inform the debate on whether to utilize public-private partnerships in the replacement of some major bridge structures in New York. In an era of increasing demands on the public purse, a little private investment may go a long way.

REFERENCES

American Society of Civil Engineers. 2009. *Report Card for America's Infrastructure.* Reston, Virginia: Author.

Fisher, G. & Babbar, S. 1996. *Private Financing of Toll Roads*. Washington, D.C: The World Bank.

Frank, T. 2009. Toll roads are paved with bad intentions. *The Wall Street Journal, January 28, 2009.*

Garvin, M. 2007. *America's Infrastructure Strategy: Drawing on History to Guide the Future*. Washington, D.C: KPMG LLP and Stanford University.

Garvin, M. & Bosso, D. 2008. Assessing the effectiveness of infrastructure in public private partnership programs and projects. *Public Works Management & Polic*y 13(2):162–178.

Miller, J.B. 2000. Principles of Public and Private Infrastructure Delivery. Boston: Kluwer Academic.

Miller, J.B. 2002. Case Studies in Infrastructure Delivery. Boston: Kluwer Academic.

Malanga, S. 2007. How to keep our bridges safe. *The Wall Street Journal, August 4th, 2007.*

New York State Commission on State Asset Maximization. 2009. *Preliminary Report*.

Ortiz, I.N. & Buxbaum, J.N. 2008. Protecting the public interest in long-term concession agreements for transportation infrastructure. *Public Works Management & Policy* 13(2):126–137.

Page, S. et al. 2008. The risks and rewards of private equity in infrastructure. *Public Works Management & Policy* 13(2):100–113.

Phillips, T. 2009. NY., NJ. port authority weighs P3s for staten island-n.j. bridge projects. *The Bond buyer, February 4th, 2009.*

Rivera, R. 2009. Port authority may help financing of tower. *The New York Times, March 22nd , 2009.*

Samuel, P. 2005. *Should States Sell their Toll Roads? Policy Study 334*. Washington, D.C: Reason Foundation.

Tawiah, P. & Russell, A. 2008. Assessing infrastructure project potential as a function of procurement mode. *Journal of Management in Engineering* 24(3):173–186.

Chapter 27

Special topics studies for baseline structural modeling for condition assessment of in-service bridges

E.S. Bell
Department of Civil Engineering, University of New Hampshire, USA

J.D. Sipple
Department of Civil and Environmental Engineering, Tufts University, USA

ABSTRACT: As the civil engineering profession continues to grow with our ever aging infrastructure, structural health monitoring using nondestructive load testing techniques with comparison to an analytical structural model is rapidly becoming an economical method for decision-making related to asset management. A structural model, verified with collected field data, can provide an objective basis on the decisions to repair or replace bridges and the importance of each action to the safety of the driving public to determine the order in which bridge repairs need to be implemented. Several challenges will be presented to modelers including, but not limited to, not relying on design assumptions when creating finite element bridge models, finding the most efficient way to transmit truck load to the modeled bridge deck, and including specific elements present at the bridge in the monitoring based model. This paper discusses special topic studies relating to the issues above as successfully deployed and validated in a model updating exercise of the Rollins Road Bridge in Rollinsford, NH.

1 INTRODUCTION

1.1 *Social need*

Bridging the Gap, published by the American Association of State Highway and Transportation Officials (AASHTO) in July 2008, addressed the issues with our nation's aging infrastructure in response to the one year anniversary of the Interstate-35W Bridge collapse (Petroski, 2007). Five major problems of our nation's bridges are age and deterioration, congestion, soaring construction costs, maintaining bridge safety, and the need for new bridges. Five proposed solutions for our nation's bridges are investment, research and innovation, systematic maintenance, public awareness, and financial options (AASHTO, 2008). The collapse of the I-35W Bridge was a tragedy, however it did bring the safety of our aging infrastructure into the public eye. The *2006 Status of the Nation's Highways, Bridges, and Transit* report published by the U.S. Department of Transportation states that of the 594,101 bridges in the National Bridge Inventory, 13.1% are rated as structurally deficient and 13.6% are rated as functionally obsolete. The terms structurally deficient and functionally obsolete mean "deteriorated conditions of significant bridge elements and reduced load-carrying capacity" and "function of the geometrics of the bridge not meeting the current design standards" respectively (U.S. Department of Transportation, 2006).

With more than 3 trillion traveled bridge vehicle miles annually, 223 billion miles being truck traffic, traffic loading is one of the major factors in the deterioration of America's bridges. The construction boom of Interstate Highway System in the mid-1950s to mid-1970s resulted in an unprecedented period of infrastructure construction. These 590,000 bridges are essential for the transportation of the nation's commerce as well as carrying thousands of commuters to and from work every day (AASHTO, 2008). Bridges are essential for the economy of this country but are easily overlooked since they are traveled safely day in and day out.

The average bridge in the United States has an age of 43 years old and a design life of 50 years. Therefore, the need for another large infrastructure construction project to replace the aging infrastructure is imminent. Prioritization of red listed bridges will be required to achieve this daunting but necessary task in an efficient fashion (NHDOT, 2008). The decision to replace or repair, and how to repair each individual bridge structure, is a common and difficult management issue for bridge owners (Farhey, 2005). For structural evaluation of bridges, various types of sensor information, such as strain distribution, vibration and natural frequencies, and deflection measurements are used to generate data that is related to the health and load carrying capacity.

Data can be collected on all types of structures in different ways, but what makes the information beneficial for decision making is how it is used to obtain value added information. Several structural health monitoring (SHM) research projects have been performed using different SHM techniques. A popular method in SHM and damage detection is the use of vibration data and modal parameters (Brownjohn, Moyo, Omenzetter & Chakraborty, 2005). This is popular because it does not require measuring displacement, strain, and rotations, which are subject to load application and environmental effects. Such data is generally used with a finite element model of the structure to attempt to determine the bridge performance parameters by solving an inverse problem.

1.2 *Baseline modeling*

With the current advancements in bridge modeling programs, such as SAP2000® (Computer Structures, Inc., 2007) and GT Strudl® (GT Strudl, 2007), finite element modeling has often become part of the bridge design process. The SAP2000® Bridge Information Modeler can be used to compute influence lines and bridge response due to applied vehicle loads, dynamic loads, moving vehicle loads, self weight, and several other load applications including thermal loads (Computer Structures, Inc., 2007). Programs like SAP2000® and other structural analysis and design programs are used mainly as an aid in the design process in conjunction with local codes. The goal of a baseline structural model is to capture accurate structural behavior. When creating a model for structural health monitoring, it needs to be more detailed than models created for design purposes.

Bridges are typically designed according to design codes which have a goal to produce a safe bridge design in a practical time frame. SHM modeling involves selecting of appropriate software where characteristics can be easily added such as the modeling of elastomeric bearing pads, carbon fiber reinforced polymers, prestressing, and bridge girder geometry. The type of model used in a SHM program has different characteristics and areas of focus than a model used for design purposes. The SHM model must be accurate enough to capture the behavior of the bridge and be used in parameter estimation and model updating. Boundary conditions are an important and sensitive detail in modeling, such as those associated with accurately modeling elastomeric bearing pads. All loads applied to the bridge during a load test, whether they are vehicle, temperature, or wind must be included in the SHM model. All structural properties and components of the bridge during load testing such as elastomeric bearing pads, carbon fiber reinforcement polymers, the New England Bulb Tee girder, bridge rails, and temperature effects must also be included in the SHM model. This type of model is created for the Rollins Road Bridge (RRB) in Rollinsford, New Hampshire and the model was verified with field data collected during an April 2008 load test of the RRB.

2 ROLLINS ROAD BRIDGE, ROLLINSFORD, NEW HAMPSHIRE

2.1 *Background*

Rollins Road Bridge is located in Rollinsford, New Hampshire. Rollinsford is in southeastern New Hampshire about 12 miles from the Atlantic Ocean, see Figure 1. The bridge is not considered to be located in a coastal region, which would add considerations associated with being

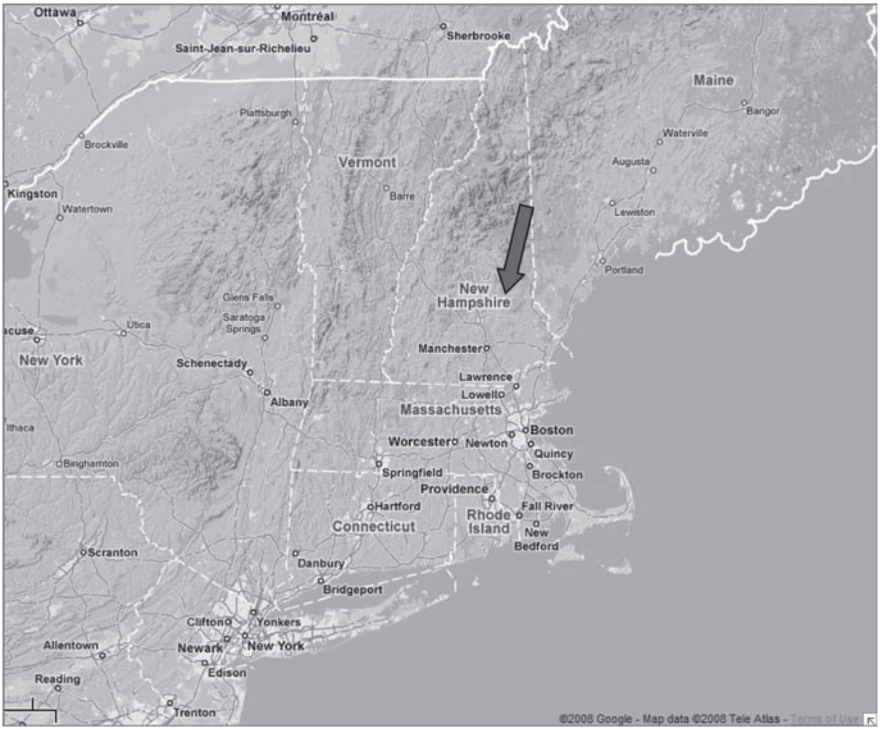


Figure 1. Location of Rollins Road Bridge (Image Courtesy of Google Maps©).

close to saltwater. The bridge serves as an overpass to carry Rollins Road over Main Street and an active B&M Railroad (NHDOT Bureau of Bridge Design, 1999). The weather in the area is typical of New England, with an annual snowfall of 60 inches, as recorded in Concord, NH about 35 miles west of the bridge (National Climatic Data Center). Such harsh winters mean a heavy use of deicing agents on the road surface throughout the winter months. The effects from the use of these harsh chemicals can be seen in the deck of the previous 70-year old RRB. The deck had to be replaced/repaired several times due to deterioration accelerated by use of deicing agents (Bailey & Murphy, 2008).

The original Rollins Road Bridge was a two lane bridge, steel stringer with concrete deck, four simple spans in series making a total length of 172-feet, and built in the 1930's, see Figure 2. The NHDOT decided that due to corrosion of both the steel reinforcement in the concrete deck and the steel stringers, the bridge needed immediate repair or replacement (Bowman, Yost, Steffen & Goodspeed, 2003). The last inspection report of the old RRB was done during the construction of the new bridge, shown in Table 1. The report notes that there were several problems with the bridge, including a rating of 3 for serious deck condition.

The NHDOT planned to remove the old Rollins Road Bridge and construct a new bridge in its place to open in the year 2000. The new Rollins Road Bridge was designed and constructed with funding from the Innovative Bridge Research and Construction (IBRC) program which is administered by the Federal Highway Administration (FHWA). The new Rollins Road Bridge, referred to from this point forward as Rollins Road Bridge, is the focus of this research project on SHM for the NHDOT. The purpose of the IBRC program is "to reduce congestion associated with bridge construction and maintenance projects, to increase productivity by lowering the life-cycle costs of bridges, to keep Americans and America's commerce moving, and to enhance safety" (Office of Bridge Technology, 2008).

Figure 2. Rollins Road Bridge prior to new bridge construction (Bowman M.M., 2002).

Table 1. Excerpt of the 2000 Rollins Road Bridge Inspection Report (NHDOT Bureau of Bridge Design, 2007).

26 October 2000 Bridge Inspection Report	
Deck	3 Serious
Superstructure	4 Poor
Substructure	6 Satisfactory

Two requirements of the IBRC program are the bridge is to be constructed with high performance and innovative materials and be instrumented. The focus of the IBRC program is using technology in the bridge to require less maintenance while keeping ease of construction a high priority in the design of the structure. The goal of the instrumentation in RRB is to follow the progress of the new materials used in the bridge, again not for SHM. However, even though the instrumentation plan was not specifically designed for SHM, this research project was able to successfully utilize some of the sensors, including strain and temperature, in the bridge to capture the behavior of the bridge during NDT load tests.

Rollins Road Bridge, opened in December 2000 and seen in Figure 3, is a simply supported single span of 110-feet with a concrete beam and concrete deck superstructure. The center pier was also not included in the new bridge design for safety purposes. The bridge has a rating of 99-tons (Fu, Feng & Dekelbab, 2003) and is in very good condition, as seen in the most recent inspection report shown in Table 2.

2.2 *Instrumentation plan*

As part of the IBRC, RRB was instrumented in order to capture the behavior of the CFRP and the bridge deck which contained an innovative material. All of the sensors in the deck are oriented in the lateral direction, perpendicular to the flow of traffic. This was done in order to understand the

Figure 3. New Rollins Road Bridge, opened in 2000.

Table 2. Excerpt from the 2007 Rollins Road Bridge Inspection Report (NHDOT Bureau of Bridge Design, 2007).

09 July 2007 Bridge Inspection Report		
Deck	9	Excellent
Superstructure	9	Excellent
Substructure	9	Excellent

behavior of the deck as it bends over the girder when a load is applied. The only gauges oriented in the longitudinal direction, with the flow of traffic, were gauges in the precast, prestressed, high performance concrete NEBT girders. The purpose of these gauges was for researchers from the University of Nebraska at Lincoln to quantify the loss of prestress in the high performance concrete girders. These longitudinally oriented gauges proved to be most beneficial for the SHM program since they capture the global bending behavior of the bridge. The instrumentation plan was not designed for SHM, however full advantage was taken of the gauges for research in SHM.

The fiber optic concrete strain sensors used in this project are Fabry-Perot strain gauges for embedment in concrete (EFO). The actual Fabry-Perot strain sensor is mounted inside a stainless steel envelope with two end flanges to ensure durability and protection of the sensor for long term monitoring projects, such as RRB. The fiber optic sensors are also small in size, lightweight, non-conductive, resistant to corrosion, and immune to electromagnetic noise and radio frequencies eliminating need for shielding and lightening protection (Choquet, Juneau & Bessette, 2000). The gauges were connected to fiberglass studs, arranged in a planned depth throughout the deck. In RRB, all of these strain gauges are concentrated between girders 3 and 4, near the longitudinal mid-span. Temperature sensors were also installed in the deck to obtain internal concrete temperatures.

The purpose of the girder sensors was originally to instrument and observe the prestress loss in the high performance concrete girders. The results from this research, which also examined several other bridges, was used in creating the model and is located in *NCHRP Report 496* (Tadros & Al-Omaishi, 2003). Girders 3, 4, and 5 have strain sensors installed at the longitudinal midspan of the bridge and at three different depths throughout the girder. These sensors were placed after tendon prestressing but before concrete placement, as seen in Figure 4. Figure 5

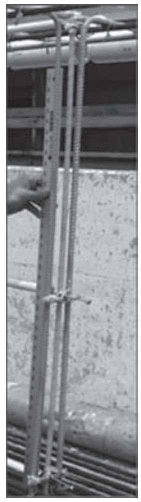
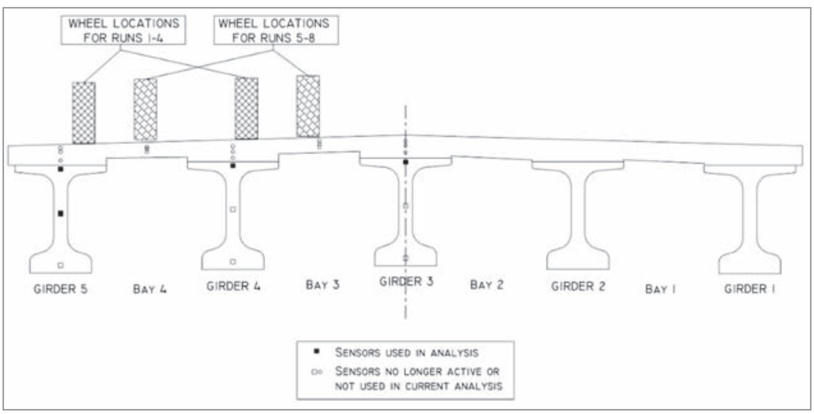
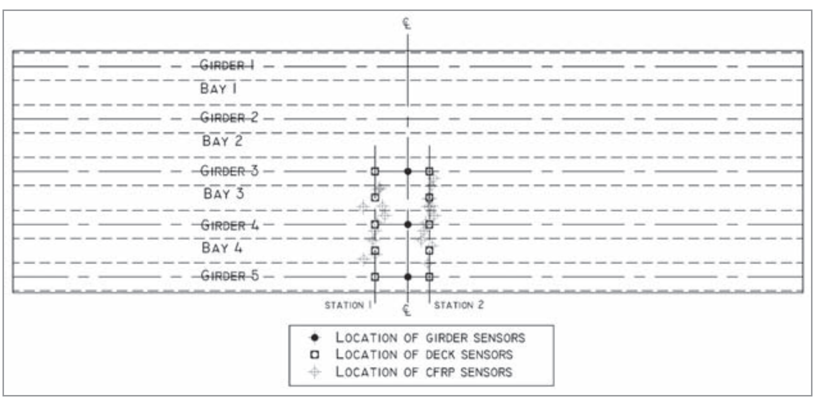


Figure 4. Strain gauges in NEBT girder before prestressing (Adapted from (Bowman M.M., 2002)).



(a)



(b)

Figure 5. Graphic of sensors used in Rollins Road Bridge analysis, (a) shows the sensors in section view and (b) shows the sensors in plan view.

shows which gauges were used for analysis in this project. Only four out of the nine girder gauges were used because they were the four that had readings from all load tests and were still working in 2008.

The data management instrument (DMI) is located on-site and is in good working condition. The DMI is a 32-channel fiber optic data acquisition system provided by FISO Technologies, Inc. This particular DMI model has the ability to record continuous data or be calibrated for a controlled static load test. Since the start of the research project, continuous temperature and strain data has been downloaded from the bridge for use by future researchers to investigate the long term thermal performance of the CFRP and concrete deck through trends and examining material properties. For the continuous, long term temperature and strain data the DMI is configured to take 60 readings over the period of an hour and average those values to produce one data point for that hour. The DMI is also attached to a modem, allowing researchers to remotely call the bridge to download data or see current conditions.

2.3 *Field testing*

The load test for the RRB was conducted on 18 April 2008. The purpose of this load test was to collect data in a similar fashion to the previous load tests, while also collected data to be used for SHM. Rollinsford Police Department was used for traffic control on the bridge during the load test. No traffic was allowed to pass while strain readings were being taken, and traffic was allowed to pass when the load truck was being moved. Three zero-load readings were also taken during the duration of the load test, which proved to be crucial in relating measured response to the monitoring model. The NHDOT Survey Crew used differential leveling to obtain displacement readings during the load test.

2.3.1 *Truck specifications*

This load test, like the previous two load tests was done in conjunction with the NHDOT. A two axle NHDOT Sand Truck, as seen in Figure 6 and Figure 7, was used for load application to the bridge. The wheel weights of the truck were taken in similar fashion to the previous load tests by the New Hampshire State Police Mobile Weigh Station, a Haenni Scales, model #WL 101 (Haenni, 2008), have a variance of less than 1% and are tested and certified by the NH State Police., as seen in Figure 7. The gross weight of the truck was 37.4-kips (18.69-tons).

The dimensions for the truck were 14-feet 9-inches between the center of the front and rear wheel. The rear dual had a thickness of 1-foot 8-and-½-inches. The rear axle, center of dual to

Figure 6. NHDOT sand truck as load application during April 2008 load test.

Figure 7. Trooper Huddleston (NH State Police) taking NHDOT wheel load measurements.

center of dual, length was 6-feet 2-inches. The front wheel had a thickness of 8-inches and the length of the front axle from center of wheel to center of wheel was 7-feet.

2.3.2 *Testing plan*

The truck ran in the north-west direction and south-east direction a total of eight times, four in each direction. Two separate marking groups were laid out on the bridge. One group had a wheel directly on the girder and the other had the wheels straddling over a girder. Each group of markings was traveled two times per direction, two directions, equaling four times per marking group, two marking groups, a grand total of eight passes. Initial measurements for the markings were done using an estimated truck size, and the actual truck that was used for the load test was similar to those estimations. In runs one through four, the trucks wheels were on girders five and four. For runs five through eight, the trucks wheels straddled girder 4.

2.4 *Structural modeling*

The initial modeling protocol focused on including specific elements and environmental impacts. The goal was to create a usable model for the NHDOT SHM program. The structural model was created using the BrIM™ (Bridge Modeler) in SAP2000®.

Once the design based model was created using the BrIM™ to a degree of satisfaction, the bridge modeler was turned off, allowing researchers take full control of element properties included in the model. The use of the BrIM™ takes full advantage of all the research done by Computer & Structures, Inc. (CSI) for the creation of the base bridge model and then allows researchers to build upon that model to reach the final goal. Structural components included in this baseline model were prestressing tendons in the girder, CFRP reinforcement in the deck, the bridge rail, and boundary conditions modeled as springs with prescribed stiffness. This model was then verified using the collected field data, which meant that the model needed to be coordinated with the truck loading locations and environmental impacts as well. Figure 8 shows the final model created for the Rollins Road Bridge, which includes the parapet link to the bridge deck and spring supports for boundary conditions.

2.5 *Prestressing tendons*

The SAP2000® BrIM™ contains preloaded concrete girder sections. Those sections can be used or modified depending on the properties of the girder located at the bridge. This was one big benefit

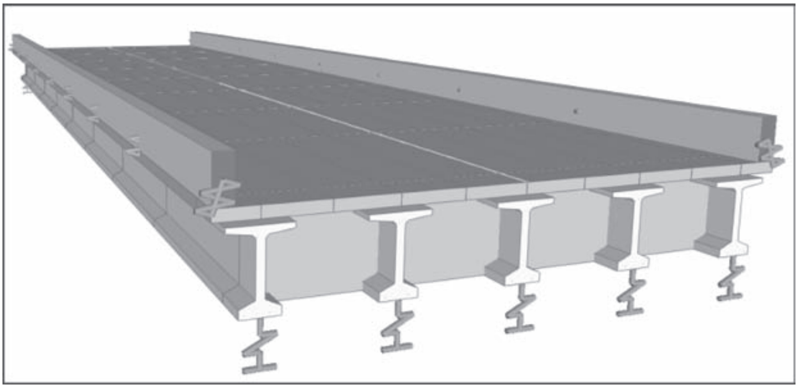


Figure 8. Final model of Rollins Road Bridge in SAP2000®.

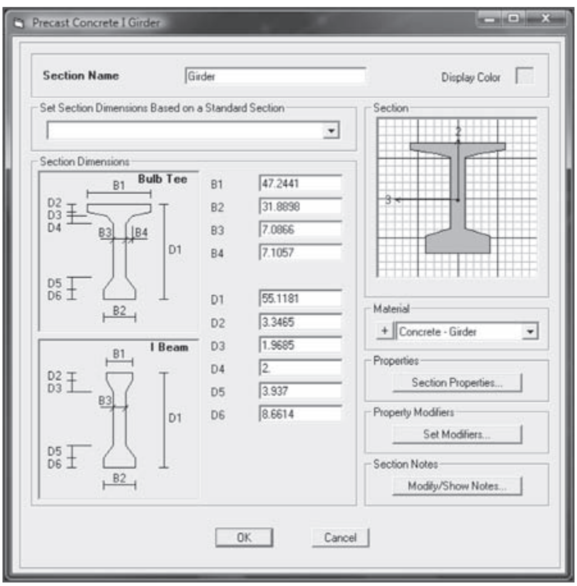


Figure 9. Preloaded NEBT section in SAP2000® (SAP2000, 2007).

to using SAP2000®, and it contains all of these different options which makes model easy for all bridges, not only RRB. Figure 9 shows the preloaded AASHTO PCI bulb tee included in the BrIM™ with the modified dimensions to match that of the NEBT.

Prestressing tendons were included in the RRB model to accurately capture the bending behavior of the girders. SAP2000® has the ability to add strand patterns, see Figure 10. The two deflection point pattern used at RRB was one of the many options in the BrIM™. The design plans were used for all of the stressing, arrangement, and steel specification information. Losses were calculated using the AASHTO Bridge Code (AASHTO, 2004). The use of these values was validated through *NCHRP Report 496* which looked at the actual losses at RRB and compared them with losses calculations using AASHTO (Tadros & Al-Omaishi, 2003). During fabrication, special care was taken to ensure that the strand pattern was accurately laid out, as prescribed in the plans, and researchers were present at time of prestressing and pouring of the precast girders to

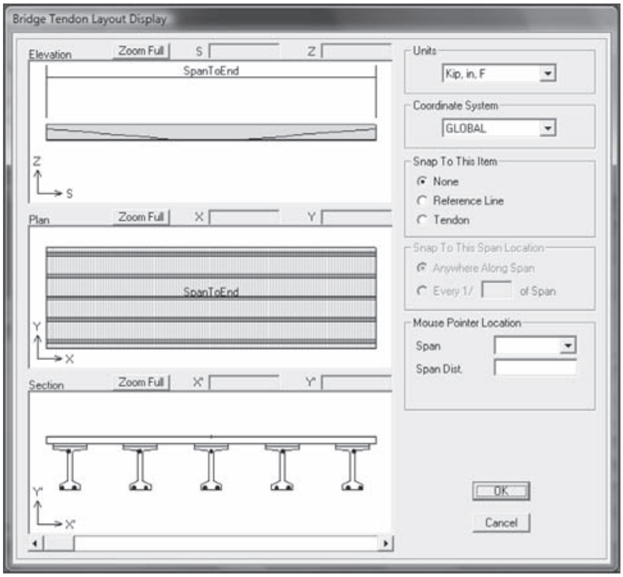


Figure 10. SAP2000® bridge tendon layout (SAP2000, 2007).

ensure compliance. Due to the research driven nature of this project, there was extra control in all aspects of construction, which allows researchers a high level of confidence that the bridge was constructed as designed and specified.

2.6 *Carbon reinforced fiber polymer in the deck*

The deck was modeled using design plans for RRB and measured distances (Bowman M.M., 2002). No as-built drawings were available for this bridge deck, so between Bowman's (2002) data and the design plans, researchers felt fairly confident in the dimensioning for the deck. CFRP reinforcement in the deck was included once the bridge modeler was turned off and the type of finite elements used for the bridge deck was changed from shell elements to layered shell elements. The deck of RRB contains two layers of CFRP reinforcement, one above and one below the centroid of the deck section.

In order to correctly model the CFRP material, the material specifications, modulus of elasticity, and density were obtained from previous work (Bowman M.M., 2002) (Trunfio, 2001). The thickness of the CFRP throughout the entire width of the deck was maintained to keep the correct moment of inertia in the transformed section and having the ability to model it in SAP2000®. Since the layered shell material was throughout the entire thickness, not just present every 6-inches, the modulus of elasticity was transformed to capture the same behavior as it is placed in the bridge. The modification was achieved by taking a ratio between the actual area of CFRP in the cross section and the modeled area and then reducing the modulus of elasticity for the layer. A graphical representation of the steps list above can be seen in Figure 11.

2.7 *Bridge rail*

The bridge rail at Rollins Road Bridge is a cast-in-place concrete rail. The use of concrete bridge rails is replacing the conventional aluminum/steel guardrail for NHDOT bridges. The rail will be modeled as a frame element and connected to the bridge deck through links since, as seen in Figure 12, it is connected to the bridge deck using stainless steel reinforcement.

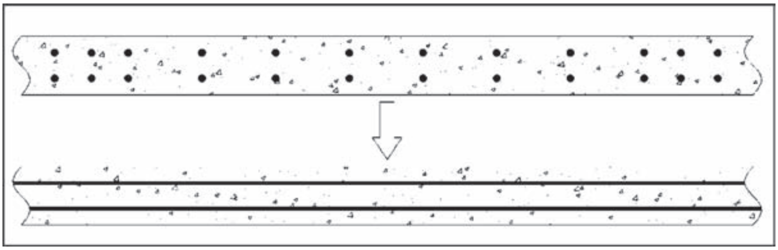


Figure 11. Graphical representation of how CFRP is modeled as layered shell element.

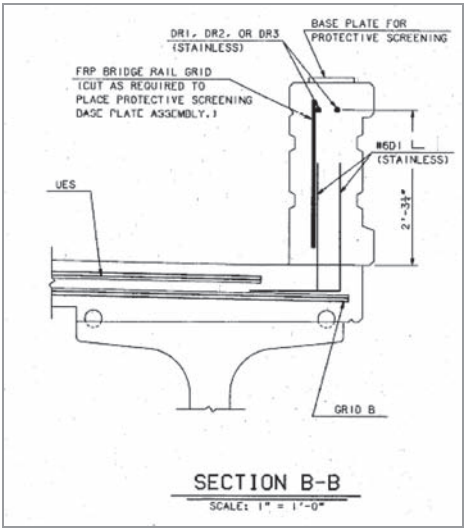


Figure 12. Section view of bridge rail connection to bridge deck (NHDOT Bureau of Bridge Design, 1999).

2.8 *Elastomeric bearing pads*

Steel reinforced elastomeric bearing pads support the Rollins Road Bridge on the abutments which transfer all loads into the ground. The bearing pads have three different possible directions of motion, as seen in Figure 13, caused by axial load, shear forces, and rotation.

Visual inspection showed no cracking or deterioration in the deck or girders. Research has been conducted beyond the initial research performed by AASHTO on both the axial and rotational stiffness of steel reinforced elastomeric bearing pads in order to develop bearing pad stiffness (Stanton, Roeder, Mackenzie-Helnwein, White, Kuester & Craig, 2008). This research and physical testing, have resulted in two formulas that can be used to calculate axial and rotational stiffness, as shown respectively in Equations (1-a) and (1-b), for one layer of the elastomer. Combining the stiffness value of each layer of elastomer and steel together produces an overall stiffness for the bearing pad (Stanton, Roeder & Mackenzie-Helnwein, 2004).

$$K_a = \frac{P}{\Delta_a} = \frac{EA(A_a + B_a S^2)}{t} \tag{1-a}$$

$$K_r = \frac{M}{\theta_r} = \frac{EI}{t}(A_r + B_r S^2) \tag{1-b}$$

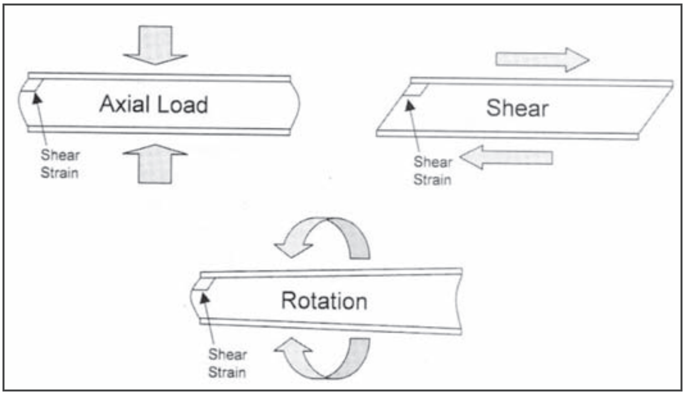


Figure 13. Deformations of a laminated elastomeric bearing pad (Stanton, Roeder, Mackenzie-Helnwein, White, Kuester & Craig, 2008).

A total of ten, 16-inch diameter, steel reinforced elastomeric bearing pads are installed at RRB, one at each end of each girder. The bearing pads allow slight vertical compression while allowing the beam to rotate. Modeling spring boundary conditions, via links, in SAP2000® is also fairly simple. The BrIM™ allows for several different types of boundary conditions to be used, from traditional fixed or pinned connections, to user defined links. When links are used, the user is allowed to specify stiffness in all directions. Links are used because they can be updated in the model updating process and more accurately capture the behavior of the actual bearing as opposed to a pinned or fixed condition. In the U2 directions (translation parallel to the abutment) a stiffness of 1.000E+09 is used to show fixity in those directions and in the R1 and R3 directions (rotation about a line normal to the abutment and about a vertical line) a stiffness of 1.000E-09 is used when rotational stiffness is not included. These values are specified instead of using the option to be fixed or free in the SAP2000® program window because using those options caused numerical instability in the analysis. Using values that accurately represent fixed and free did not cause the numerical instability but essentially gave the same response.

Stanton et al. (2008) has equations to calculate axial and rotational stiffness of the elastomeric bearing pads, however does not provide equations for the calculation of horizontal stiffness caused by shear effects. That value estimated using the collected field data from the April 2008 Load Test.

2.9 *Load application*

Typical load application is achieved by applying a load to a node in the model. The BrIM™ has a predetermined pattern for creating joint locations in the bridge model, not necessarily where the truck will be. A solution was to place nodes where there was a point of load application. That led to confusing creation of shell elements to get a solid deck. There could be an infinite number of different locations for load application during a load test that may not necessarily already be a point. Typical load application is done by a truck, which in realty are applying the wheel loads over an area.

If a finite element mesh was created and the area loads were applied to this separate mesh, resultant forces could be calculated at points of actual node locations on the bridge. A fine mesh, using 3-inch spacing, was created to obtain the force resultants. Once this mesh was created, it could be moved to any place on the bridge to find resultant forces. This universal method proved to be useful during the analysis portion of this research project, allowing loads to be applied in different locations on the bridge depending on the specific load case. Once the mesh was moved

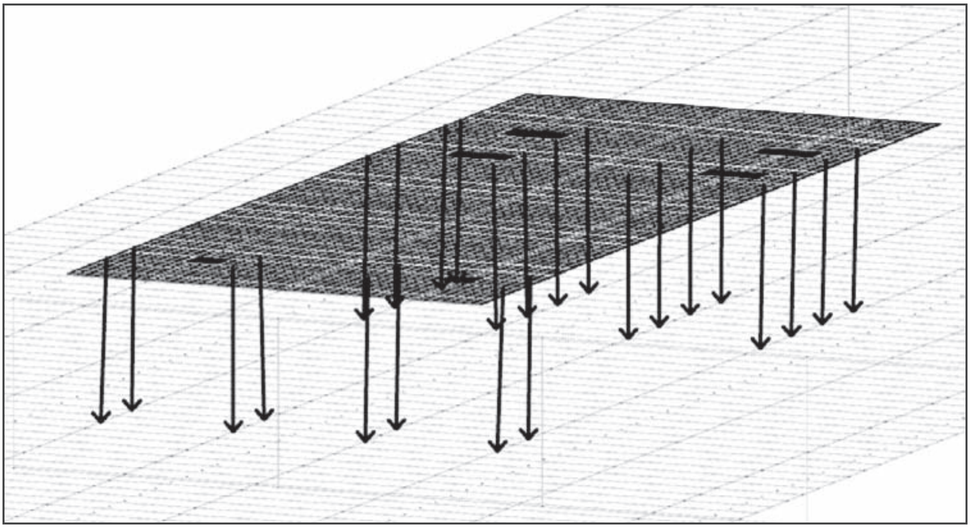


Figure 14. Truck load mesh to bridge deck graphic.

to the area of load application, the equivalent area loads were applied to the mesh model, and the two existing nodes on the deck were selected as boundary conditions in the mesh model. This was done for all areas of load application and the mesh model was run. The resulting reaction forces from the mesh model where then applied to the deck nodes, as seen in Figure 14.

The use of force resultants can be done because the focus of the load tests was to look at the overall effect on the bridge. The sensors used in the analysis were in the girders, so local effects from the truck wheels were not of concern. It also takes full advantage of using the BrIM™, while still being universal enough to apply loads to existing nodes at any location on the bridge.

3 VERIFICATION RESULTS

Table 3 shows the five different support conditions (SC) used in second manual model updating analysis. The vertical stiffness values and horizontal are modified in the first four cases, and the fifth case shows that modification of the horizontal stiffness value must be done in order to get the change in model strain to match the measured change in strain. The error of ±0.40-microstrain shown in the error bars for the measured strain corresponds to the accuracy of the gauges as set when installed (Figure 15).

Table 3. Manual model updating cases and corresponding bearing pad stiffness values for second analysis.

	Vertical Stiffness *(kips/in)*	**Rotational Stiffness** *(kips/rad)*	**Horizontal Stiffness** *(kips/in)*
Support Condition 1	46833	224651.5	fixed
Support Condition 2	46833	free	fixed
Support Condition 3	fixed	free	fixed
Support Condition 4	46833	fixed	fixed
Support Condition 5	46833	224651.5	10000

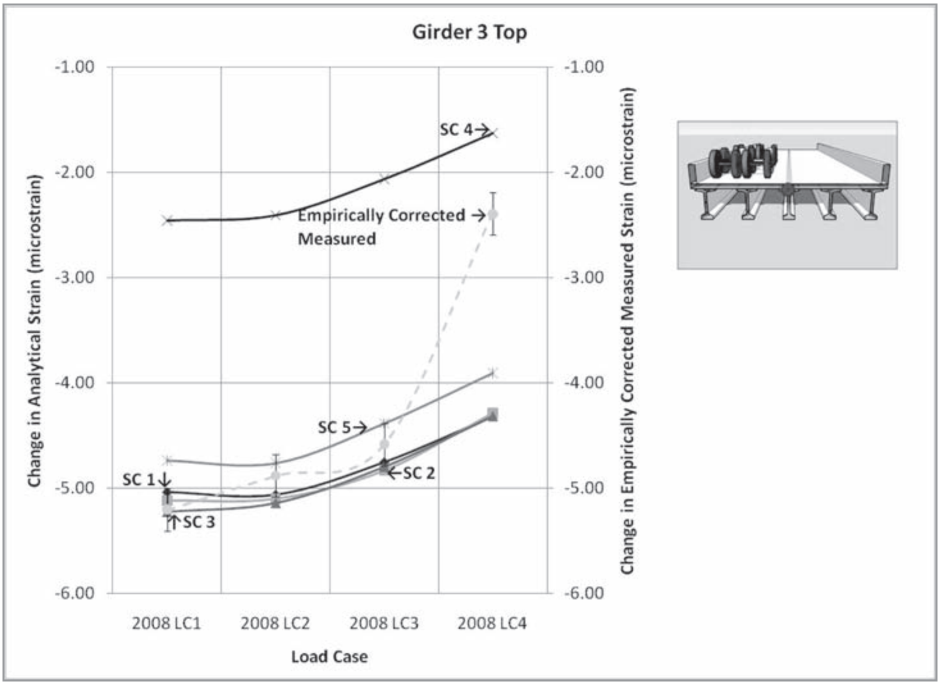


Figure 15. Manual model updating using girder 3 top strain sensors.

3.1 *Conclusions on manual parameter estimating results*

The results from the manual parameter estimation show that the change in measured structural response could match the change in modeled response by modifying the horizontal stiffness of the elastomeric bearing pad. The final bearing pad stiffness ended up being 46,833-kip/in in the axial direction (k_a), 10,000-kip/in in the horizontal direction (k_h), and 224,651-kips/rad for rotation (k_r). Figure 16 and Figure 17 show a quantification of the bearing pad stiffness values used as compared to a roller, pinned, and fixed connection. This is only to show the effects of the spring on an example 40-foot beam with a 10-kip point load, not the actual bridge configuration. The axial and horizontal stiffness remained as calculated since there was nothing to suggest otherwise, and the horizontal direction was modified to get the structural response to match. According to Stanton et al. (2008), there are no standard calculations for the horizontal stiffness value.

3.2 *Analysis of removing specific structural elements*

The bearing pad stiffness obtained from the above analysis, support configuration 5 now benchmark, will be kept constant in the next analysis of modeled response.

Table 4 shows the four cases that will be used to show the effect of specific parameters in the model. Structural parameters such as CFRP, prestressing, and bridge rail will be removed from the SAP2000® model, and the response will be seen in Figure 18.

These results show that not including the bridge rail in the model had significant effects on the change in measured response of the bridge model. Removing prestress and/or CFRP had a smaller effect in change of strain but it must also be remembered that this is a change in strain, so the benchmark model for the base also has no CFRP or prestress which explains why the values appear to be similar.

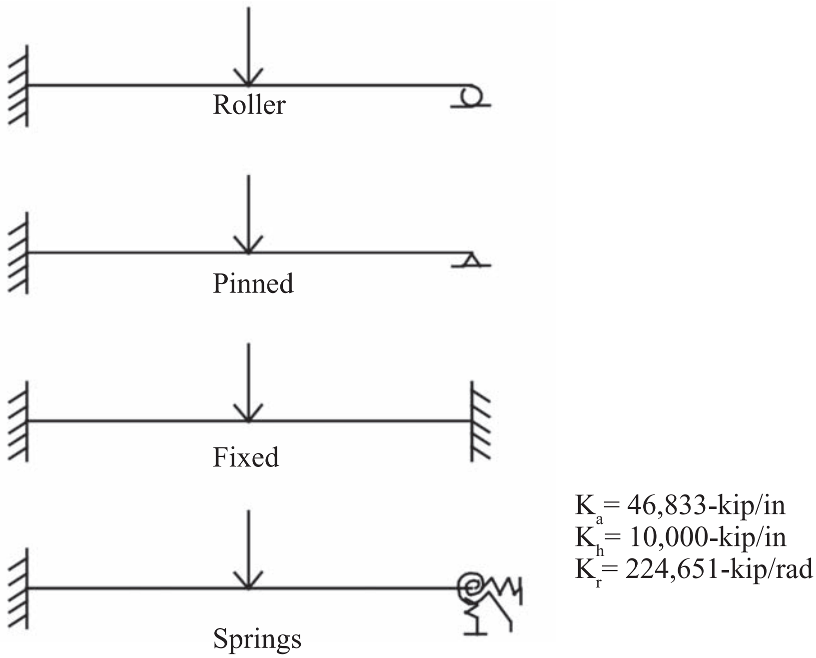


Figure 16. Quantification of bearing pad stiffness examples.

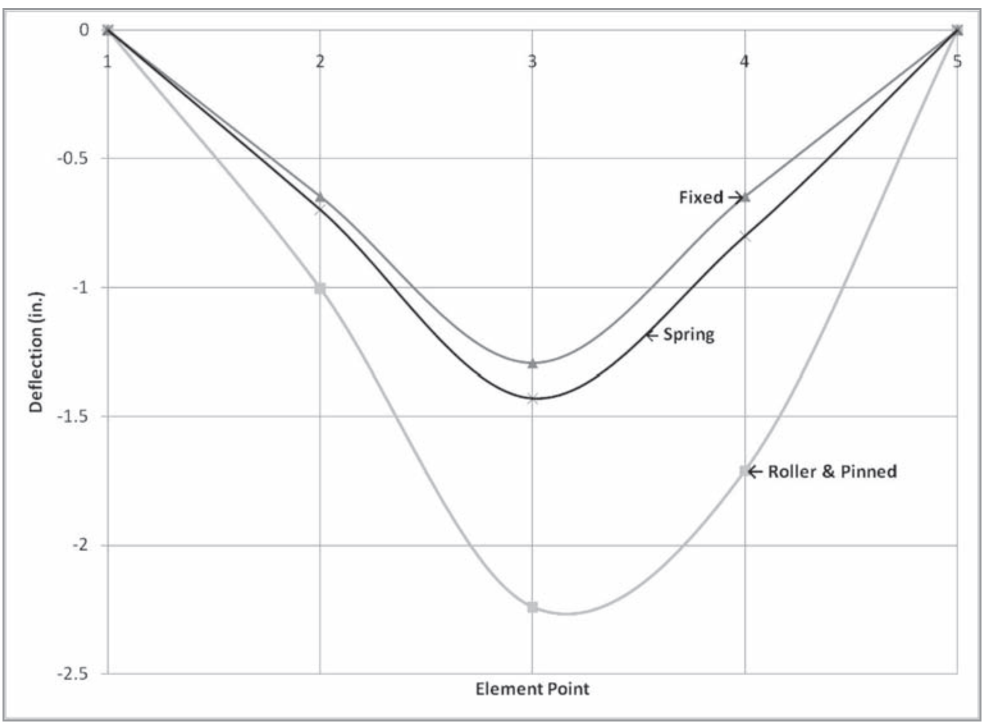


Figure 17. Quantification of bearing pad stiffness results.

Table 4. Manual model updating cases and corresponding bearing pad stiffness values for third analysis.

	Vertical Stiffness *(kips/in)*	**Rotational Stiffness** *(kips/rad)*	**Horizontal Stiffness** *(kips/in)*
Benchmark	46833	224651.5	10000
No CFRP	46833	224651.5	10000
No Prestress	46833	224651.5	10000
No Bridge Rail	46833	224651.5	10000

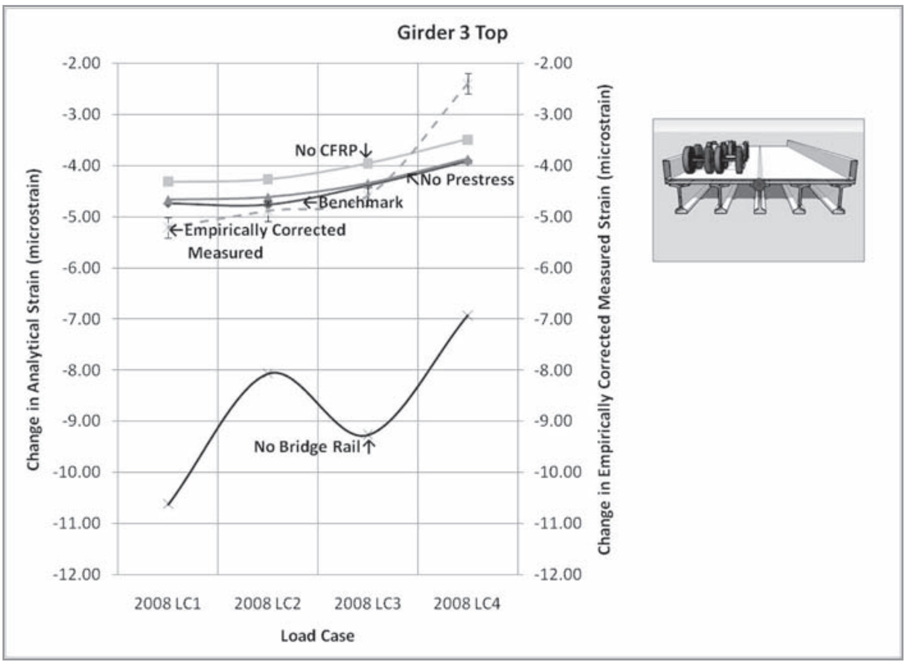


Figure 18. Manual model updating using girder 3 top strain sensor.

4 CONCLUSIONS

The stiffness of the bearing pads was updated solely for the reason of experimentally determining the horizontal stiffness of the elastomeric reinforced bearing pad, the one stiffness value not given through experimentally verified, industry-accepted equations. The effects of including the bridge rail can be seen when that element in removed. Another option to deal with the bridge rail would be to break up the element that models the bridge so it is not modeled as a continuous bridge rail, which would more accurately reflect how it is cast on the bridge.

4.1 *Special studies*

A baseline model, with added specific structural components, was created to capture the behavior of the bridge. The effects of removing those components can be seen the analysis with results seen in Figure 18. This model and the data from the load test is currently in a phase where it can be easily be continuously updated to reflect the state of the RRB.

As noted in the current bridge inspection report, there are no visible signs of deterioration or cracking, which caused the main focus of the parameter estimation to be the horizontal stiffness of the elastomeric bearing pads. Visual inspections will continue to be performed at RRB, and once there is noted deterioration, the model will be easily updated to model that change in behavior. The modeling of structural deterioration will also allow that deterioration to be quantified as a reduction in area, moment of inertia, or modulus of elasticity instead of a note on an inspection report.

The truck load mesh model used to apply the truck load provides a universal approach to truck load application to the bridge model. Using this technique allows a truck load at any location, with any load configuration to be applied to a monitoring based bridge model. This will facilitate this model use for future load test and incorporation with future weigh-in-motion station for traffic excitation strain readings. Including specific structural components into the monitoring based model that are not typically included in a design based model, allow for more accurate behavior to be captured in the model. Capturing the behavior more accurately allows for a much better comparison with measured data from a static load test. Using the load mesh and including specific structural components creates a monitoring based model that, as seen in the verification, can accurately capture bridge behavior while still maintaining usability in the model. This model will be handed over to the NHDOT of use by their bridge engineers in their everyday practices. The protocol used to create this model will considered for incorporation into the bridge design and management program at the New Hampshire Department of Transportation and other bridge owners and managers.

REFERENCES

AASHTO (2008). Bridging the Gap—Restoring and Rebuilding the Nation's Bridges.

AASHTO (2004). LRFD Bridge Design Specifications Third Edition. *14.5 Bridge Joints*, pp. 14–8.

Bailey, J. & Murphy, K. (2008). The Effectiveness of Fiber Reinforced Polymers in Modern Day Bridge Building. *Eigth Annual Freshman Conference* (p. 5). Pittsburgh, PA: University of Pittsburgh.

Bowman, M.M. (2002). *Load Testing of the Carbon FRP Grid Reinforced Concrete Bridge Deck on the Rollins Road Bridge, Rollinsford, New Hampshire*. Durham, NH: University of New Hampshire.

Bowman, M.M., Yost, J.R., Steffen, R.E. & Goodspeed, C.H. (2003). Diagnostic Testing and In-Service Performance Monitoring of a CFRP Reinforced HPC Bridge Deck. *Second New York City Bridge Conference—Recent Developments in Bridge Engineering* (pp. 361–371). New York, NY: Bridge Engineering Association.

Brownjohn, J.M., Moyo, P., Omenzetter, P. & Chakraborty, S. (2005). Lessons from monitoring the performance of highway bridges. *Structural Control and Health Monitoring*, 12:227–244.

Choquet, P., Juneau, F. & Bessette, J. (2000). New generation of Fabry-Perot fiber optic sensors for monitoring of structures. *SPIE's 7th Annual International Symposium on Smart Structures and Materials.* Newport Beach, CA.

Computer Structures, Inc. (2007, October). CSI Analysis Reference Manual for SAP2000®, ETABS®, and SAFE™. Berkeley, California, USA.

Farhey, D.N. (2005). Bridge Instrumentation and Monitoring for Structural Diagnostics. *Structural Health Monitoring*, 301–318.

Fu, G., Feng, J. & Dekelbab, W. (2003). *NCHRP Report 495—Effect of Truck Weight on Bridge Network Costs.* Washington, D.C.: Transportation Research Board.

GT Strudl. (2007). *Version 29*. Georgia Tech—CASE Center.

Haenni. (2008). Retrieved November 2008, from HAENNI Wheel Load Scales, a division of Baumer Borudon Haenni AG: http://www.haenni-scales.com/e/

National Climatic Data Center. (n.d.). *Snowfall—Average Total in Inches*. Retrieved August 18, 2008, from http://lwf.ncdc.noaa.gov/oa/climate/online/ccd/snowfall.html

NHDOT Bureau of Bridge Design. (2007). *Bridge Inspection Report—Rollinsford 091/085*. NHDOT.

NHDOT Bureau of Bridge Design. (1999, July). Rollins Road Bridge Over B&M Railroad and Main Street Plans.

NHDOT (2008, August 5–6). *New Hampshire bridges and related maintenance issues.* Northeast Bridge Preservation Partnership Meeting, Worcester, Massachusetts.

Office of Bridge Technology. (2008, August 18). *IBRC—Bridge*. Retrieved from FHWA: http://www.fhwa.dot.gov/bridge/ibrc/

Petroski, H. (2007, August 4). Learning from bridge failure. *Los Angeles Times*.

SAP2000 (2007). Verision Advanced 11.0.7. Berkely, California, USA: Computer and Structures, Inc.

Stanton, J.F., Roeder, C.W. & Mackenzie-Helnwein, P. (2004). *Rotational Limits for Elastomeric Bearings Appendix F.* Washington: Transportation Research Board.

Stanton, J.F., Roeder, C.W., Mackenzie-Helnwein, P., White, C., Kuester, C. & Craig, B. (2008). *NCHRP Report 596—Rotational Limits for Elastomeric Bearings.* Washington, D.C.: Transportation Research Board.

Tadros, M.K. & Al-Omaishi, N. (2003). *NCHRP Report 496—Prestress Loss in Pretensioned High-Strength Concrete Bridge Girders*. Washington, D.C.: TRB.

Trunfio, J.P. (2001). *Experimental Testing for the use of Carbon FRP Grids in the Rollins Rd. Bridge, Rollinsford*, NH. Durham, NH: University of New Hampshire.

U.S. Department of Transportation. (2006). 2006 Status of the Nation's Highways, Bridges, and Transit—Conditions and Performance.

Chapter 28

Merging and moving forward with New Jersey Turnpike Authority's Bridge inspection program

J.H. Laird
New Jersey Turnpike Authority, Woodbridge, New Jersey, USA

J.A. Paul
LS Engineering Associates Corporation, Montville, New Jersey, USA

ABSTRACT: New Jersey Turnpike Authority's bridge inspection program is evolving to encompass its recently combined inventory of 1,000 bridges on the New Jersey Turnpike and Garden State Parkway, in a challenging and high-usage environment. The article will highlight recent modifications and innovations, and the implementation of a ne w computerized bridge inspection and management database that is capable of handling the diverse structure populations on both roadways.

1 INTRODUCTION

New Jersey Turnpike Authority owns and operates the New Jersey Turnpike and the Garden State Parkway, and is dedicated to the safe and efficient movement of people, goods and information on one of the nation's largest and busiest toll-road systems. NJTA is responsible for the inspection and structural integrity of 1,000 routine and complex bridge structures, as well as sign support structures, culverts, antenna towers and high mast light poles.

In the past, these two major roadways operated as independent entities by the New Jersey Turnpike Authority (NJTA) and the New Jersey Highway Authority (NJHA), which managed distinct bridge inspection programs for their respective inventories. NJTA's bridge inspection program has been designed to meet the needs of a variety of critical stakeholders, including the Authority's Engineering and Maintenance Departments for planning maintenance repairs and capital program improvements, and New Jersey Department of Transportation (NJDOT) and the Federal Highway Administration (FHWA) for compliance with National Bridge Inspection Standards [NJDOT 2003] [FHWA 2005].

Numerous consultants are engaged by NJTA to perform bridge inspections each year. Organizing and maintaining all of this information into paper and computer formats for the various stakeholders is a difficult challenge. NJTA recently embarked on a program to address these challenges and standardize the inspection report format for both roadways. This is being accomplished by implementing a new integrated field and office based computer system that is capable of handling the wide variety of structure types and user needs, which will provide a greatly enhanced inspection and management process. The system will also be capable of handling upcoming changes to the Authority's inventory, resulting from major widening programs which are underway on the New Jersey Turnpike and Garden State Parkway [NJTA 2008].

2 NEW JERSEY TURNPIKE AND GARDEN STATE PARKWAY—PAST AND PRESENT

The New Jersey Turnpike was opened to traffic in 1951 by NJTA, and today covers 148 miles. It is one of the most densely traveled roads in the nation, with a 2007 total traffic volume of

250,000,000 vehicles. The Turnpike (Figure 1) serves as a primary link for both commercial and non-commercial traffic through the Northeast. The first toll section of the Garden State Parkway (Figure 2) was opened to traffic in 1954 by NJHA, and today covers 173 miles from the New York Line at Montvale to Cape May. The 2007 total traffic volume on the Parkway was 450,000,000 vehicles.

The combined bridge inventory consists of 1,000 bridges which comprise a total deck area of over 20 million square feet. Following the completed construction of the Turnpike Mainline, NJTA began to perform condition and maintenance inspections of the facilities with the assistance of the Authority's General Consulting Engineer, HNTB Corporation. The results were documented in letter form with highlights of deficiencies which were few and minor in nature. Starting in 1961, with the facility about 10 years old and growing evidence of more pronounced and frequent deficiencies caused by wear and tear, formal bridge inspection reports were prepared in a simplified checklist format for the structures grouped in each maintenance district. As no federal requirements or standards were in force at this time, the inspections were undertaken by NJTA strictly on its own initiative for safety assurance and scheduling of bridge maintenance activities.

Today, NJTA's bridges are not only inspected on a biennial basis to comply with National Bridge Inspection Standards (NBIS), but they are also inspected at a cursory level every year to address emergent structural deficiencies in a timely and cost-effective manner. The inspection report deliverables consist of NBIS reports of the routine bridges grouped by district/area, individual detailed reports of the major bridges, and specialized reports covering fracture critical members (FCMs) [FHWA 1986b] and other unique elements [ACI 2008] [AASHTO 2003] [FHWA 1986a].

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.

Figure 2. The Garden State Parkway varies from a 4-lane land service highway at the southern terminus, to multiple lane roadways further north.

3 CONSOLIDATION OF TURNPIKE AND PARKWAY BRIDGE PROGRAMS

In July 2003, the two toll road authorities were consolidated under NJTA, in an initiative to improve conditions for New Jersey's motoring public. Since the consolidation, NJTA has been maintaining a multitude of databases for the Turnpike and Parkway structure inventories. The Turnpike bridge inspection database started twenty years ago as a paper-based compilation of NJTA's traditional individual checklist forms, then evolved about ten years ago into a Microsoft Access database, and is now being transformed again into an integrated inspection and database system designed by InspectTech of Pittsburgh, PA.

4 STANDARDIZING THE INSPECTION REPORT FORMAT

NJTA's post-consolidation effort has also involved the task of merging the distinctive inspection report formats for both roadways. In 2007, the individual Parkway bridge inspection reports were reformatted to follow the district-wide style format used for the Turnpike routine bridge inspections, which combined bridges within a maintenance district into one report.

These standardized inspection reports are comprehensive and typically include an executive summary of structural conditions and priority considerations, discussion of bridge deficiencies on an element-by-element and repair category basis, conclusions and recommendations for priority repairs, tabulated deficiency summaries by repair category, photographs of highlighted deficiencies, and an individual checklist form for each bridge. Recommended repairs are categorized as follows: Category A—Emergency, Category B—Contract, Category C—Deck Repair, Category D—Maintenance, and Category E—Monitor.

The "heart" of the Turnpike bridge inspection report is the eight-page individual checklist form for each structure, which is used by the inspectors in the field. The checklist form covers all bridge elements and commonly observed deficiencies that are considered worthy of notation and repair.

The checklist form has been refined and expanded throughout the years to fully describe the findings of a biennial NBIS inspection [FHWA 1995], including sections on approach features, above deck elements and fixtures, substructure elements, superstructure units and attachments, bearings, right-of-way and utilities, FCM elements, fatigue-sensitive details, and underwater foundations [FHWA 2006]. The checklist form is electronically linked to deficiency summary tables which are grouped according to their repair categories. The deficiency summary tables are populated with the recorded deficiencies and locations, and serve as an important tool for NJTA to prioritize maintenance repairs and capital improvements on its structure population.

5 NEXT GENERATION BRIDGE INSPECTION PROGRAM

In an effort to organize and consolidate the assorted databases, NJTA embarked on a pilot project with InspectTech in early 2007, to standardize the inspection report format for both roadways by utilizing their integrated field and web-based BridgeInspect™ Collector software. The main goal of the pilot program was to verify that the software could be efficiently configured and utilized per NJTA's custom specifications on a select group of I-95 Extension bridges on the Turnpike approaching the George Washington Bridge. LS Engineering Associates Corporation (LSEA) was assigned to coordinate the pilot program in conjunction with its NBIS bridge inspection assignment.

The BridgeInspect™ Collector software is designed for the efficient entry, retrieval, analysis, and management of data. A strong advantage of this particular software is its flexibility to be easily customized. The software was tailored to incorporate and link the eight-page individual checklist form (Figure 3), deficiency summary tables (up to 19 tables), and a photograph section

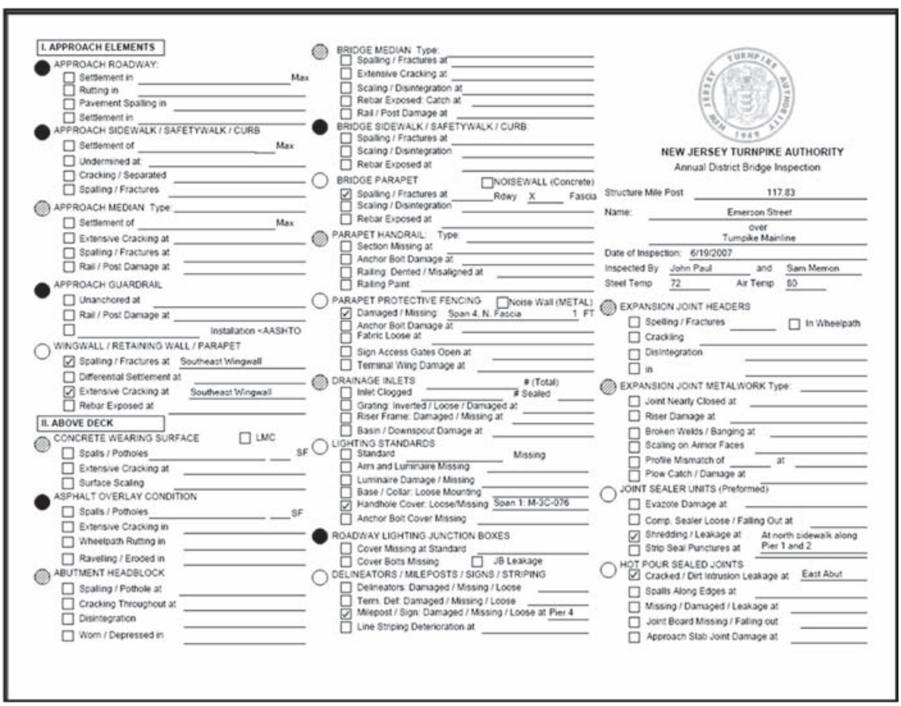

I. APPROACH ELEMENTS
APPROACH ROADWAY:
Settlement in ___ Max
Rutting in
Pavement Spalling in
Settlement in
APPROACH SIDEWALK / SAFETYWALK / CURB
Settlement of ___ Max
Undermined at
Cracking / Separated
Spalling / Fractures
APPROACH MEDIAN Type
Settlement of ___ Max
Extensive Cracking at
Spalling / Fractures at
Rail / Post Damage at
APPROACH GUARDRAIL
Unanchored at
Rail / Post Damage at
___ Installation <AASHTO
WINGWALL / RETAINING WALL / PARAPET
☑ Spalling / Fractures at: Southeast Wingwall
Differential Settlement at
☑ Extensive Cracking at: Southeast Wingwall
Rebar Exposed at
II. ABOVE DECK
CONCRETE WEARING SURFACE ☐ LMC
Spalls / Potholes ___ SF
Extensive Cracking at
Surface Scaling
ASPHALT OVERLAY CONDITION
Spalls / Potholes ___ SF
Extensive Cracking in
Wheelpath Rutting in
Ravelling / Eroded in
ABUTMENT HEADBLOCK
Spalling / Pothole at
Cracking Throughout at
Disintegration
Worn / Depressed in
BRIDGE MEDIAN Type:
Spalling / Fractures at
Extensive Cracking at
Scaling / Disintegration at
Rebar Exposed: Catch at
Rail / Post Damage at
BRIDGE SIDEWALK / SAFETYWALK / CURB
Spalling / Fractures at
Scaling / Disintegration
Rebar Exposed at
BRIDGE PARAPET ☐ NOISEWALL (Concrete)
☑ Spalling / Fractures at: Rdwy X Fascia
Scaling / Disintegration
Rebar Exposed at
PARAPET HANDRAIL Type:
Section Missing at
Anchor Bolt Damage at
Railing: Dented / Misaligned at
Railing Paint
PARAPET PROTECTIVE FENCING ☐ Noise Wall (METAL)
☑ Damaged / Missing: Span 4, N. Fascia 1 FT
Anchor Bolt Damage at
Fabric Loose at
Sign Access Gates Open at
Terminal Wing Damage at
DRAINAGE INLETS # (Total) # Sealed
Inlet Clogged
Grating: Inverted / Loose / Damaged at
Riser Frame: Damaged / Missing at
Basin / Downspout Damage at
LIGHTING STANDARDS
Standard ___ Missing
Arm and Luminaire Missing
Luminaire Damage / Missing
Base / Collar: Loose Mounting
☑ Handhole Cover: Loose/Missing Span 1: M-3C-076
Anchor Bolt Cover Missing
ROADWAY LIGHTING JUNCTION BOXES
Cover Missing at Standard
Cover Bolts Missing ☐ JB Leakage
DELINEATORS / MILEPOSTS / SIGNS / STRIPING
Delineators: Damaged / Missing / Loose
Term. Def: Damaged / Missing / Loose
☑ Milepost / Sign: Damaged / Missing / Loose at Pier 4
Line Striping Deterioration at
NEW JERSEY TURNPIKE AUTHORITY
Annual District Bridge Inspection
Structure Mile Post: 117.83
Name: Emerson Street over Turnpike Mainline
Date of Inspection: 6/19/2007
Inspected By: John Paul and Sam Memon
Steel Temp: 72 Air Temp: 80
EXPANSION JOINT HEADERS
Spalling / Fractures ☐ In Wheelpath
Cracking
Disintegration
In
EXPANSION JOINT METALWORK Type:
Joint Nearly Closed at
Riser Damage at
Broken Welds / Banging at
Scaling on Armor Faces
Profile Mismatch of ___ at
Plow Catch / Damage at
JOINT SEALER UNITS (Preformed)
Evazote Damage at
Comp. Sealer Loose / Falling Out at
☑ Shredding / Leakage at: At north sidewalk along Pier 1 and 2
Strip Seal Punctures at
HOT POUR SEALED JOINTS
☑ Cracked / Dirt Intrusion Leakage at: East Abut
Spalls Along Edges at
Missing / Damaged / Leakage at
Joint Board Missing / Falling out
Approach Slab Joint Damage at

Figure 3. Sample page 1 from individual checklist form.

Figure 4. Overall inspection process flow. 1) Inspections can be started in the field utilizing laptop or tablet computers. 2) Reports can be uploaded to the server module where they are 3) accessible for editing via the web interface from any internet connected computer. 4) When the report is done the inspectors can still print out the same paper reports and submit data electronically.

for each bridge. The photograph section allows the inspector to select and upload photographs from individual bridges, place them in a particular order, and write detailed descriptions for inclusion in the report.

NJTA's pilot program demonstrated that the software is a very user friendly system that can be quickly learned and applied by both bridge inspection consultants and NJTA staff. The database is completely Internet enabled, and installed on laptop computers for use in the field while disconnected. When the laptop is brought back to the office, it can be connected to the network and all information is synchronized with the online edition. When the laptop synchronizes, it also receives the latest program updates and data changes on the server for the next time it is taken into the field. The bridge inspection staff can continue, review, or QA/QC the inspection report from any of their computers. Once completed, the project manager can electronically submit the inspection report to NJTA for approval (Figure 4).

A major advantage with the new software is the allowance of simultaneous data and photo input and review by multiple users, both laptop and web-based, which was unattainable with the prior database. This feature enables multiple prime consultants to work simultaneously with their sub consultants on their respective groups of bridges. Access to the program is controlled through usernames and passwords, which is provided to NJTA bridge inspection staff and consultants engaged in active assignments.

6 FULL BRIDGE ROLLOUT

Based on the promising results of the pilot program (Figure 5), NJTA moved forward with the full utilization of the software for the 2008 NBIS inspections of Turnpike and Parkway bridges. In 2008, over 400 bridges had first cycle reports completed within the system, generating several thousand pages of individual and summary reports. For 2009 this software is being utilized on

Figure 5. Inspection of an I-95 extension bridge included in the pilot program.

the remaining inventory of NJTA's bridges, and by year's end the entire bridge inventory will be entered into the system. The web-based interface is proving extremely valuable, allowing for an integrated team based approach between prime consultants and their sub consultants. Users in different offices can simultaneously review and enter information on inspection reports, increasing productivity and usability. The ability to handle the hundreds of photos entered per assignment, and then link them to individual maintenance needs and bridge elements, is a feature that has proven especially popular with users.

7 INTEGRATION OF ANCILLARY STRUCTURES AND NEW FEATURES

The Authority's ultimate goal is to have all of its key structures integrated within the system for inspection and management purposes. For 2009, NJTA is beginning to enter its inventory of sign structures, culverts, and antenna towers into the system [FHWA 1986a] [TIA 2003]. The software used for bridge inspections will now be utilized by consultants for the inspection of the ancillary structures [FHWA 2005]. Existing inspection forms for cantilever, butterfly and sign bridges, together with new forms for culverts and antenna towers, will be integrated into the system in a similar manner as the bridges. Upon authentication within the system, consultants can now select from a hierarchy of NJTA structures they have access to. When an inspector starts an inspection of a bridge or ancillary structure, the appropriate forms are automatically displayed based on the type of structure being inspected. Another feature currently being implemented into the software is the ability to create a letter which formally notifies NJTA of a Category A—Emergency deficiency discovered during the inspection. This will allow consultants to upload information and photographs for a Category A—Emergency deficiency and notify NJTA via email set up by the software. The integration of this feature into the system will allow NJTA to easily track high priority deficiencies and their resolution.

8 CURRENT AND FUTURE ENHANCEMENTS

NJTA seeks to make the most efficient and reliable process for the collection and management of all of its structure data, and is currently integrating other software programs into the Bridge Inspect™ software to serve as the complete structure inspection and management system. One example is the integration of NJDOT's PontisLite software program [NJDOT 2007]. This is being integrated to facilitate a seamless, automatic transfer of NBI and other data between NJTA, NJDOT and FHWA, which will prevent consultants from exiting the BridgeInspect™ software and entering duplicate data into different software. It will also allow transfer of data through the software and enhance the flow of information between the consultants, NJTA and outside agencies. Another important feature being fully integrated in 2009 is NJTA's Bridge Prioritization System, which permits the generation of detailed bridge repair prioritization reports. Instead of entering data into individual spreadsheets for each bridge, data can be entered and reports run within NJTA's new software.

9 CONCLUSIONS

In summary, the new software allows for the system-wide retrieval of photos, files, inventory and inspection data on the entire inventory of bridges and ancillary structures. This "one stop location" approach will enable multiple users to access and manage current structure inspection data in NJTA's inventory (Figure 6). These and future upgrades will significantly improve quality control measures, provide centralized access for various users, and yield long-term cost savings to the bridge inspection program. The New Jersey Turnpike Authority views its task very seriously in helping to safeguard the lives of millions of users and the large public infrastructure investment, and will continue to diligently apply the best practices to inspecting and managing its structure inventory.

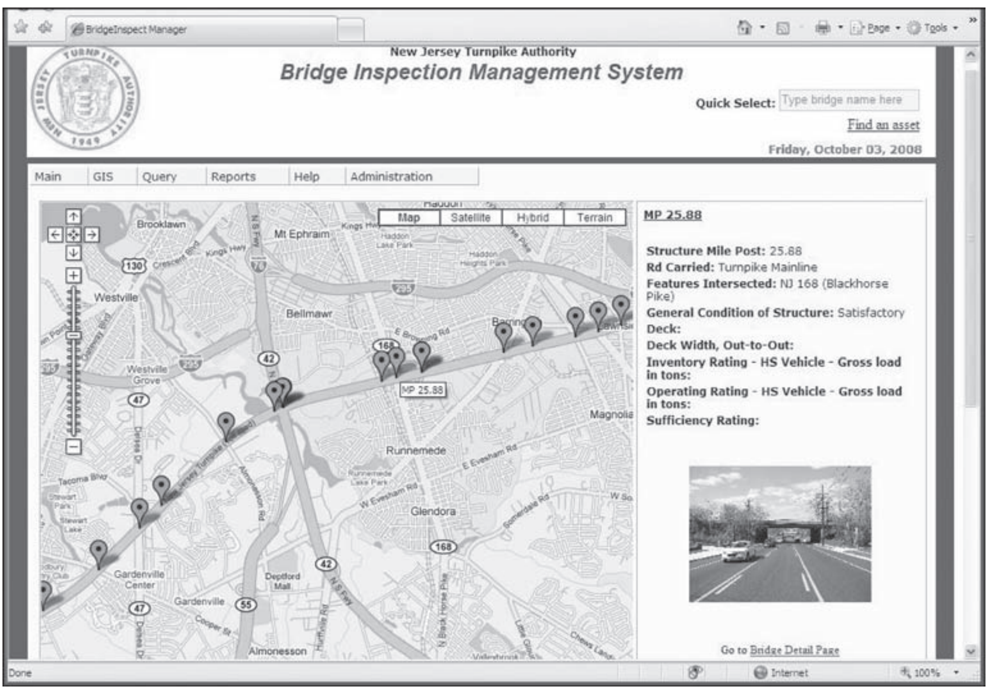


Figure 6. The new structure management system will allow NJTA to easily access all structure information in one place in a variety of formats including interactive maps.

REFERENCES

American Association of State Highway and Transportation Officials (AASHTO), *Manual for Condition Evaluation of Bridges*, 2nd edition with 2003 interim revisions, Washington, D.C.

American Concrete Institute (ACI), *Guide for Conducting a Visual Inspection of Concrete in Service*, 2008, ACI 201.1R-08, Farmington Hills, MI.

Federal Highway Administration (FHWA), *Bridge Inspector's Reference Manual*, December 2006, Washington, D.C.

Federal Highway Administration (FHWA), *Culvert Inspection Manual*, 1986a, FHWA-IP-86-2, Washington, D.C.

Federal Highway Administration (FHWA), *Inspection of Fracture Critical Bridge Members*, 1986b, FHWA-IP-86-26, Washington, D.C.

Federal Highway Administration (FHWA), *National Bridge Inspection Standards*, 23 CFR Part 650, January 2005, Washington, D.C.

Federal Highway Administration (FHWA), *Recording and Coding Guide for Structure Inventory and Appraisal of the Nation's Bridges*, December 1995, FHWA-PD-96-001, and Errata Sheet, March 2004, Washington, D.C.

New Jersey Department of Transportation (NJDOT), *Pontis Lite Users Manual*, 2006 Version 1.1 with February 2007 interim revisions, Trenton, NJ.

New Jersey Department of Transportation (NJDOT), *Recording and Coding Guide for Structure Inventory and Appraisal of New Jersey Bridges*, 2003 edition with May 2008 interim revisions, Trenton, NJ.

New Jersey Turnpike Authority (NJTA), *Bridge Inspection Reports and Program Summary Reports for New Jersey Turnpike and Garden State Parkway Bridges*, 2007 and 2008, Woodbridge, NJ.

Chapter 29

Washington Metropolitan Area Transit Authority's computerized structure inspection system

J.K. Shaffer & M.C. Schellhase
InspectTech Systems Inc., Pittsburgh, PA, USA

ABSTRACT: The Washington Metropolitan Area Transit Authority (WMATA) "DC Metro" maintains a large network of bridge and aerial structures to support our 106 miles of system track. A variety of bridge structures must be maintained and inspected that support over 20 miles of elevated track. The Metro system recently celebrated its 30th anniversary of operation and is increasingly dealing with the effects of an aging infrastructure while also planning for major system expansions. In order to ensure the maximal safety of our nearly 800,000 daily users WMATA has a dedicated workforce of 16 inspectors to conduct annual inspections of our bridges and other structures. This inspection program is significantly different from the FHWA's NBIS and is primarily focused on getting detailed findings and needs of every pier, abutment, span, wall, and other components of our structure inventory.

This paper/presentation will present an overview of the structures maintained by WMATA, including bridges and tunnels, our basic philosophy for inspection and maintenance/management, and our adoption of new technologies and techniques that allow us to effectively address the increasing challenges we are facing. A significant focus will be on our recent adoption of a new system implemented for conducting the inspections and managing the data utilizing tablet computers in the field and integrated network access. The new software system has allowed us to move from a cumbersome paper-intensive process to one in which data is collected electronically and accessible for review, searching, and planning tasks from all of our facilities.

1 INTRODUCTION

The Washington Metropolitan Area Transit System (WMATA) operates one of the world's premier transit systems locally known as 'Metro'. The WMATA rail system serves the greater Washington, DC area including the states of Virginia and Maryland. The system is one of the youngest major transit systems in operation. The first parts of the rail network opened for usage in 1976 and continual expansion has been underway since that time. As parts of the system now exceed over 30 years in age there is an increasing need to be vigilant in monitoring the safety and condition of the structures that form its back bone (Figure 1). For highway bridges, loss of structural capacity can be handled via load posting, allowing the structure to remain open. However, on WMATA's transit bridges all structures must be able to maintain the standard vehicle weights and posting is not an option. Bridges and other structures serve as system critical features. For example, the closure of a single span would effectively result in shutting down service on an entire line (Figure 2). New software has been implemented system-wide to allow it to stay at the leading edge of best practices in structure inspection and management and allow structures to be maintained in the most efficient manner.

As with other agencies WMATA's inspection processes and procedures had evolved into a system that was heavily based on paper forms and a collection of old computer programs. An immense amount of critical information is collected and stored by 16 well-trained on staff inspectors conducting regular inspections on over 5,000 structure assets composed of nearly 10 miles of bridges, 50 miles of tunnel, 87 stations, numerous shafts, parking and bus garages, retaining

Figure 1. WMATA Bridge crossing the potomac river from Virginia into Washington, DC.

Figure 2. Dual elevated metro track structure near Ronald Reagan Washington National Airport.

walls, support systems, and other miscellaneous structures. The Authority realized several years ago that the existing methods for collecting and utilizing the information could be significantly streamlined.

2 PREVIOUS SYSTEM

The old inspection approach of using paper and various computer programs caused many difficulties that WMATA staff identified.

- Paper reports were often difficult to find when needed, especially given the multiple office locations involved.
- Generating the final inspection reports was a time consuming and error-prone process of having to re-type information in the office already written in the field.
- System management of all of the structures and their data was a very difficult challenge. With no central database available there was very limited capabilities to do searching and summary reporting on all of the information on the inspection reports.

- An additional problem was that the picture, sketches, and other attachments were stored in separate locations and often difficult to attach and retrieve with the inspection reports.
- The system lacked an easy to trace accountability process so that changes made could be tracked to individual people when questions arose.
- In addition to lacking inspection software the inspectors also lacked field/tablet computers to take in the field with them and record needed data.

The METRO system recognized these problems along with several others and sought to solve them by implementation of a new software inspection and management system.

3 INSPECTION APPROACH OF WMATA

The current WMATA structural inspection has continued to evolve and improve over time. The authority conducts very detailed inspections on all of its key assets to ensure the safety of the travelling public and the proper maintenance of the structures. To collect all of the needed information these inspections are conducted at a detailed level exceeding typical highway standards.

All bridges are inspected every year and inspections are performed on the individual component/span level. Structures are divided up into their primary components (Abutments, Piers, Spans, and Walls). Forms and data fields are unique to the type of component. For example, when an inspector is performing work on a Plate Girder structure the form and data collected are different than that of a Concrete Box Beam. In total there is a library of 40 different forms that are assigned to each type of structure component (Figure 3). These forms have been developed by WMATA's office of Track and Structures System Maintenance as new structures have been added. Items on each form are rated following the FHWA 0-9 rating code system which in 2006 replaced WMATA's internal rating system which utilized about half the options for each item.

WMATA maintains a team of 16 inspectors split across two divisions (offices) that inspect its structures. For larger bridges multiple (4 or more) inspectors can go out the site and work from different areas completing each inspection by component. All of the individual inspection reports (typically 3–6 pages long) are submitted to an office based reviewer. Progress on the overall structure inspection is monitored by the status of the inspections on all of its individual components.

4 SOLUTION OVERVIEW

After an extensive process the Authority selected the BridgeInspect™ software from InspectTech to be utilized for all of its structural assets. This software coupled with new ruggedized field computers that the authority has acquired for all of its inspectors now forms the core of the inspection process. All paper forms have been converted into new dynamic computer screens that the inspectors can utilize while at the structure site. The process is helping to save considerable time

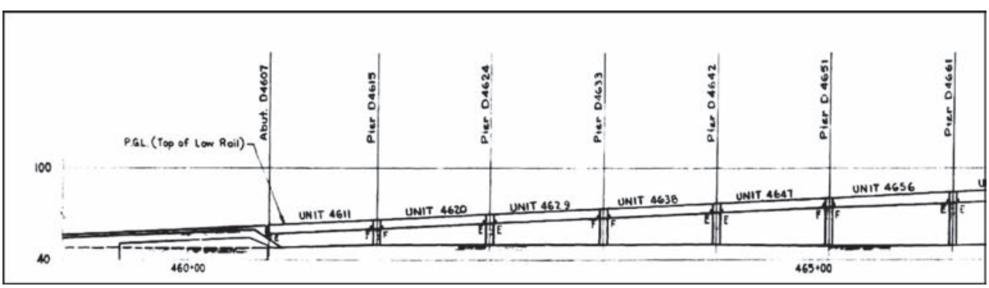


Figure 3. Typical divisions of a structure for inspection. Diagram shows the labeling of each of the primary parts of the bridge: Abutment, spans/units, and piers. For each item an inspection report is created with forms specific to it.

and effort by eliminating the need for cumbersome report typing and data entry while back in the office. Additionally, the software allows for the inclusion of all digital photographs, sketches, and other attachments permitting all information related to the structures to be located in a single location.

The system is composed of three primary parts. The first is the field collection software, which the inspectors have installed on their tablet computers. This software is taken on-site and used to conduct the inspection at the structure. When the inspector returns to the office the tablets can be automatically synchronized with the server. The central server is running the second part of the software: a web-based version of the inspection program that allows reports to be continued or finalized from any desktop computer via an internet browser. Additionally, this module allows the reports to be reviewed and QA/QC to be performed by managerial personnel once the inspectors are completed and before the report is finalized. The third software component is the web-based management software allowing for full searching across any field, historical trending, access to summary reporting, and full structure history with photos, sketches, and other attachments.

5 ORIGINAL PROBLEMS AND SOLUTION DETAILS

5.1 *Traditional problems*

InspectTech has studied the current inspection process extensively through our work with consultants and governments in a variety of areas. Most face a set of common problems for which the BridgeInspect™ Collector Software has been designed to provide solutions. Studying data from public inspection records, it has been shown on average that for every hour of field time two hours are spent in the office. There exist significant capabilities for streamlining the process and freeing inspectors from the clerical office burdens imposed in trying to generate a report. In the process of allowing inspectors to efficiently collect data on-site the software also solves the following specific problems which were identified through studies:

- Duplicate Data Entry: Inspectors are often faced with having to record the same information in different parts of the reports leading to wasted time or worse inconsistent data. The software solves this by allowing data field to be entered anywhere it appears and then automatically filling in all of the others, eliminating the need to re-type the data and providing consistency.
- Lack of Historical Data: Typically, inspectors have copies of the past inspection report. Mostly these are black and white and of poor quality making it hard to compare past pictures with present status. Sometimes past reports are not available or too difficult and time consuming to obtain. The software removes this obstacle by automatically preloading all past information in and color coding it. Inspectors can quickly edit the information and be able to visually determine what has changed.
- Inspection Manual Lookups: Bridge inspectors carry with them inspection manuals that they use in helping to determine ratings for specific items and what procedures to use. These manuals require time to look up the specific entry and are an additional burden to carry out in the field. The software program intelligently links the hundreds of pages of these manuals into the software via drop-down select lists and more detailed links to the full page descriptions displayed whenever a field is clicked.
- Formatting Reports: Typical inspection reports can include a cover page, table of contents, summary narrative data, forms, pictures, sketches, and calculations. The task of compiling the report, especially for inspectors on larger structures, can be unwieldy. By simply entering data into the software the program is automatically able to fill in the correct forms and perform all of the painful and time-consuming tasks that often lead to hours of office time: page numbering, picture layout, table of contents, headers/footers, cover page, etc. This allows inspectors to focus on the tasks of performing the inspection and not wasting time on formatting and report presentation issues.

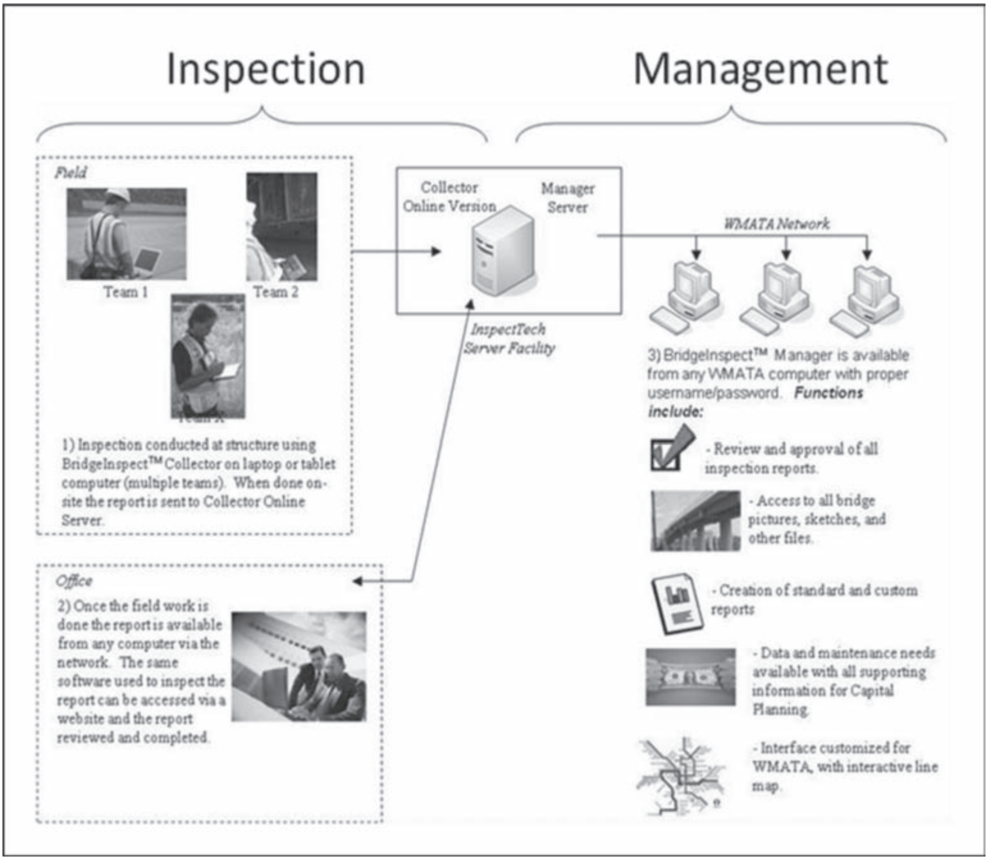


Figure 4. System solution diagram of new WMATA software system.

- Long Approval Process: For some inspectors the review and approval process involves weeks of delay in hand delivering or mailing copies of reports back and forth to managers. Comments are written on the printed versions until the report is approved. The software provides the mechanism to digitally submit the report over any internet connection for review and approval, offering the ability to cut approval time from months or weeks to days. Further it helps to ensure that nothing is missed by tracking each individual report.
- Specify Maintenance Needs and Estimates: nspection results are often directly used by maintenance departments. Inspectors are asked to adequately document repair needs and even develop cost estimates and priorities for each individual need resulting in a time consuming and error prone process. The Collector software allows the inspector to use codes for maintenance items. These items automatically generate cost-estimate guidelines based on the unit and pricing, inspectors can use or chose to change these auto-estimates. Further pictures can be directly linked to each deficiency Total costs and priority breakdown pages are automatically generated in the report from the data entered.

Overall, the process of generating an inspection report is a time consuming one that involves intensive data entry and formatting. These tasks have proven ideal to automate allowing for significant time-savings and quality improvements.

5.2 *METRO solution details*

The BridgeInspect™ software was customized to match WMATA's exact needs. This was a major advantage for the authority as the desired workflow, forms, and roles could be retained. The standard features along with the minor customizations created a system that has been adopted with great success. Some specific aspects include:

- Web Access: With personnel in multiple offices/facilities being able to access the information easily and in real-time via a centralized location was a critical need. The web modules of the system allow this to be done securely and further allow access to be limited by structure or read/write capabilities based on the user's account.
- Easy to Use: Metro personnel desired a very easy to use yet full-featured system; they did not want to create a system that was so cumbersome that it complicated rather than simplified the process. With the BridgeInspect software these needs were met and inspectors were provided with a program that literally was "pick up and go" with minimal training required (Figure 5).
- Flags per component: Every component on a structure can not only be ranked on a 0-9 condition scale, but they can also be flagged according to certain conditions/needs that exist such as Safety Hazard, Engineering Evaluation, Emergency, or Aesthetic. These flags help to drive the corrective actions taken.
- Pictures linked to components: The software has greatly streamlined the process of taking and attaching digital pictures. Pictures can be instantly loaded into the inspection report and also attached to the specific components of the structure that they are related to. This provides significant benefit when trying to track problems over time (Figure 6).
- Videos: The system supports the ability for inspectors to attach videos and other electronic files to the fields. This allows for the display of the effect of heavy dynamic loads on various components. This has already been used to demonstrate how in unloaded situations the structure

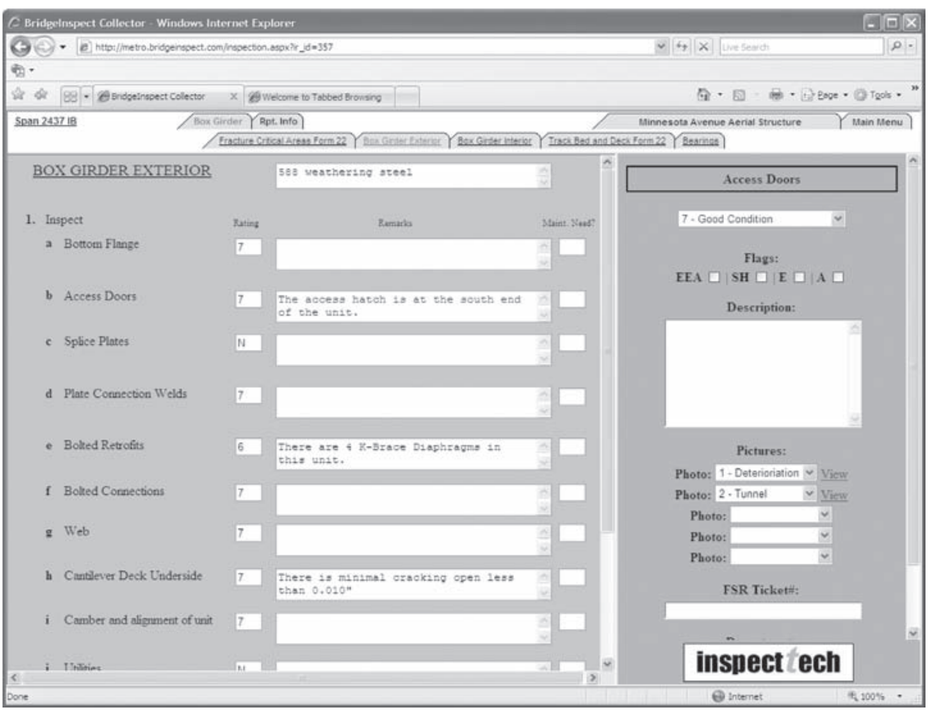


Figure 5. Typical data input screen.

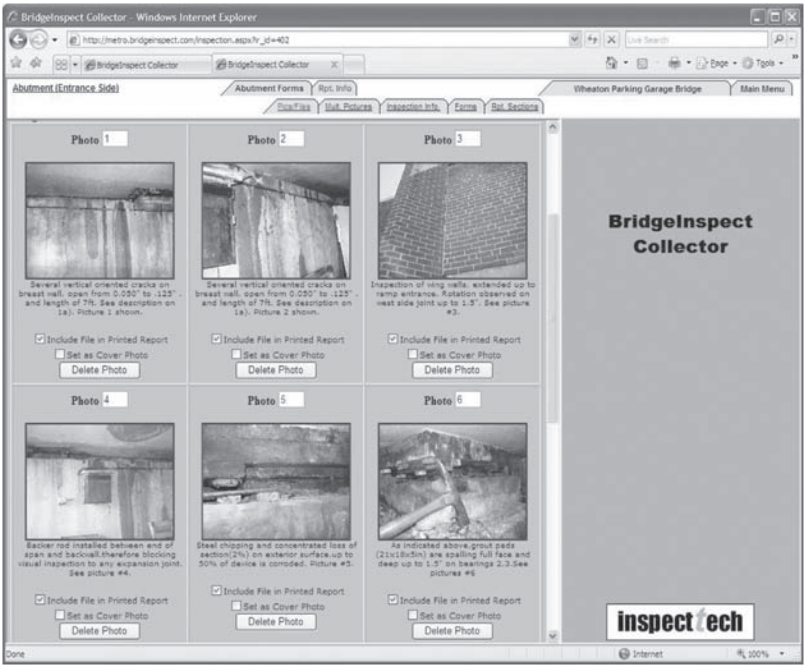


Figure 6. Picture input page showing thumbnails of images that have been uploaded for a bridge.

appears normal, but when a load transverses it's various components begin vibrating and contacting each other out of tolerance.

- Standard Report Output: The same paper reports plus photos and attachments are still available from the system at the click of a button. They are generated as PDF files and can be printed out just as before providing for a consistent reliable format with no work needed by the inspector (Figure 7).
- Summary Reports and Tools: ith all of the inspection data available in a powerful database, a wide range of new summary reports and search tools are now available to WMATA personnel. Line managers have access to reports that show all items under their responsibility that have received flags (i.e. safety hazard) or they can sort by those receiving low condition ratings.

Additionally, there are reports that allow the monitoring of the status of inspections by structure. This helps in scheduling and tracking scheduling work that remains done. Inspections are tracked through each phase of the process from creation/start, to field work, to review, to final approval.

Finally, the software has a built in query/report tool that allows users to create their own dynamic reports based on any adhoc information that is needed. This powerful functionality allows for criteria pertaining to different components to be combined using Boolean logic (and/or) and detailed lists obtained on any structures or components that match that criteria. Now the answer to any question on current system status is literally a few clicks away.

6 IMPLEMENTATION/TRAINING

6.1 *Pilot implementation*

Due to the critical nature of its inspection program and wanting to ensure a seamless transition WMATA utilized a deliberate approach for the project. When WMATA first recognized the need for upgrading to a digital system extensive conversations were conducted on the various options and approaches available. Before committing to an entire system the authority wanted to evaluate the software in a controlled pilot implementation. For this test, two inspectors were selected and a new portable field computer was purchased. Scope was kept very limited since only a single moderate sized bridge structure was selected as the test-bed for performing the pilot on. Only those forms and information pertaining to that structure needed to be implemented. Various options and layouts were also provided on the different forms for review by WMATA personnel.

M metro

WASHINGTON METROPOLITAN AREA TRANSIT AUTHORITY

STRUCTURAL MAINTENANCE INSPECTION REPORT

Wheaton Parking Garage Bridge

Abutment (Entrance Side)

Inspected By: M. Perez

Date Inspected: 5-13-08 Report Generated: 5-29-08

Condition Rating

N - Not Applicable
9 - Excellent Condition
8 - Very Good Condition
7 - Good Condition
6 - Satisfactory Condition
5 - Fair Condition
4 - Poor Condition
3 - Serious Condition
2 - Critical Condition
1 - Imminent Failure Condition
0 - Failed Condition

Inspection Frequency

ITEM	NO.	REMARKS
i. Drains and weep-holes.	4	i)Mud/dirt draining out of 3"D. pipes on east/west side wing walls and breast wall.Collecting up to 2 backets full. Material draining form approach slab.
j. Joints (expansion, construction, cold).	4	j)Backer rod installed between end of span and backwall,therefore blocking visual inspection to any expansion joint.However,joint is leaking water and soil onto bearings causing corrosion and affecting movement See picture #4.
k. Bearings Assemblies (List type and size)	4	k)Pot bearings fix at entrance side.
1. Physical condition.	4	K1)Steel chipping and concentrated loss of section(2%) on exterior surface.up to 50% of device is corroded. Picture #5.
2. Working condition.	4	K2)Working condition is affected by heavy corrosion. Ongoing construction on deck, unable to determine working conditions.
3. Sole Plate.	5	K3)Stable, with some steel exposed due to grout pad deterioration troughout.
l. Exposed / deteriorated re-bars.	6	
m. Concrete Patches.	4	m)Spalled concrete on grout pads. Bearings
n. Grout / masonry pads.	4	1n)As indicated above,grout pads(21x18x5in) are spalling full face and deep up to 1.5" on bearings 2,3.See pictures #6.

THINK SAFETY FIRST

Figure 7. Example output page from WMATA inspection report.

6.2 *Phase 1—Bridges*

After the completion of the Pilot/Trial project the system was judged to substantially improve the existing process. A full-scale system wide-implementation was planned. This full implementation was divided into two phases, approximately equal in size. The first phase covered all of the system's bridge structures while the second phase covering all other structures. For the bridge structures all of the forms and hierarchy of the structures were created and entered into the system. For the initial part of Phase I the new full system was first rolled out and verified on one of the largest bridges which contained a wide variety of structures. For the initial inspection of Phase 1, four inspectors were trained on the software's usage and provided valuable feedback on some final changes needed. After making the final changes the software was installed on all of the 12 remaining inspectors' field computers and a full training session was conducted.

6.3 *Phase 2—All other structures*

Immediately following the completion of Phase 1 (Bridges) the process for Phase 2 began with the implementation of all of the system's other structures ranging from stations to tunnels to parking garages and several other items (Figure 8). As with the bridges, all of the relevant forms and inventory of structures and their components were entered into the system. Once completed and tested the modules were released to the inspectors and they could automatically receive them via their web synchronization.

6.4 *Continued changes*

As the software is being widely used the authority noticed with the new capabilities it can add some additional fields or change entries on long-standing forms to better reflect their needs. These changes are continually being incorporated into the software. With the ability of the synchronization and update features via the web the changes can be made in one central location and automatically propagate to all of the inspectors computers.

Figure 8. Metro screen showing all structure types now present in the system.

7 CONCLUSIONS

This project continues to demonstrate how properly designed software is able to significantly streamline the inspection and management process in organizations of all sizes. The core software by WMATA has also been utilized by numerous highway agencies and consultants in the Washington, DC area and across the U.S. The software met all of WMATA's goals and helped to link together all of the structure information in a single location providing for a flexible tool to address the needs of the Metro system as it continues to age and require more extensive maintenance. By proving easy enough to quickly learn for inspectors of all computer skill levels, it has been able to achieve universal adoption with little resistance. The system is allowing the WMATA inspectors and managers to do the jobs they know best with the tools they need and not have to worry about past problems of managing data and paper reports. Based on its success additional modules related to other tasks that structures personnel must also handle are being planned for future implementation phases.

REFERENCES

United States Federal Highway Administration: Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges (FHWA-PD-96-001): December 1995.

United States Federal Highway Administration: Bridge Inspector's Reference Manual (FHWA-NHI 03-001, 002, 004): October 2002.

Virginia Transportation Research Council 04-CR: Mills, T.H. and Wakefield, R.R.: Modernizing Bridge Safety Inspection with Process Improvement and Digital Assistance.

8 Bridge construction, maintenance and retrofit

Chapter 30

Construction engineering considerations for highway bridges

B. Chavel
HDR Engineering, Inc., Chicago, Illinois, USA

S. Tunstall & R. Eaton
HDR Engineering, Inc., Pittsburgh, Pennsylvania, USA

ABSTRACT: Once a bridge project enters the construction phase, does involvement by a professional engineer cease? After a bridge is designed and let for construction, the engineering of the project does not simply end. There are many different aspects of construction that should be investigated by a professional engineer, working for a contractor, which will provide for safe and efficient construction. The engineering effort during construction typically consists of both design computations and procedures with figures. This paper will draw upon the experience of the authors with regard to construction engineering for highway bridges, by discussing some of the issues a bridge engineer may face when working on the construction side of a project. This paper will examine design items typically encountered during bridge construction, including temporary shoring walls, erection and demolition procedures and calculations, construction loading, temporary support structures, girder bracing, cranes and construction equipment.

1 INTRODUCTION

The ultimate goal of any highway bridge project is the safe and efficient construction of the bridge sub and superstructure. In many cases, especially complex projects, a professional bridge engineer is often hired by the bridge contractor to develop plans and procedures for the bridge to be constructed, and in certain instances demolished. Furthermore, governing specifications are more often requiring the bridge contractor to employ a professional engineer to produce stamped and sealed calculations examining the construction and or demolition phase of a bridge project. The plans and procedures for a bridge construction project should be based on calculations performed by the bridge engineer which may include girder erection analysis, girder stability checks, girder and/or bridge jacking measures, the design of temporary support structures, and the design of temporary excavation support details. In the case of a bridge demolition project items that are typically investigated by the engineer include the deck removal procedure, equipment loads, and girder and system stability.

For these construction issues, the most important design methods and assumptions along with appropriate ways of communicating the information, via procedures and drawings, will be highlighted throughout this paper. Some of the most important aspects regarding bridge construction will be presented, as not every single issue can be presented in a paper such as this. The information provided herein will give the reader a better sense of a bridge engineer's involvement in construction engineering and what type of design calculations may need to be performed during bridge construction and or demolition projects.

2 APPLICABLE SPECIFICATIONS AND GUIDELINES

This section will highlight some of the specifications and guidelines that a bridge engineer working as a construction engineer should be aware of and apply in the development of construction calculations, plans, and procedures.

2.1 *AASHTO specifications and guidelines*

Currently, the governing bridge design specifications are the AASHTO *LRFD Bridge Design Specifications* (AASHTO 2008a), which should be utilized when performing calculations associated with bridge construction. For example, steel I-girder flexural capacities should be calculated in accordance with the provisions of the AASHTO *LRFD Bridge Design Specifications*, when checking girder bending stresses for a given erection sequence. Furthermore, the AASHTO *LRFD Bridge Design Specifications* specifies load factors to be used for construction type loads.

The AASHTO *LRFD Bridge Construction Specifications* (AASHTO 2009) is a companion to the AASHTO *LRFD Bridge Design Specifications*, but focuses mainly on the construction aspects of highway bridges. Construction and fabrication information regarding bridge construction items such as pile driving, temporary support structures, concrete structures, steel structures, and bearings, for example, can be found in the AASHTO *LRFD Bridge Construction Specifications*.

Two other documents that are useful references for construction engineers are the AASHTO *Guide Design Specification for Temporary Works* (AASHTO 2008b) and the AASHTO *Construction Handbook for Bridge Temporary Works* (AASHTO 2008c). The *Guide Specification* contains specifications for falsework, formwork, and temporary earth retaining structures, including data on applicable loads. The *Construction Handbook* is more construction orientated, and mainly focused on standards for materials and construction methods, and providing details with regard to falsework, formwork, and temporary earth retaining structures.

2.2 *State DOT specifications and guidelines*

It should be noted that State Departments of Transportation often have bridge construction guidelines and specifications in addition to, or superseding those by AASHTO. The construction engineer must implement specifications and guidelines that may be unique to a particular state.

2.3 *AASHTO/NSBA collaboration*

Procedures required for general steel erection of highway bridges is provided in the *Steel Bridge Erection Guide Specification* developed through the AASHTO/NSBA Steel Bridge Collaboration (AASHTO/NSBA 2007). This document highlights minimum requirements for the development of steel erection procedures, including steel erection drawings and calculations.

2.4 *ASBI bridge construction practices*

The ASBI Construction Practices Handbook for Concrete Segmental and Cable Supported Structures (ASBI 2008) provides guidance to engineers regarding the construction of segmental concrete bridges. The handbook provides a basic understanding of segmental bridge construction and the construction process for various methods of erection, along with providing lessons learned with the objective of avoiding repeated instances of construction problems.

2.5 *Support of excavation references*

The governing design specifications for designing temporary support of excavation structures on bridge projects are the AASHTO *LRFD Bridge Design Specifications* (AASHTO 2008a). In addition to the *Specifications*, more detail on anchored walls can be found in the Geotechnical Engineering Circular No. 4—Ground Anchors and Anchored Systems (FHWA 1999). For sheet pile wall design, the United States Army Corps of Engineers manual for the *Design of Sheet Pile Walls* (USACE 1994) can be referenced for additional information.

3 DESIGN LOADS

3.1 *Structure dead loads*

Dead loads include the self weight of the bridge superstructure and substructure components or even temporary support structures. For example, dead loads to be considered by the construction engineer include, but are not limited to the bridge girders, detail attachments, deck concrete, substructure concrete, and grillages associated with temporary supports.

3.2 *Equipment and construction live and dead loads*

Construction dead and live loads also need to be considered, which may consist of deck placement machinery (i.e. screeds), contractor's equipment, stock-piled material, deck overhang brackets, formwork for the concrete deck, or other similar attachments applied in the appropriate sequence. Some of these loads are typically considered during the design of the subject bridge, in accordance with the constructability checks provided in the AASHTO *LRFD Bridge Design Specifications* (AASHTO 2008a). However, they may also need to be investigated during the bridge construction by the contractor's engineer.

3.3 *Wind loads*

Wind loads must be considered by the contractor's engineer in the analysis of the steel erection sequence. The calculation of the wind load is typically performed in accordance with the governing specification or owner guidelines. The AASHTO *LRFD Bridge Design Specifications* (AASHTO 2008a) provides wind speeds and loads that can be used to calculate loads on the bridge components during erection. In addition, the construction engineer should discuss wind limits with the contractor, as the wind limits may vary at different stages of construction. For example, the design wind speed is often 32 kmh (20 mph) when a single girder is being erected since it is a temporary condition. However, after more girders are erected, the design wind speed can often be raised to 112 kmh (70 mph).

3.4 *Earth pressure and surcharges*

In construction engineering, earth pressure and surcharge loads are of utmost importance when evaluating temporary support of excavation designs, for staged construction of substructure units such as abutments, or when installing foundations for superstructure temporary support structures. A typical construction-type live load surcharge is one that can result from a crane working at the inside edges of an abutment, for example. Not only does the construction engineer need to be aware of the erection methods and sequence to be used by the contractor, he/she needs to be aware of the soil and/or rock properties associated with the construction site, typically available in project geotechnical reports. The AASHTO *LRFD Bridge Design Specifications* (AASHTO 2008a) provides design provisions for calculating earth pressures and surcharge loads that should be utilized by the construction engineer.

4 TYPICAL CONSTRUCTION DESIGN ISSUES

4.1 *Temporary support structures*

Temporary support structures are often used during the erection of a bridge superstructure, independent of the structure type, i.e concrete or steel superstructure. A temporary support structure, as shown in Figure 1, can reduce bridge girder displacements and stresses during erection, as well as help to control the geometry of the partially erected superstructure. For any bridge

Figure 1. Temporary support structure under a single I-girder.

superstructure erection project, whether it is a simple span prestressed girder bridge or a complex curved and skewed steel plate girder bridge, the control of the geometry at each stage of erection is of vital importance. Temporary support structures typically consist of an upper grillage, vertical and bracing members as part of the "tower," substantial foundations, and lateral supports. Furthermore, the vertical load applied to the tower may change throughout the bridge erection process, depending on the sequence of the construction. The construction engineer must ensure that the load capacity of the temporary support structures are sufficient and the support structure remain stable throughout the entire erection process.

4.1.1 *Types of support structures*

The temporary support structures, often called a falsework tower, can consist of actual designed members, or be selected from a supplier's catalogue. For a designed temporary support structure, the vertical legs of the tower may consist of W-shapes or hollow steel shapes, while the horizontal and internal bracing members may consist of angles or WT sections. The loads that may need to be supported will consist of superstructure dead loads, construction dead loads such as wood formwork, construction equipment live loads, and horizontal loads. For temporary support structures selected from a supplier's catalogue, the construction engineer must ensure that the assumptions used by the supplier are applicable to the actual use of the temporary support. In many cases, supplier support structures may not be able to resist horizontal loads, and additional measures of providing lateral support may need to be designed by the construction engineer.

4.1.2 *Upper grillage*

The upper grillage typically refers to the framework at the top of the temporary support structure, often where the superstructure will bear. An upper grillage, as shown in Figure 2, often consists of a temporary bearing assembly, bearing lateral supports, jacking areas, and supporting beams. The bearing assembly may consist of a typical bearing assembly, such as an elastomeric bearing pad, or consist of steel plates and a vertical jacking device. The grillage supporting beams often consist of rolled beam, W-shapes. The supporting beams are often governed by web crippling, web yielding, as opposed to flexural behavior. Additionally, another concern for these supporting beams that must be considered by the construction engineer is with regard to a "rolling over" effect of the support beams due to longitudinal or transverse loads.

4.1.3 *Foundations*

Depending on the size of the temporary support structure, the vertical loads the towers must resist, and the bearing material below, a foundation of some type may need to be designed by the construction engineer. In many cases, this foundation may consist of a simple timber mats or a concrete pad, similar to a spread footing. However, some heavy-duty temporary support structures

Figure 2. Upper grillage of a temporary support structure.

Figure 3. Example of a temporary support structure lateral support (circled).

may require an equally large foundation consisting of a large spread footing or even steel piles. The construction engineer must verify that the bearing material below the temporary support structure is capable of supporting the applied loads.

4.1.4 *Lateral support*

Temporary support structure lateral loads can result, for example, from wind, horizontal curvature of the superstructure, skew of the superstructure, and seismic events. These lateral loads must be considered by the construction engineer in the design of the temporary support structures. As stated previously, in many cases, supplier-type support structures may not be designed to resist lateral loads, and therefore addition lateral restraint, as shown in Figure 3, may need to be provided.

4.1.5 *Procedures*

If temporary support structures are used, the bridge construction or demolition plans and procedures must clearly state where, and at what stage, the temporary supports are to be used. The locations of the temporary support structures must be shown on the construction plans to ensure that the contractor can place the structures at the proper locations. In cases where geometry control during erection is very important, elevations at the top of the temporary support structures can be provided in the plans. Additionally, details of the temporary structures, including the upper grillage, bracing members, bearing assemblies, and any assumptions (such as wind loading) should also be provided in the construction or demolition plans.

Figure 4. Deck overhang bracket.

4.2 *Deck overhang brackets*

Article 6.10.3.4 of the AASHTO *LRFD Bridge Design Specifications* (AASHTO 2008a) dictates that the effects of the forces from deck overhang brackets acting on the fascia girders shall be considered. (See Figure 4 for a deck overhang bracket.) These effects can impart significant lateral forces on the girder web and top flanges. In cases of deep girders, where the bottom point of the overhang bracket does not meet the bottom flange/web junction, the contractor's engineer may be required to perform calculations ensuring the web is not overstressed or displaces significantly due to the out-of-plane loading or if temporary support of the web at the overhang bracket reaction is required.

4.3 *Support of excavation*

Various structure types are available to provide temporary excavation support. Support of excavation is often used during the construction of bridge foundation elements, independent of the structure type. In addition, support of excavation may be required to provide access for construction equipment such as erection cranes or for temporary support structures. The AASHTO *LRFD Bridge Construction Specifications* (AASHTO 2009) provides details regarding the construction of earth-retaining systems such as sheet pile walls, soldier pile walls, and anchored types of walls. Five different types that are used extensively are presented in this section.

4.3.1 *Cantilever soldier pile and lagging walls*

Cantilever soldier pile and lagging walls are used where the maximum exposed height for the excavation is approximately 4.9 m (16 ft) or less, where competent rock is close to the bottom of the excavation, or where the supporting soils are extremely dense/contaminated with obstructions. Typical spacing between soldier piles ranges from 1.2 m to 2.4 m (4 ft to 8 ft) depending on the type of lagging and size of the soldier pile being used. A significant advantage to this type of support system is that standard construction materials such as rolled steel beams, unreinforced cement concrete and timber are used to construct the wall. The construction of the wall can proceed in a linear fashion with multiple construction activities occurring simultaneously. For this wall type, vertical holes are augured into the existing ground, steel sections (typically HP's or W's) are lowered into the holes, the steel sections are then cemented into place. Once the sections are placed, excavation proceeds downward along the face of steel members, while lagging (typically wooden timbers) is placed behind the flanges spanning between the steel members. This wall type can be very efficient since the steel sections and lagging materials can be reused many times.

4.3.2 *Cantilever sheet pile walls*

Cantilever sheet pile walls are used where the maximum exposed height for the excavation is approximately 4.9 m (16 ft) or less, where competent rock is well below the bottom of the excavation, and where the supporting soils are reasonably free from obstructions. This type of system is widely used since standard steel sheet piling materials and construction equipment are used to construct the wall. The construction of the wall can proceed with two simple activities occurring in a successive fashion. The steel sheet piling is driven forming a continuous supporting element. Once all the sheet pile sections are placed, excavation proceeds downward along the face of sheeting. This wall type can be very efficient since the steel sheets can be reused many times.

4.3.3 *Anchored soldier pile and lagging walls*

Anchored soldier pile and lagging walls are used where the maximum exposed height for the excavation is greater than approximately 4.9 m (16 ft), where deflection control at the top of the wall is critical, where competent rock is close to the bottom of the excavation, or where the supporting soils are extremely dense/contaminated with obstructions. The construction of this wall type is identical to the cantilever wall except that soil/rock anchors are installed at various depths as the downward excavation proceeds. The number of anchor rows is determined to "balance" the anchor loads and force effects on the vertical steel members. Horizontal walers are usually installed at each anchor level so that the anchor can be located between the vertical steel members. Typical vertical anchor spacings range from 2.4 m to 3.6 m (8 ft to 12 ft) with the top anchor located a distance equal to roughly 80% of the spacing between interior anchor rows. The exposed height of the wall is limitless. This wall type can be very efficient since the vertical steel sections, walers and lagging materials can be reused many times.

4.3.4 *Anchored sheet pile walls*

Anchored sheet pile walls are used where the maximum exposed height for the excavation is greater than approximately 4.9 m (16 ft), where deflection control at the top of the wall is critical, where competent rock is well below the bottom of the excavation and where the supporting soils are reasonably soft and free from obstructions. The construction of this wall type is identical to the cantilever wall except that soil/rock anchors are installed at various depths as the downward excavation proceeds. The number of anchor rows is determined to "balance" the anchor loads and force effects on the vertical steel sheet piles. Horizontal walers are usually installed at each anchor level so that the anchor can support numerous vertical sheet piles. Typical anchor vertical spacings range from 2.4 m to 3.6 m (8 ft to 12 ft) with the top anchor located a distance equal to roughly 80% of the spacing between interior anchor rows. The exposed height of the wall is limitless. This wall type can be very efficient since the steel sheets and walers can be reused many times.

4.3.5 *Soil nail walls*

Soil nail walls are used where the maximum exposed height for the excavation is greater than approximately 4.9 m (16 ft), where other wall types are not feasible or where the temporary construction can be incorporated into the final permanent construction. This system is not recommended where sandy soils are present since "washing" of the soil from behind the wall during construction can cause loss of support and global instability. The wall is constructed by excavating vertically while constructing a concrete facing and installing soil/rock anchors at various depths as the downward excavation proceeds. The theory is similar to that used for MSE walls where the soil mass behind the wall is stabilized using the anchors while the facing supports the existing soils between anchor locations. This wall type is usually not economical unless it can be incorporated into a permanent application since all of the materials are lost at the end of the construction.

4.3.6 *Procedures*

If temporary support of excavation is used, procedures must define the locations of the walls, the elevations along the top of the walls, the elevation for each row of tie-back anchors, the elevation of the bottom of excavation, the materials specifications, etc. The construction sequence must be defined to explicitly state when anchors must be installed, specify the anchor loads and the sequence of stressing.

4.4 *Lifting beams*

Lifting beams, also called spreader beams, are used by bridge contractors to assist in the hoisting of materials, equipment, reinforcement cages, or girders. Lifting beams can be fabricated new for each specific project, or maybe utilized on a variety of projects by a particular contractor over the course of time. A lifting beam can be as complex as a telescoping truss, as shown in Figure 5a, or be simply fabricated from a W-shape (Figure 5b) or a hollow steel section. A lifting beam will typically be used to provide two pick points on the item to be hoisted, therefore reducing the bending stress in that particular item.

The size and type of lifting beam required will depend on the weight and length of the item to be hoisted. For a heavier lift, a larger beam section is often required; while a longer lifting beam will allow for the hoisting of a longer item, or girder as shown in Figure 5. The construction engineer must consider the interaction of axial forces, bending moment, and shear in the design of a lifting beam. Furthermore, lifting lugs need to be designed, as this defines where the lifting cables will attach to the lifting beam. Proper hole size, edge distance, shear tear-out, and bearing all need to be considered in the design of the lifting lugs. Additionally, the construction engineer must also properly design the connection (typically welded) between the lifting lugs and the beam. Lastly, if the contractor wants to use a lifting beam that has been used repeatedly on other projects over time, the contractor needs to verify that the subject lifting beam is in a sufficient condition to be employed again.

In the construction plans and procedures, the construction engineer should provide drawings of the lifting beam so that it can be fabricated. Additionally, the construction engineer should provide, in the plans, the hoist load capacity of the beam to ensure that it is not overstressed while used by the bridge contractor.

4.5 *Cranes*

Cranes come in various types and sizes. Each crane type has specific advantages and disadvantages depending on variables such as pick weight, pick height and radius, number of picks, site access, site constraints, etc. The following four crane types are used consistently in typical bridge erection. Additional information regarding all crane types can be found in the NSBA *Steel Bridge Design Handbook* (NSBA 2007).

Mobile hydraulic cranes are used for light- to medium-weight picks up to 650 tons. These cranes are used where the construction site is readily accessible via existing roadways, where pick heights are relatively low, and where crane area is limited. A typical application would be in the replacement of an existing grade separation bridge. Hydraulic cranes come in a wide variety of sizes such that the appropriate crane can be used for the given pick weight and space availability. The set-up and tear down is quick through the use of telescoping hydraulic outriggers. In addition, the mobility and reach is versatile due to the telescoping boom and 360 degree rotational capability.

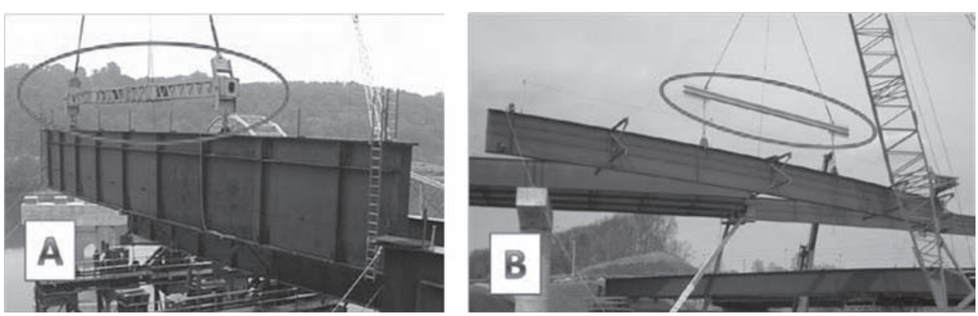


Figure 5. Lifting Beams, a) Truss type, b) W-shape beam type.

Mobile lattice boom cranes are used for light-to medium-weight picks up to 272 t (300 ton). These cranes are used when the site is accessible via existing roadways and where pick heights are high. Through the use of telescoping hydraulic outriggers and self-assembly capabilities, the set-up and tear down is quick compared to other crane types, generally one to two days for assembly of multiple trailer loads. In addition, the reach is versatile with 360 degree rotational capability.

It should be noted that mobile lattice boom cranes and mobile hydraulic cranes cannot move once the pick is lifted.

Lattice boom crawler cranes are used for light-to medium-weight picks up to 272 t (300 ton). These cranes are used where the site is typically unfinished terrain and where pick heights are high (up to 122 m (400 ft)). A typical application would be in the erection of a new bridge over a gorge containing a stream. These cranes come in a wide variety of sizes such that the appropriate crane can be used for the given pick weight and height requirements. The set-up and tear-down is long and labor intensive due to the number of components that must be site-assembled to meet hauling and site access restrictions. Once assembled, the mobility and reach is versatile with 360 degree rotational capability. In addition, crawler cranes are able to travel with the load lifted.

Lattice ringer cranes are used for heavyweight picks up to 1270 t (1400 ton). These cranes are used where the site is typically unfinished open terrain and where pick heights are high (up to 122 m (400 ft)). Typically, once assembled the crane is immobile due to the track work used to support the massive counterweights. A typical application would be the erection of a new bridge over a wide river or bay where the crane could be mounted onto a barge. By mounting the crane on a barge, it becomes mobile increasing it versatility and ultimately making it more productive. The set-up and tear-down is long and labor intensive due to the number of components that must be site-assembled to meet hauling restrictions.

4.5.1 *Pick radii and pickpoints*

The pick radius is the horizontal distance from the rotation center of the cranes boom to the lifting point on the load. This dimension is critical in determining the allowable lifting capacity associated with a crane. As the radius increases, the lifting capacity decreases. When locating the position of the crane during the erection process, it is critical to determine the maximum pick radius and the associated lift weight to ensure the crane has the necessary capacity to perform the operation. In addition to the crane location, the delivery location of the material is vital. The maximum lift radius is often controlled by the material delivery location do to the physical constraints associated with the construction site.

4.5.2 *Crane capacity*

The lifting capacity of a particular crane is a function of the pick radius, boom length, magnitude of counter weight, outrigger extension, load location, etc. As noted above, as the pick radius increases the cranes lifting capacity decreases. In addition, as the boom length increases the cranes lifting capacity decreases. These two variables are vital in sizing the crane for every application. Each crane manufacturer provides lifting capacity charts for various configurations of boom length, pick radius, magnitude of counter weight, outrigger extension and load picking location. The manufacturer's load charts typically have a factor of safety associated with them ranging from 20 to 25 percent of the static tipping load. Typical crane capacities were discussed previously for each particular crane type.

4.5.3 *Crane mats*

Crane mats are a series of timber or steel members assembled in sections beneath the crane to provide a level sound base to support the crane during lifting operations. Most often large timbers are used, since they are widely available, inexpensive and easily transported and assembled. If existing ground conditions are poor (such that proper support of the crane would be marginal), the use of a more complex system of steel framing or other custom designed site-cast crane pad could be implemented to span the poor soils.

4.5.4 *Crane access*

Crane access to the construction site can influence the type of crane selected for the project, the cost associated with the bridge erection and the time (schedule) needed to perform the bridge erection. If access to the site is poor, the cost and schedule associated with the bridge erection will increase. Reasons for the increases could be multiple assemblies/tear downs associated with movements of the cranes do to physical constraints. An example of this would be erecting a bridge over a gorge carrying a stream where the stream can't be traversed by the crane. The crane would need to be assembled on one side of the stream, used to erect part of the bridge, disassembled, moved to the other side of the stream, re-assembled and used to erect the remainder of the bridge.

4.6 *Bridge/girder jacking*

There are many reasons why jacking of a bridge may be required including to provide temporary support during erection of the bridge, to properly align the structure during the initial erection process, and to replace/reset bearings during the service life of the bridge. There are numerous devices that can be used to accomplish the jacking operation. Frequently, hydraulic jacks are used in combination with various temporary support structures or jacking frames to accomplish the task as described below.

4.6.1 *Vertical jacking*

Vertical jacking is typically used to replace existing bearings and provide allowances for removing temporary support structures during bridge erection.

To replace existing bearings a viable method of lifting the existing bridge must be determined. Often the existing bridge can be lifted by locating a series of jacks directly beneath the main girders/members. For this condition, the existing member must be checked for bearing at the new reaction location above the jack. The jack is sometimes located directly on the existing substructure element if enough bearing seat is available. If the existing substructure unit can't accommodate the jacks, temporary support structures can be constructed adjacent to the substructure. The temporary support structures must be capable of supporting the jacking loads including verification of the foundation pressures on the existing soils below. On girder bridges, another method that is used quite often is to place the jacks beneath the diaphragms at the substructure locations. If this method is implemented, the diaphragms must be checked for the bearing forces imposed by the jacks, the connections between the diaphragms and the main girders must be verified for capacity and the existing bridge seat must be analyzed for bearing on the concrete.

To accommodate removal of temporary support structures at the completion of bridge erection, the jacks must have enough vertical stroke to allow all of the temporary reaction to be transferred into the bridge members. If the displacement capacity of the jack can't relieve the entire temporary reaction in one motion, the jacking system must be designed to permit resetting of the jack at different elevations so that the load can be incrementally removed from the temporary support structures.

4.6.2 *Longitudinal jacking*

Longitudinal jacking can be used for a number of different situations during the bridge erection process. One reason for jacking a bridge longitudinally is to align bearings of a continuous multi-span girder bridge with multiple consecutively fixed piers. For this situation, the location of the bearings will only align if the erection temperature is exactly equal to the temperature used for the design (typically 68 or 70 degrees Fahrenheit). Longitudinal jacking devices can take on many forms, usually the less complicated the device the lower the cost associated with the jacking operation. One type that has been used successfully in the past consists of two hydraulic jacks used in combination with a steel frame around the bearing as shown in Figure 6. In Figure 6, the hydraulic jacks allow for movement of the bridge in either direction while the jacking loads are reacted by the existing substructure unit.

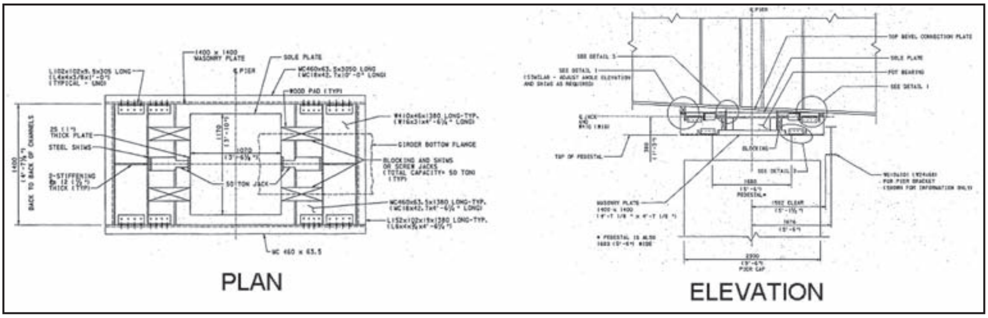


Figure 6. Example of a longitudinal jacking device.

4.6.3 *Jacking procedures*

When jacking is used on a project, a detailed jacking procedure must be developed by the construction engineer. The procedure must identify the exact stages for the insertion and removal of the jacks; the jacking loads at each location; the type of loading that is permitted on the bridge during the jacking operations; and the jack capacities and allowable bridge displacements. This information must be coordinated with the contractor performing the operations to ensure that all aspects of the procedure are possible. Sequencing of the work is vital to verify that the analytical model of the structure matches the actual structure at the various stages of construction.

4.7 *Bridge erection*

Even though a bridge has been designed to accommodate all possible loading conditions in its final condition, the strength of the bridge may not be adequate for temporary conditions that can arise during erection. For example, in a plate girder bridge, the final bridge will have typical cross frame spacings in the range of 4.6 m to 7.6 m (15 ft to 25 ft). Temporary conditions, however, can include cross frame spacings several times larger and in some cases, such as the first girder being erected, will not have any functioning cross frames within its span. The resulting long unbraced lengths can have a tremendous effect on the buckling capacity of the girder. In addition, during girder erection the deck is not present so any additional stability and strength provided by the deck cannot be relied upon. Therefore, although the loading may be many times lower than what the bridge will see in its final condition, the buckling strength may be reduced by an even larger amount.

It is imperative that the construction engineer examine each stage of erection to ensure the structure is stable during all stages. Both the member demands and member capacities can vary significantly from stage to stage as the bridge is erected. For many structures this requires the creation of a finite element model to examine each different stage of erection. Additionally, the member capacity should be determined for each stage based on the actual structure configuration to ensure it can handle the demands at that stage.

4.7.1 *Single girder buckling*

For plate girder bridges, one of the most critical stages during girder erection is when the first girder has been erected but the second girder in the cross section is not yet in place. Without any additional bracing, the unbraced length for the girder will be the distance between supports, assuming the girder is adequately braced at its supports. For spans of any reasonable length, the girder will not be stable for lateral torsional buckling if the crane is released and the girder is left to stand on its own. With unbraced lengths of this magnitude, the lateral torsional buckling strength can be reduced to nearly zero. To stabilize the girder, temporary hold cranes, additional brace points using tie downs, temporary support structures, or compression flange stiffening trusses are often used.

Figure 7. Tie-down at end support.

4.7.2 *Tie downs at supports*

When the first girder of a multi-girder system is erected, the girder must be adequately restrained (tied down) at the supports to ensure it does not roll over. The tie downs typically extend from the top flange of the girder to the top of the substructure at an angle (see Figure 7). The tie downs must be designed to resist wind loading and any dead load forces due to girder rolling from effects such as curvature.

4.7.3 *Dead-Men tie downs for brace points*

If a bridge member is unstable, a wire rope (or cable) can be tied to the member to provide support or a brace point. The wire rope must be tied off to something that is both strong enough and stiff enough to provide the resistance needed to support and/or brace the bridge member (see Figure 8). Often times, a solid block of concrete is used to tie the wire rope to (often called a dead-man). For example, when a single plate girder is erected and found to be unstable, a tie down is often used to provide a lateral brace point for the girder thereby increasing its lateral torsional buckling capacity, by reducing the unbraced length.

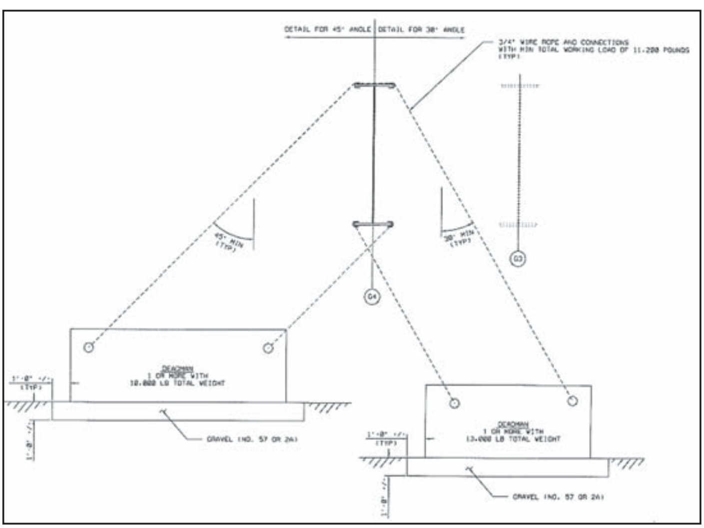


Figure 8. Dead-Men tie-down for brace point.

4.7.4 *Temporary hold cranes*

In many cases a temporary hold crane is required during the erection process. A hold crane can serve several purposes such as: providing a vertical load to limit vertical displacements and out-of-plane rotations; or providing vertical load to limit the vertical bending demands on the structure. For example, a hold crane is especially important for a single erected girder that has a long unbraced length, as the load imparted on the girder by the hold crane can reduce the total vertical bending moment of the erected girder caused by its own self-weight. These hold crane loads can be determined via the finite element analysis used to investigate the steel erection sequence.

Furthermore, it should be noted that in accordance with the AASHTO *LRFD Bridge Design Specifications* (AASHTO 2008a), a hold crane should not be considered as a brace point during erection. In most cases, a hold crane does not provide the necessary lateral support to be considered as a brace point.

4.7.5 *Twin girder buckling*

With two girders erected in a span with sufficient cross frames installed to ensure single girder buckling between the cross frames will not occur, the two girders may still be unstable when examined as a system. This phenomenon, referred to as global buckling, is not a typical design requirement specified in the codes (such as yielding, local buckling, and lateral torsional buckling). Failures have occurred in the past as a result of global buckling. For example, a single box girder bridge in Marcy, New York failed during deck placement from global buckling. Global buckling is the two girder system buckling laterally as a group. In papers by Yura (Yura 2007) and Yura et al. (Yura et al. 2008), the global buckling phenomenon was studied and an equation was developed to determine the global buckling capacity of a pair of girders. Although the most critical application for this equation is for two-girder structures during deck placement, temporary two girder systems during bridge erection should be examined to ensure that the system is stable.

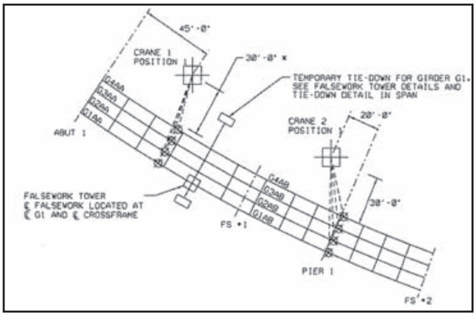
4.7.6 *Cross frame bracing*

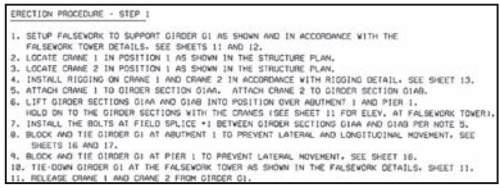
The presence of cross frames during girder erection is a critical issue. The contractor typically prefers to save as much of the cross frame installation as possible until after the bulk of the girder erection has been completed. The construction engineer, however, needs to balance this goal with the need for cross frames to provide girder brace points such that lateral torsional buckling is prevented. In addition to buckling, a sufficient number of cross frames must be installed to ensure the girders are not overstressed due to lateral wind loads. Yura (Yura 1998) provides a method to check the cross frames in a girder system to ensure they are adequate to properly brace the girders. The procedure checks both the strength and stiffness of the cross frames since both conditions must be met to ensure the cross frames are adequate to brace the girders.

4.7.7 *Compression flange stiffening truss*

Another method of temporarily increasing the flexural capacity of a girder, is using a compression flange stiffening truss, as shown in Figure 9. A stiffening truss is a lateral truss that is temporarily

Figure 9. Example of a compression flange stiffening truss.





ERECTION PROCEDURE - STEP 1

1. SETUP FALSEWORK TO SUPPORT GIRDER G1 AS SHOWN AND IN ACCORDANCE WITH THE FALSEWORK TOWER DETAILS. SEE SHEETS 11 AND 12.
2. LOCATE CRANE 1 IN POSITION 1 AS SHOWN IN THE STRUCTURE PLAN.
3. LOCATE CRANE 2 IN POSITION 1 AS SHOWN IN THE STRUCTURE PLAN.
4. INSTALL RIGGING ON CRANE 1 AND CRANE 2 IN ACCORDANCE WITH RIGGING DETAIL. SEE SHEET 13.
5. ATTACH CRANE 1 TO GIRDER SECTION G1AA. ATTACH CRANE 2 TO GIRDER SECTION G1AB.
6. LIFT GIRDER SECTIONS G1AA AND G1AB INTO POSITION OVER ABUTMENT 1 AND PIER 1. HOLD ON TO THE GIRDER SECTIONS WITH THE CRANES (SEE SHEET 11 FOR ELEV. AT FALSEWORK TOWER).
7. INSTALL THE BOLTS AT FIELD SPLICE #1 BETWEEN GIRDER SECTIONS G1AA AND G1AB PER NOTE 5.
8. BLOCK AND TIE GIRDER G1 AT ABUTMENT 1 TO PREVENT LATERAL AND LONGITUDINAL MOVEMENT. SEE SHEETS 16 AND 17.
9. BLOCK AND TIE GIRDER G1 AT PIER 1 TO PREVENT LATERAL MOVEMENT. SEE SHEET 18.
10. TIE-DOWN GIRDER G1 AT THE FALSEWORK TOWER AS SHOWN IN THE FALSEWORK DETAILS. SHEET 11.
11. RELEASE CRANE 1 AND CRANE 2 FROM GIRDER G1.

Figure 10. Sample stage of a bridge erection procedure with portion of write-up.

attached to the compression flange of the girder. The stiffening truss significantly increasing the lateral stiffness of the compression flange, which increases the flexural capacity of the girder for lateral torsional buckling and wind loading.

4.7.8 *Procedures*

Bridge erection procedures must clearly convey all of the assumptions made during the analysis of the bridge erection. Procedures consist of a written procedure, a plan view of the bridge showing crane locations for each stage, and details. The written procedure must sequentially step the contractor through each stage of erection. Any requirements and limits must be clearly spelled out. For example, the wind load limit during erection must be provided. This limit should correspond to the assumption made during the erection analysis. The plan view should show each crane position for each different pick along with the crane requirements for each pick such as boom length, crane counterweight, pick radius, etc. See Figure 10 for example.

Each pick should be clearly labeled and referenced in the written procedure. Requirements such as hold loads, temporary braces, temporary support structures, cross frames to be installed, and stiffening trusses must be clearly shown and detailed. The details associated with erection procedures may include tie down details at supports, temporary bracing details, temporary support structure details, and stiffening truss details. The details must clearly convey the information necessary for the contractor to properly fabricate and construct the items, and perform the sequential bridge erection.

4.8 *Bridge demolition*

Bridge demolition can be accomplished in several ways including using explosives or excavators and cranes. The most common method of demolishing a typical bridge consists of using an excavator to remove the deck and using cranes to remove/disassemble the girders.

4.8.1 *Deck removal and staging*

When removing the deck, the deck slab is typically saw cut into manageable sized pieces to allow removal of each piece. An excavator is often placed on the deck to pick the slab pieces. The excavator must have adequate capacity to make the deck slab picks. In some cases, when the area below the bridge is open, a hoe ram is used on the excavator to break up the deck slab and allow it to drop to the ground below the bridge. When the excavator or any other equipment is on the deck it must be ensured that the deck slab is adequate to handle the weight of the equipment including consideration of any saw cuts and other damage. In some cases, the tracks of the excavator can be kept directly over the beam lines to limit the load on the deck.

The steel girders or concrete beams must be examined to ensure they can adequately support the excavator and other equipment on the bridge (such as a triaxle hauling debris away). Whether or not the deck slab is composite with the girders is an important feature for the girder checks. When the deck slab is saw cut transversely, the composite action between the girders and deck slab is lost, if the girder was originally composite. For this reason, non-composite girders are oftentimes easier to remove since cutting the deck slab does not theoretically effect the strength of the girders (although some level of composite action will most likely be present). For composite girders, however, cutting the deck slab significantly reduces the girder capacity and often requires making the transverse saw cuts as the excavator progresses off of the bridge rather than making all saw cuts up front. In addition, as the deck slab is removed the lateral support it provides disappears and lateral buckling of the girders can become an issue.

4.8.2 *Girder removal*

Once the deck slab is removed, the girders must be removed. The girders are typically removed using a crane. However, sometimes in cases such as concrete beams, the beams may be broken at midspan and allowed to drop onto the ground below. In these cases care must be taken to ensure that structures and other features to remain are not damaged such as substructure units or roadways below the drop area. When the girders are picked using a crane, the girders must be checked for the support conditions that exist during the pick.

As girders are removed from a bridge, it must be ensured that the remaining girders do not become unstable. Similar checks as discussed earlier in the erection section must be examined for a girder removal procedure, such as single and twin girder stability. If a girder is determined to be unstable at any stage during demolition, then appropriate measures must be taken to stabilize the girder, such as adding tie-downs. The cross frame removal sequence must also be examined to ensure they do not create an unstable girder when removed. For example, the contractor may wish to remove the cross frames along with the deck removal procedure, but the engineer needs to determine if they can be removed without creating an unstable girder condition.

4.8.3 *Procedures*

Demolition procedures can be divided into several distinct components including deck removal procedures, girder removal procedures, and substructure removal procedures. The features of a girder removal procedure are similar to an erection procedure as described earlier such as the plan view showing all stages of girder removal and crane locations for each stage. The deck slab and substructure demolition procedures are additional requirements for demolition.

For deck slab removal procedures, a written procedure must be provided along with supporting figures. Each stage of removal should be clearly shown. The deck slab removal should be shown in both plan view and a cross section view. All saw cuts should be clearly called out including the depth of the saw cut. It is important to call out the depth of saw cut because when the saw cut depth is not examined prior to cutting, the top flange of the supporting girders is oftentimes accidentally cut. This can be a serious problem since it could make the girders unstable and cause premature failure or in cases where the girders are to remain in the final structure the girders will need repaired. The allowable locations of the excavator and other equipment on the deck slab must also be clearly shown.

The substructure removal procedures are typically less involved than the other procedures but will vary project to project depending on the substructure type and amount being removed. The procedures will typically consist of a written procedure along with any supporting figures as required. If only a portion of a concrete unit is being removed, a saw cut will typically be used to provide a clean break between the old and new concrete. As demolition approaches the saw cut, the removal will typically switch to using handheld tools to prevent damage to the portion to remain. For typical bridges there are usually minimal calculations involved with substructure removal procedures.

5 CONCLUSIONS

Some of the different challenges that a professional engineer, working for a contractor, may face during construction have been presented. Each different project will present its own unique set of challenges all of which cannot be addressed in this paper. The most significant point presented through this paper is the importance of an engineer's involvement in the construction process rather than trying to examine every possible situation. Both codes and specifications are increasingly requiring the engineer to be involved in the construction process instead of only the upfront design. As we continue to stretch the limits of design, the engineer's involvement in construction is becoming a necessity.

Possibly the most important aspect of an engineer's involvement in the construction process is communication. Communication between the contractor and his engineer is crucial for the success of a construction project. Without ongoing communication, the engineer often proceeds with design without fully understanding what the contractor prefers to do. When this happens, the engineer invariably ends up re-designing features of the project after learning that the assumed method is not the contractor's preferred method. This makes it difficult for the engineer to complete his tasks on budget and on time. So, for almost any project, one of the first and most important things to be done is for the contractor and his engineer to sit down and come up with a plan of attack. Then, as things change and evolve, continued communication will ensure everyone is on the same page and result in a successful project. Communication in the opposite direction is just as critical. If the engineer creates a flawless method to erect a bridge but does not properly convey the information to the contractor, the upfront work might have just been wasted and the results can be disastrous.

REFERENCES

AASHTO. 2008a. *LRFD Bridge Design Specifications, 4th edition, 2008 Interims.* American Association of State Highway Transportation Officials. Washington, D.C.

AASHTO. 2008b. *Guide Design Specification for Bridge Temporary Works, 1st Edition, 2008 Interims.* American Association of State Highway Transportation Officials. Washington, D.C.

AASHTO. 2008c. *Construction Handbook for Bridge Temporary Works, 1st Edition, 2008 Interims.* American Association of State Highway Transportation Officials. Washington, D.C.

AASHTO. 2009. *LRFD Bridge Construction Specifications, 2nd edition*, *2008 Interims.* American Association of State Highway Transportation Officials. Washington, D.C.

AASHTO/NSBA Steel Bridge Collaboration. 2007. *Steel Bridge Erection Guide Specification.* American Association of State Highway Transportation Officials/National Steel Bridge Alliance. Washington, D.C.

American Segmental Bridge Institute (ASBI). 2008. *Construction Practices Handbook for Concrete Segmental and Cable Supported Bridges*. ASBI. Phoenix, Arizona.

FHWA. 1999. Geotechnical Engineering Circular No. 4—Ground Anchors and Anchored Systems. United States Department of Transportation, FHWA, Washington, D.C.

NSBA. 2007. *Steel Bridge Design Handbook, Chapter 13—Design for Constructability*. (available at http://www.steelbridges.org/)

USACE. 1994. *Design of Sheet Pile Walls*. Department of Army, USACE, Washington, D.C.

Yura, J.A. 1993. "Fundamentals of Beam Bracing", *Is your structure suitably braced?—Structural Stability Research Council Annual Stability Conference Proceedings*, Milwaukee, WI, April 6–7: 1–1.20.

Yura, J.A. and Widianto. 2005. Lateral Buckling and Bracing of Beams—A Re-evaluation After the Marcy Bridge Collapse. *Proceedings of Annual Structural Stability Research Council Conference.* pp. 277–294.

Yura, J., Helwig, T., Herman, R. and Zhou, C. 2008. Global Lateral Buckling of I-Shaped Girders. *Journal of Structural Engineering*. 134(9): 1487–1494.

Chapter 31

Concept and practice of accelerated bridge construction in California, USA

P. Chung & J.J.Q. Fang
California Department of Transportation, Diamond Bar, California, USA

K.J. Thompson
California Department of Transportation, Sacramento, California, USA

ABSTRACT: The concepts of Accelerated Bridge Construction (ABC) offers significant advantages over the conventional cast-in-place construction in minimizing traffic disruption, improving work zone safety, and reducing on-site environmental impacts. For these reasons, as one of the world leaders in infrastructure planning and construction, recently years, California Department of Transportation (Caltrans) has already embraced the ABC philosophy in bridge design and construction projects. In this paper, the ABC practices in California over the last five years have been review, and lessons learned are presented. The new concept in developing an ABC project such as construction impact index, impact tolerant index and project delivery index are proposed and analyzed, and the design impact questions are developed for quantifying the index. A planned ABC project located in Los Angeles urban area is described in this paper.

1 INTRODUCTION

In California more than 24,000 bridges support one of the world's most vibrant economies link the nearly 45,000 miles of pavement. In the current 21st century, this highway network massively built in 1950's has failed to meet the fast-paced traffic demands due to the population increase. Recent years the severe congestion in statewide has adversely affected the economic growth. Local, regional and state transportation partners grapple constantly with congestion relief strategies. The vast majority of structures constructed in California are cast-in-place concrete that require substantial construction effort and extended construction completion time. This conventional project delivery processes in place for decades, which include planning, design and construction, are no longer acceptable norms in congestion relief projects. The state-of-the-art concept, accelerated project delivery (APD) that towards delivering infrastructure projects sooner offers a feasible tool for transportation planners, engineers and contractors to reduce overall impacts to the motoring public while completing improvements quickly. Typical project delivery schedules are controlled by construction of bridges, which require the most time to complete. ABC, as a main component of APD, has been widely recognized as utilizing prefabricated bridge components in bridge planning and design to substantially reduce on-site construction time. The careful planning, design, and implementation for an ABC project offers significant advantages over onsite cast-in-place construction in lowering costs, improving safety in job site, and mitigating environmental impact due to minimized construction time.

The Federal Highway Administration (FHWA) has been actively promoting the advantages of accelerated bridge construction. A framework of decision-making in developing ABC project utilizing prefabricated bridges components has been established [1]. Caltrans continues to strive for innovative ways to expedite project delivery while maintaining stewardship of entrusted public funds. Over the last few years, working with academic institutions and industry, attempts have been made to investigate viable alternative engineering practices to achieve ABC. The researches and practices concern with project planning, type selection, design criteria, and project management and performance assessment, and pilot ABC projects. This paper presents the efforts made to develop ABC concept and practice, which includes the ABC decision making process, and a brief introduction of a representative project in urban area.

2 LESSONS LEARNED REPORT AND SURVEY

In the first phase of ABC implementation in California, the Caltrans ABC Advisory Council has conducted "lesson learned" survey for seven ABC projects completed and planned in the last five years. These seven bridges covered the most structural types implemented in current ABC project, and demonstrated the most state-of-the-art of ABC technologies. In this "lessons learned" survey, the effectiveness and challenges of the various ABC practices in California over the last five years were analyzed [2, 3]. For emergency and non-emergency projects, the recommendations for improving future ABC project delivery were made and summarized as follows.

2.1 *Emergency bridge project delivery*

To response an emergency rapidly and effectively, the ABC Advisory Council suggests:

a. Form an ABC-Delivery Team, which consists of structural design, Geotechnical Services (GS), Structural Office Engineer (SOE), and detailers, to rapidly response to emergence disaster event based on the requirements of the Structural Maintenance & Investigation (SM&I) Emergency Response team.
b. Implement Incident-Command System (ICS) to provide a responsive management system that can be scaled up or down to suit the needs of the project delivery effort.
c. Establish an Executive Management Team (EMT), which consists of Office Chiefs in the Districts and Division of engineering Services (DES), to be responsible for ABC project delivery decisions.
d. Develop and train key staff and engineers to attain and assign quick decision making skills needed to respond effectively to an emergency-recovery effort.
e. Accelerate Plan, specification & Estimate (PS&E) process by setting attainable deadlines for Structure draft PS&E and final PS&E packages, and addendum requests.
f. Streamline addendum request process.
g. Address and develop procedures and criteria for the use of large incentives/disincentives and responsive bid requirements.
h. Expedite shop drawing review process
i. Create a post-event investigation team similar to the Post-Earthquake Investigation Team (PEQIT) to document lessons learned for non-seismic multi-hazard disasters.
j. Improve 'As-built' plans and shop drawings quality and availability.
k. Enhance information technology support.
l. Enhance office support.

2.2 *Non-emergency project delivery*

In California, the standard construction practice is cast-in-place concrete, in which most project planning and design criteria are based this conventional construction. The ABC, therefore, practice typically demands a premium with respect to construction costs. "Lessons learned" survey indicated that the delivery cost of a typical ABC project is expected to be 30%–100% higher than conventional construction costs in California over the last few years.

In the long run, using ABC methods, however, create great economical benefits that potentially offset the construction cost premiums. Conventional bridge construction typically induces traffic delays and congestion for an extended time period (average of 9 to 15 months) which creates significant impact on local transportation and economy. It is recommended that Caltrans initiate a practice development and implementation task force to develop a strategic work plan that provides standards, guidelines, and key policies for implementing structure design for accelerated bridge construction of future projects. The task force would consist of subject matter experts from design, construction, research, maintenance, materials, and other relevant fields. The members will develop much needed research and guidelines, implementation work plans, and promote future ABC practice in California. In addition, the new project delivery cost should be defined by taking into account the public delay costs, direct project costs, and escalation of funding costs.

3 ABC PROJECT DEVELOPMENT METHODOLOGY

Starting in 2008, Caltrans initiated a practice development and implementation for ABC, and has established a task force that is headed by an ABC Executive Committee and Advisory Council to develop standards, guidelines, and key policies for implementing structure design for ABC. Consisting of subject matter experts from design, construction, research, maintenance, materials, and other relevant fields, the task force outlined a strategic plan to develop, implement, and promote ABC practice in California. The significant efforts have been made, the road map of developing a new project has proposed, and criteria for evaluating a project performance has been established for Caltrans review and use [4, 5, 6].

3.1 *ABC Project decision-making*

It is important, though, to understand that the success of ABC implementation rests largely on widespread acceptance of the associated techniques by project development staff (both internal and external), funding partners, and the contracting industry. The Caltrans' larger goal, as stated in its Mission/Vision statement, is "Caltrans improves mobility across California". The goal of ABC is to deliver projects earlier to the travelling public: to effectively reduce the impacts of on-site construction to motorists. Therefore, as stated precedent, ABC is viewed as a subset of a larger APD effort encompassing all aspects of project development through construction contract acceptance. Considerations related to lane rental rates should also be considered as part of this to address funding issues. This latter requirement stems from the fact that quite often new techniques involve unassigned risk that must be borne by the Contractor at a premium until the comfort level garnered from successes has been realized.

ABC means significantly reducing on-site construction time to mitigate traffic impact on public travelling, improving safety, and alleviating the effects on environmental sensitive area. The longer construction duration will induce the in-directed cost due to traffic delay or other impact, herein defined as impact cost (*IC*). Besides the construction cost (*CC*), in structural type selection, therefore, the *IC* should be considered in decision making. Currently, Caltrans ABC Advisory Council suggests a cost objective criterion for total cost (*TC*) estimate as follows:

$$TC = CC + 1C \tag{1}$$

As mentioned precedent, ABC can substantially mitigate the construction impact on the site to benefit the business, commerce and environment. The benefits due to ABC implementation should be taken into account, and the earned value analysis [7] is suggested.

Eq. (1) suggests that, for any construction type, the *IC* should be included in the total cost estimate. In general the construction impacts consist of traffic and environmental impacts. The traffic impact includes all construction related traffic delay or interruption, and the environmental impact includes all environment damage in the environmental sensitive area due to bridge construction. To accurately quantify the *IC*, the construction impact index (*CII*) is defined by impact intensity, which consists of three levels, low or mild, moderate and severe. The details of the construction impact index are as follows (Table 1).

It is noted that, for a project, the longer construction duration, the higher impact will be. The impact induced by construction duration is defined as construction impact time (CIT). Usually the

Table 1. Construction impact index.

Impacts	Intensity level	Description
Traffic	T1-Low or mild	Reduce widths of lanes and shoulder, closure of 1–30% of total lanes and/or shoulder or lane realignment
	T2-Moderate	Closure of 31–66% of total lanes + shoulder
	T3-severe	Closure of 67–100% of total lanes + shoulder
Environment	E1-Low or mild E2-Moderate E3-severe	Site and structural type dependent. To specify the impact level, the environmental analysis is required

Table 2. Impact tolerant index.

Construction site	Tolerant level	Description
Residential community Local streets	High	For a reasonable period, road closure in day and night time may be acceptable
State routes, major city arties	Moderate	Partially lane closure in a day and night time may be acceptable
Interstate or State freeways	Low	No lane closure allowed in day time, partially lane closure in night time may not be acceptable

ABC has shorter CIT than that of the conventional construction. In determining the impact intensity, the ABC will have relative lower intensity level.

There are significant differences in tolerating an "impact" for different construction site. For a local street, a road closure may be acceptable, but for an interstate freeway, one lane closure in day time can create unacceptable traffic impact. The impact tolerant index (*ITI*) of the construction site is defined as tolerant level that scales the capacity of a site to tolerate the impact (Table 2).

The two impact indexes can be quantified using score system in impact cost analysis. The Currently Caltrans ABC-Advisory Council is working on the establishment of the score system for each index. Conceptually, the total impact can be expressed as:

$$Total\ Impact = CII * ITI \tag{2}$$

The impact cost, then, can be estimated based on the total impact. It should be pointed out that the CII is structural type dependent. The prefabricated bridge type, for instance, will have relative less impact on the construction site.

As described in this paper, for some emergence case, such as restoration of the traffic capacity interrupted by accident or incident, the project must be delivered in a very short time. In this case, ABC may be an only viable option to complete the project. For some project, the longer construction duration may be appropriate due to the cost considerations. In order to describe the degree of urgency of the project, the project delivery index (PDI) is defined, and is tabulated in the following (Table 3).

In order to quantify the impact indexes, Caltrans ABC-Advisory Council have developed a number of questions given structure type, which are listed in the following Table 4.

4 ABC PILOT PROGRAM IN CALIFORNIA

In 2008 Division of Engineering Services (DES) at Caltrans started a pilot program that implemented ABC technologies. In this pilot program, the 10 project were first selected. The bridge

Table 3. Project delivery index.

Project type	PDI	Description
Capacity restoration	High	Emergence response, to restore traffic capacity in a very short period (less than 30 day, for example)
Capacity improvement	Moderate	Severe congestion adversely affects the traffic and economical activity, increasing capacity become urgent
Safety improvement & rehabilitation	Low	Retrofit structure to meet updated codes

Table 4. Project delivery: Design impact questions.

Structural type	
Questions	**Scores**
General	No Yes 1 2 3 4 5
Is this an emergency bridge replacement?	
Is bridge on an emergency evacuation route or over railroad/waterway?	
Is there a funding requirement to accelerated project delivery?	
Is rapid recovery from natural/manmade hazards or rapid completion of future planned repair/replacement needed for this bridge?	
Is the bridge construction a critical path of the total project?	
Are there significant economic benefits if construction/project is completed ahead of schedule?	
Traffic	
Bridge carries high ADT or ADTT?	
Bridge over existing high ADT or ADTT facility?	
Bridge construction significantly impact traffic?	
a. Does it have high user-delay costs?	
Can the bridge be closed during off-peak traffic periods?	
Will the traffic control plan be significantly impacted?	
Construction	
Do worker safety concerns at the site limit conventional methods, e.g., adjacent power lines or over water?	
Is the bridge location subject to construction time restrictions due to adverse economic impact?	
Does the site create problems for conventional methods of construction (falsework, concrete delivery, etc.)?	
Utilities	
Are there existing utilities that impact the construction window?	
Are there existing utilities that impact construction operations?	
Environmental	
Is the site environmentally sensitive area requiring minimum disruption (e.g. wetlands, air quality, and noise)?	
Are there natural or endangered species at the bridge site?	
a. Shorten construction window needed?	
Local weather limit the time of year for construction?	
Is the bridge on or eligible for the National Register or Historic Places, or a designated landmark structure?	
Total Scores	

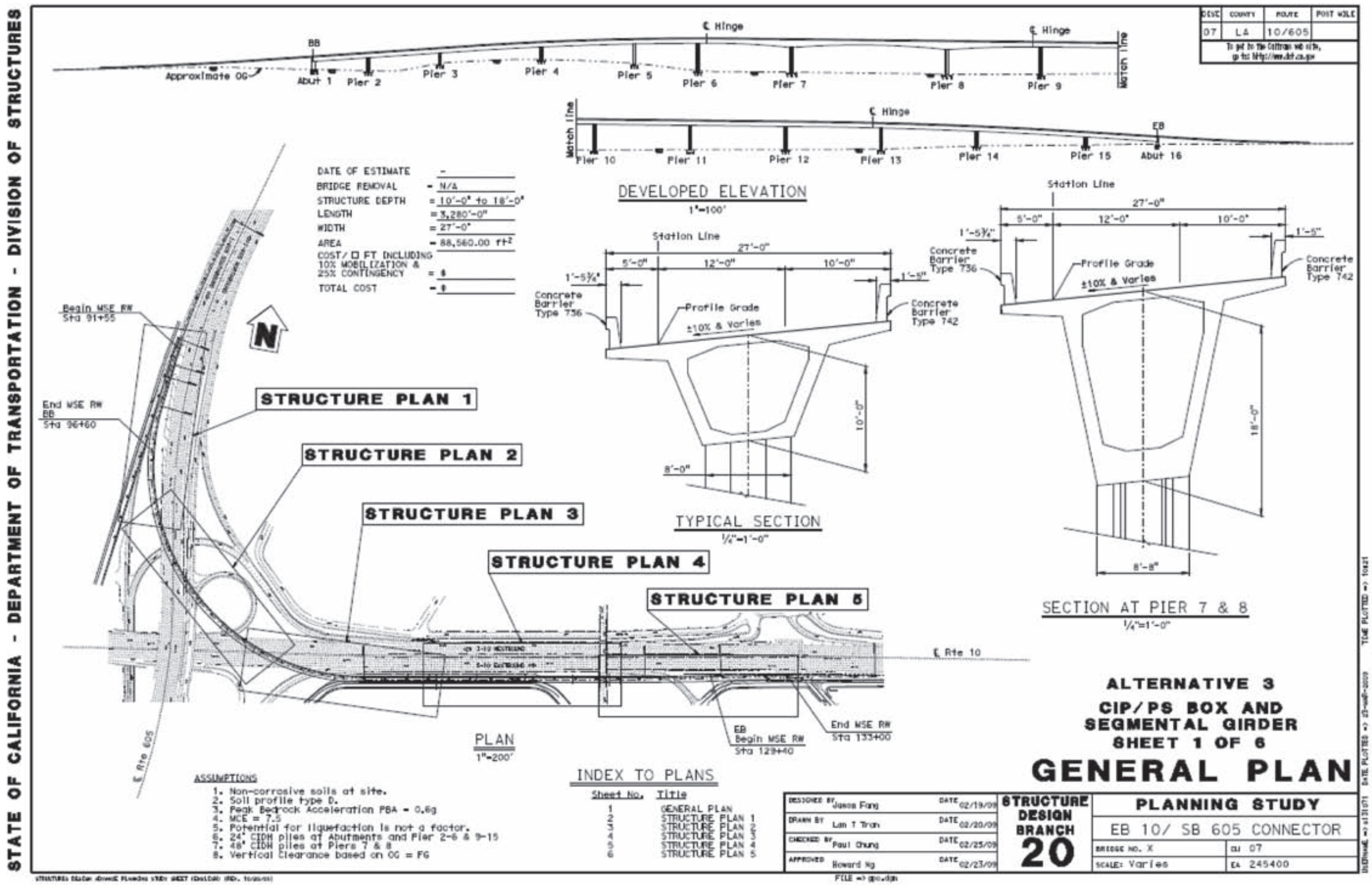


Figure 1. General plan of alternative 3.

types cover most structural types currently utilized in ABC application, which included PC/PS concrete, steel I girder, spliced precast girder, and prestressed concrete segmental bridges. The objectives of this program are to improve the management process and coordination of Caltrans functional units, and develop design procedures and seismic details for ABC bridges.

4.1 *EB I-10/SB I-605 Connector*

The new EB I-10/SB I-605 Connector viaduct is planned for construction at interchange between Interstate Routes 605 and 10 in the eastern portion of Los Angeles County. The existing southbound I-605 to eastbound I-10 connector is an at-grade connector that merges with westbound I-10 to the southbound I-605 Connector and the northbound I-605 to the eastbound I-10 Connector. The joint segments have produced severe traffic weaving conflicts, which has queuing on the outer lane of westbound I-10 and has created many weaving-related accidents and heavy congestion. The proposed project is a 4145 ft long elevated direct connector includes a main flyover structure spanning 3280 ft and two 505 ft and 360 ft ramps, respectively. The proposed flyover connector would provide a direct connection between southbound I-605 to eastbound I-10. The new flyover span will eliminate the existing weaving movement, reduce the queuing of westbound traffic on I-10, provide for improved goods movement, and enhance the safety and operation of the I-10 and I-605 interchange. Construction is expected to start in 2011.

The project site is located on a heavily traveled freeway interchange. Heavy congestion and long delays occur during commuting hours daily. A 4145-feet stretch of elevated connector construction would have a significant impact to the existing traffic on I-10 and I-605. Any further potential traffic delay and congestion will create a severe bottleneck and have undesirable

Table 5. Structural data of planned EB I-10/ SB I-606 alternatives.

Structural type	No. of span	Max span length (ft)	Structural depth (ft)
Steel I girder	16	270	9
Segmental	16	270	9
CIP/PS Box, Segmental hybrid	15	368	10–18 varied

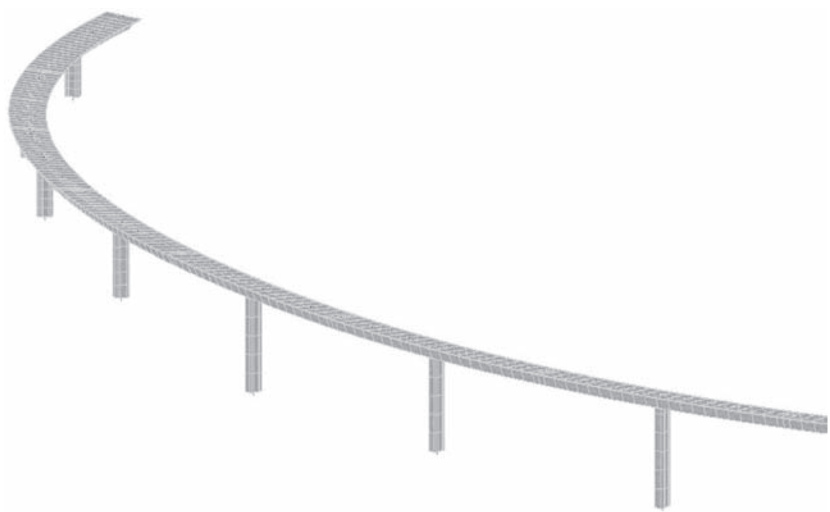


Figure 2. A Portion of RM-2006 model of alternative 3.

economic impacts. Thus, engineers are encouraged to use innovative technologies and techniques to accelerate construction and minimize falsework for the purpose of reducing user delay and community disruption. One of the authors of this paper is the project engineer who is responsible for planning and design. Three alternatives, steel I girder, PC/PS concrete segmental girder, and CIP/PS box and segmental girder hybrid bridges, were proposed. The following Table 5 lists the structural data for all three alternatives.

At 2009 price, the Alternative 2 has highest estimated cost and Alternative has lowest estimated cost. Figure 1 shows the general plan of Alternative 3. The span between Piers 4 and 5, which is over I-605, the progressive segmental construction was planned. From Pier 6 to Pier 9, a three balanced cantilever segmental structure, of which a 368 ft span is over I-10, was proposed. Since CIP portions of Alternative 3 located outside of I-605 and I-10, and large portion are parallel to I-605 and I-10. The preliminary construction impact analysis shows that in this project the difference of construction impact for these three Alternatives to the site are small. Comparison of the estimated cost for three Alternatives, the cost of Alternative 1 is 22.6% higher, and Alternative 2 is 32.9% than that of Alternative 3, respectively.

Using RM-2006, the 3D computer model has been setup for Alternative 3, and preliminary analysis has performed. Figure 2 shows a portion of the RM-2006 model.

5 CONCLUSIONS

In California the economic and population growth, coupled with the decaying infrastructure, have led to the omnipresent need to rapidly replace, widen, and build new highway infrastructure and thus bridge structures and transportation planners are under increasing pressure to improve highway and bridge systems in accelerated time. Over the last few years, Caltrans has already embraced the ABC philosophy for the bridge construction projects. In this paper, the ABC practices in California over the last five years have been reviewed, and lessons learned have been presented. In order to successfully develop an ABC project, the new concept of the construction impact index, impact tolerant index and project delivery index have been proposed and analyzed. The decision making system in developing and managing ABC project is being established in Caltrans ABC-Advisory Council. The Caltrans pilot program of ABC has been briefly introduced, and a typical planed ABC project located in Los Angeles urban area has been presented.

REFERENCES

1. California Department of Transportation, *Caltrans ABC Action Plan Practice and Policy for Bridge Projects*, Caltrans, ABC-Advisory Council, 2009.
2. California Department of Transportation, *Caltrans ABC Strategic Plan-Development of practice and policy for Future bridge projects*, Caltrans, ABC-Advisory Council August 2008 Version 1.1.
3. California Department of Transportation, *Project Management Concepts and Working*, Caltrans, 2008.
4. Caltrans, ABC Advisory Council, *Accelerated Bridge Construction Applications in California, A Lessons Learned Report, August 2008 Version 1.1*, http://www.dot.ca.gov/hq/esc/
5. Chung P., Fang, J.Q. and Wolfe, R.W., *Developing Concept of Fully Segmental Bridge Design for Accelerated Bridge Construction in High Seismic and Urban Areas*, Proceedings of ASBI International Symposium on Future Technology for Concrete Segmental Bridges, San Francisco, CA , USA 2008.
6. FHWA, *Framework for Prefabricated Bridge Elements and Systems (PBES) Decision-Making*, http://www.fhwa.dot.gov/bridge/prefab/framework.cfm
7. Wolfe, R.W., Chung, P. and Dermegerdician, A., *Accelerating Bridge Construction—Precast ABUTMENT Solutions*, Concrete Bridge Conference, 2007.

Chapter 32

The maintenance of the main expansion joints on the Forth Road Suspension Bridge, Scotland

B.R. Colford
Forth Road Bridge, Forth Estuary Transport Authority, South Queensferry, Scotland

S. Jones & D. Timby
Atkins, Bristol, England

K. Brown
Atkins, Edinburgh, Scotland

ABSTRACT: The main expansion joints on the Forth Road Bridge have been in service since the bridge opened in 1964 and in that time traffic volumes and the weight of vehicles crossing the bridge have increased significantly.

The joints are of the rolling plate type whereby a series of plates slide over fixed curved girders. There are eight joints in total and they allow movement of the 1006 metre long main span and 408 metre side spans and are thought to be the largest of their type in Europe.

The joints are now considered to have reached the end of their service life and a decision was taken to replace them. It was deemed unacceptable to close traffic lanes for the period required for the work to be carried out. Therefore, an innovative scheme to construct temporary ramps over the joint locations to allow traffic to pass over the replacement works was devised. However, a combination of a high tender price, due in the main to the high cost of the temporary works required to keep the traffic moving, and the announcement of a proposal to build a new bridge across the Forth to be open in 2016 led to a review of the project to determine if the service life of the joints could be extended until the new bridge opened extending the life of the joints until the new crossing was opened would lead to a saving of some £6 million.

The review was carried out using failure mode and effect analysis (FMEA) techniques. This type of analysis is more commonly used in the aeronautical and process industries than in construction but it was felt that it was the best means of identifying the likelihood and consequences of failure of the various components that make up the joints. Once the failure mode and consequence of that failure of the individual components had been identified an analysis to determine what mitigation could be put in place was then carried out.

The review concluded that the service life of the joints could be extended by increasing inspection and monitoring and by installing failsafe devices without reducing the current high operational safety levels on the bridge. However, it was recognised that there was a risk of a reduction in operational service levels which had to be balanced against the potential cost savings.

1 INTRODUCTION

1.1 *General*

The Forth Road Bridge is a long span suspension bridge and when opened in September 1964 had the fourth longest span in the world. The bridge crosses the Firth of Forth some 15 km west of Edinburgh and is a vital link in Scotland's strategic road network. The bridge site is at latitude 56 degrees north. That is some 1000 miles north of New York City (Figure 1). Over 24 million

Figure 1. Location of the Forth Road Bridge.

vehicles cross the bridge each year. It is a Category A listed structure. This paper describes the maintenance of the main expansion joints on the bridge.

The Forth Road Bridge is owned and operated by the Forth Estuary Transport Authority (FETA).

1.2 *Details of the original construction of the Bridge*

The bridge has a main span of 1006 metres and the side spans are each 408 metres long (Figure 2). The deck on the main span is made up of a 12.7 mm stiffened steel plate overlain with 38 mm thick mastic asphalt, on a waterproofing layer. The deck to the side spans is a 203 mm thick reinforced concrete slab with a similar surfacing detail to the main span. The decks on both the main and side spans are supported on steel stringer beams that span between large steel cross girders spaced at 9.144 metre centres. These cross girders are supported by two longitudinal stiffening trusses, which in turn are supported by the main cables. The main cables are 600 mm nominal diameter and are each made up of 11618 4.98 mm diameter galvanized wires that transfer the loads from the bridge to the main and side towers and also down to the north and south anchorages. The anchorages are tapered rock tunnels, filled with concrete and post tensioned.

Linking the stiffening trusses to the main cables are 192 sets of steel wire rope hangers at 18.288 metre centres that vary in length from 2.5 metres at mid span to 90 metres adjacent to the main towers.

The cables at the main towers, side towers and in the anchorage chambers are seated directly on cast steel saddles. These saddles on the main tower are fixed directly onto each tower leg whilst those at the side towers and anchorages are fixed to steel rocker boxes that are free to rotate in the direction of the bridge axis.

The main towers are of steel box construction rising some 156 metres above river level and are formed from three fabricated steel boxes that are joined by cover plates to provide a five cell structure in plan. The legs of each tower are connected by cross members at the top, and just below deck level, and by diagonal stiffened box bracing above and below the deck.

Figure 2. View of the bridge looking north.

The approach viaducts to the bridge are themselves large steel twin box girders with a reinforced concrete deck slab. The eleven span south viaduct is 438 metres long and the six span north viaduct is 253 metres long.

The design and construction of the bridge is described in the Proceedings of the Institution of Civil Engineers (1965). The maintenance challenges facing the bridge are described by Andrew & Colford and Colford.

2 BACKGROUND DETAILS OF THE EXPANSION JOINTS

2.1 *General*

Deck expansion joints allow for the movement of the bridge structure. Movements on a suspension bridge such as the Forth Road Bridge occur as a result of expansion and contraction of the deck and cables from temperature effects and in response to vehicle and wind loading. Such movement occurs in a combination of vertical and horizontal directions as well as in rotation about these axes.

On Forth Road Bridge in each carriageway of the suspended spans, at both main towers, there are two joints to accommodate movement in the main span of the bridge and the side spans. Therefore, there are in total eight number joints, set within the two carriageways, accommodating movement in all three suspended spans. Each of these joints is of the roller shutter type and they are the original joints to the bridge. This type of joint is able to accommodate the large movements of the deck that occur in a bridge of this type. Information from the ‘as built’ drawings indicate that the joints were designed to have a movement capacity of +810 mm/–920 mm (total 1730 mm) in the main span and +150/–260 mm (total 410 mm) in the side span.

The deck expansion joints comprise a series of units 1232 mm wide, giving a total of six units per carriageway per joint. Each unit has effectively two movement joints, one for each (side and main) span. Each individual joint unit comprises an anchor (or bridge) plate which is articulated on one (deck) side of the joint. This plate effectively spans over the structural gap of the joint itself. Attached to the opposite edge of the anchor plate are a series of link plates, all connected together by hinge mechanisms to form a train. The anchor and link plate train slide, via discrete feet, over the top flanges of curved support beams as the structural gap moves. The pier side of the joint supports a tongue plate. This tongue plate is also supported on the link plate train and the end of the deck to form a smooth running surface for traffic (Figure 3).

2.2 *Construction of the expansion joints*

The construction of the joint means that complete water tightness was impossible to achieve and this has led to corrosion problems and ingress of contaminants. In addition, this type of joint has a reputation of being noisy, particularly when wear becomes advanced in hinges and the mating surfaces between the link plate feet and the surface of the support beams. However, this type of joint is generally of robust construction and has proved to have had a good service life on the Forth Road Bridge as well as on other large bridges around the world.

Roller shutter joints of this type have a typical design life, before major maintenance, of 20 to 30 years and the joints on the Forth Road Bridge are the original ones, which are now almost 45 years old. The roller shutter joints on Forth are known as the Demag joints after the German company that manufactured and installed them (Figure 4). In 1975 the joints were subject to a

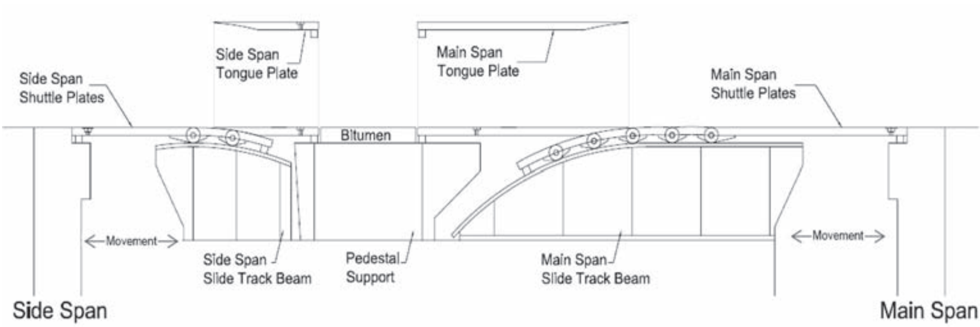


Figure 3. Long section through the joint.

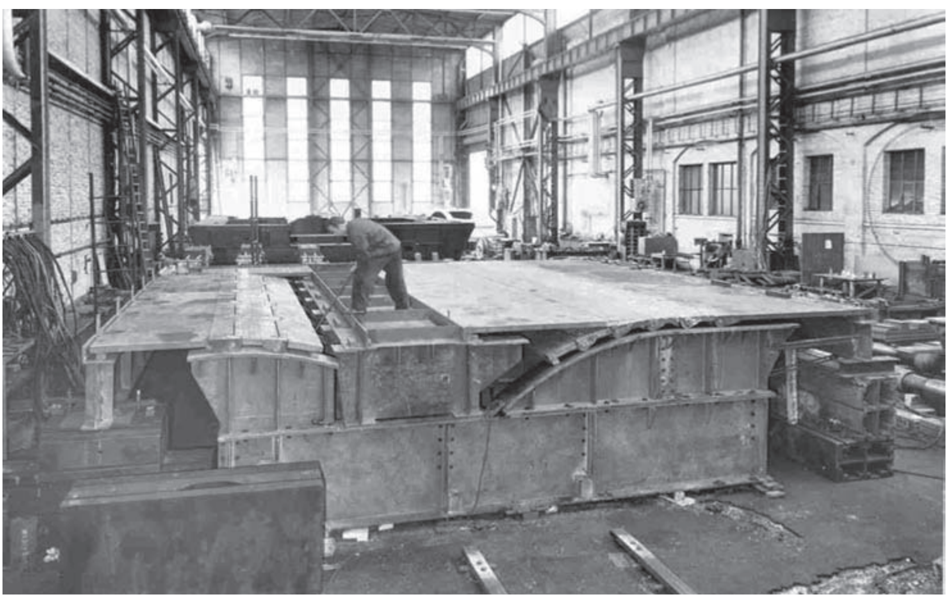


Figure 4. Trial erection.

detailed inspection and overhaul by Demag Lauchhammer. The 1975 inspection reported that the joint was generally performing well, but evidence of wear was becoming apparent. Defects noted included:

– Two of the tongue plates were cracked;
– Some of the link and anchor plates were loose;
– Play was detected in the hinge joint pins;
– The springs to the holding down bolts for the plates were corroded;
– General wear in the interface between the link plates and the support beam;
– The approach carriageway surfacing was uneven.

Since the 1975 inspection FETA have also undertaken further inspection and maintenance work. This has included:

– Re profiling of the curved girder top flanges and feet of the link plates to try to rectify the effects of uneven wear;
– Welding ‘keeper plates’ on the sides of the hinges to prevent the hinge pins from migrating outwards;
– Lifting a joint train and tongue plate to inspect the condition of the sliding surfaces and wear to hinge pins.

More recent inspections by FETA had shown further excessive wear in the curved link plate support beams and in the hinges and feet of the link plates (Figures 5 and 6). The effect of this wear is that the joint becomes increasingly noisy and play in the mechanisms and wear surfaces causes rapidly increasing damage to the joint. Ultimately the joint will break up with safety implications for the travelling public.

In addition to the wear in the joint itself, the traffic surfaces of the steel plates had also become very worn. This had resulted in the wear of the leading edge of the first link plate (Figure 7). In addition, originally the traffic face of the main link plates had grooves cut into them in a diamond pattern to form an anti-skid surface and these have worn away. The steel plates had formed a very smooth surface with a corresponding low vehicle skidding resistance, particularly when wet.

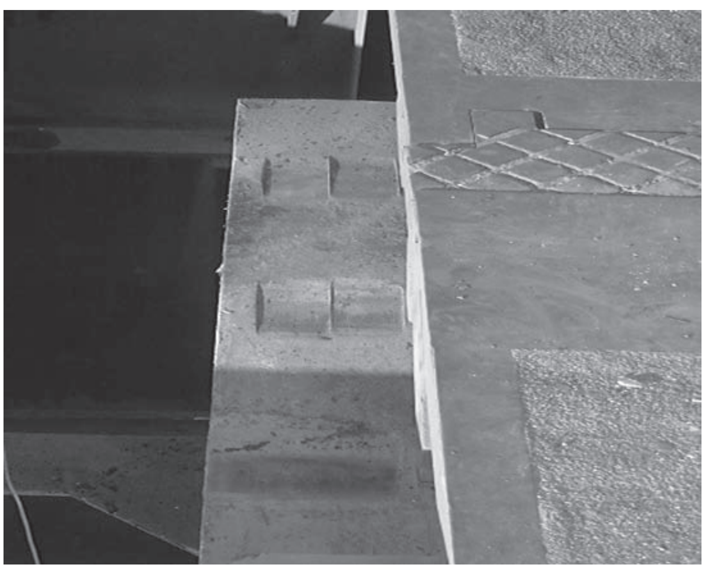


Figure 5. Wear on top flange of curved girder (track beams).

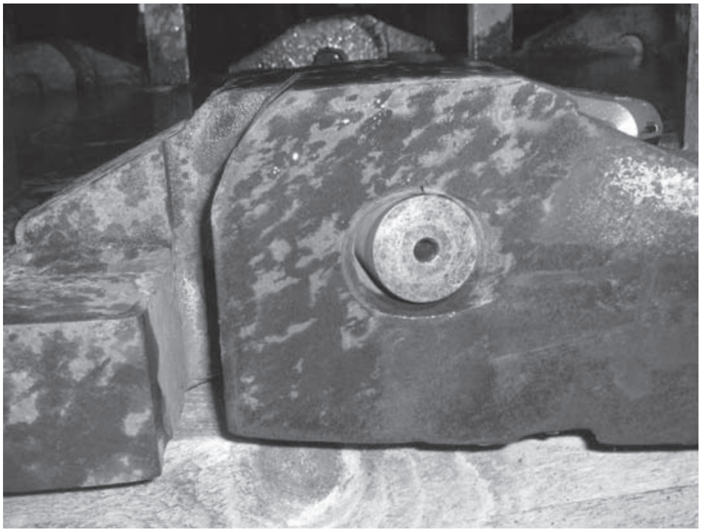


Figure 6. Corresponding flattening of hinge feet; wear in pin hole and misalignment of the pin.

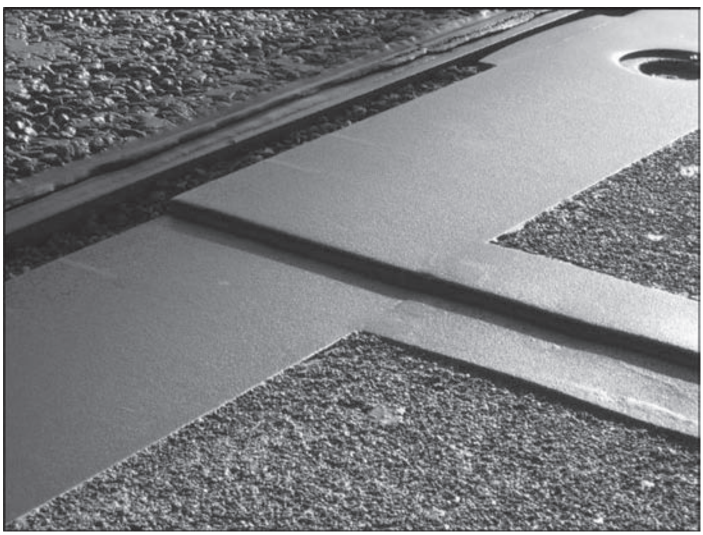


Figure 7. Misalignment between adjacent shuttle plates.

In addition to the suspended span joints, there are further joints in both carriageways of the approach viaducts comprising of interlocking combs (or fingers).

At each carriageway joint in both the suspended spans and approach viaducts there are also corresponding joints in both foot/cycle ways. These comprise steel plates that slide over each other.

All of the existing bridge deck joints have been in place since the opening of the bridge to traffic in 1964 and have generally performed well with regular inspection and maintenance having been carried out. When the bridge was first opened traffic flows were of the order of 4 million vehicles per annum. Last year over 24 million vehicles crossed the bridge and, in addition, the maximum weight of unrestricted heavy goods vehicles on UK roads has increased from 22 tons in 1964 to 44 tonnes today.

Although all of the three types of bridge deck expansion joints have generally performed well, it was recognised that they had reached the end of their service lives and should be replaced or refurbished before becoming a safety and maintenance liability. The replacement or refurbishment of all the expansion joints was recognised as a maintenance priority especially as the rate of wear was likely to rapidly increase.

3 FEASIBILITY STUDY AND DESIGN WORK

3.1 *Consultant procurement and restraints*

In April 2007, following a quality/cost tendering process, the consulting engineers Atkins were appointed to carry out a feasibility study and then provide the necessary design work and contract supervision to allow the refurbishment or replacement of the joints to be carried out.

The brief for Atkins' feasibility study was to consider the options available to either repair, refurbish or replace each type of expansion joint. Factors and constraints to be considered were health and safety, traffic management, programme, access, resources, procurement, future maintenance and budget.

Carrying out essential maintenance work while minimising delays to the public is a key FETA objective. The nearest road crossing of the Firth of Forth to the Forth Road Bridge is some 20 miles upstream at Kincardine. Therefore, any restrictions on Forth Road Bridge will cause significant traffic problems on the network.

In order to minimise the disruption to the road users, the programme for undertaking the works on site needed to consider a number of constraints. The Forth Road Bridge crossing is a strategic route and traffic disruption on the Bridge will affect other roads in the area and also other Firth of Forth crossings. Conversely, works on the other strategic routes in the area will affect the Bridge. Therefore, the timing of the works needed to be coordinated with planned works on other adjacent routes as well as on the Bridge itself. Other factors that would cause additional traffic on the roads, such as large public events (e.g. the Edinburgh Festival), were also considered as was the fact that traffic volumes are generally higher in the summer months.

3.2 *Outcome of the feasibility study*

In September 2007, Atkins produced their feasibility report in which a number of options were considered for each type of bridge deck joint. For the roller shutter type joint, five options were examined. These included continuing with the current maintenance regime; a full refurbishment or a complete replacement with a new roller shutter joint or an alternative joint type. For the sliding plate type joint in the foot/cycle ways two options were examined. These were also continuing with the current maintenance regime or a full refurbishment with improvements to the design. The third type of joint is the comb joint. For this four options were examined. These included continuing with the current maintenance regime, a full refurbishment, a full replacement with a new comb joint or an alternative multi-element steel girder joint.

For the options considered for each type of bridge deck joint a recommendation was made based on minimal disruption to the local road network, value for money and long term costs and maintenance.

- For the roller shutter type joints it was recommended that all of the joints are replaced with a new roller shutter type joint. This option has the advantage over the others in ensuring that the whole joint 'works together' by being able to be manufactured to tighter tolerances than repairing or refurbishing the existing joint. Advantage can be taken of improved harder steels, a more robust link plate hinge mechanism and a better traffic running surface. This option was also likely to result in the least time for traffic management on the highway. Improved access to the joints was considered to be necessary for the works. This could be installed in advance of the main works and it would also serve for future maintenance and inspections.

- For the sliding plate joints in the foot/cycle way it was recommended that the sliding plate joints were fully refurbished with the plates incorporating an anti slip surface for path users and also PTFE pads between the plates to reduce friction.
- For the comb joints it was recommended that, as with the roller shutter joints, they were completely replaced. Refurbishment would practically involve the entire replacement of the joint anyway without the advantage of using improved materials and designs.

To progress the work on the joints, it was necessary to further develop each of the preferred options into detailed designs, specifications and works contracts. The work would also require advance planning for programme, publicity and improvements in access. With further detailed development, and discussions with suppliers and fabricators, more accurate costs and timescales would be calculated to assist with forward planning and procurement.

In considering the safety of the workforce and to minimise the risk of disruption to the work, it was considered preferable to undertake the work in the summer months when the weather was more likely to be suitable. The time periods to undertake each section of work needed to be planned in advance. Allowance had to be made for unexpected/unforeseen difficulties that would cause time over runs. Some of the work operations were particularly weather sensitive, such as painting and repairs to the deck waterproofing.

4 ACTION FOLLOWING THE FEASIBILITY REPORT

4.1 *General*

The feasibility report was comprehensive and thorough and documented that although as much as possible of the work for the replacement of the roller shutter and comb joints would be fabricated off site, installation work on the bridge was estimated to require a closure of the first carriageway of about 8 weeks. For the second carriageway it was hoped that the benefit of experience on the first carriageway would enable both pairs of Demag joints to be replaced at the same time with some savings in the duration of carriageway closures required. The other smaller expansion joints within the viaduct carriageways were also programmed to be carried out during these carriageway closures.

It was recognised that these carriageway closures would cause widespread and significant disruption to the whole of the road network in the East of Scotland.

In terms of carrying out work on the bridge the period April to June was considered the most suitable time in respect of weather and daylight hours available. Traffic flows in the late spring/early summer months are generally below the monthly average before increasing to a maximum in July and August.

Recent work carried out during the Main Cable Replacement/Augmentation Study had shown that the notional road user delay costs of closing one carriageway on the Forth Road Bridge are of the order of £650,000 per weekday. Therefore, every week that is saved from the carriageway closure programme would result in a reduction of user delay costs alone of over £3 million. These costs do not take into account the costs to the wider business community which, as can be seen from the Main Cable Replacement/Augmentation Study, are of a higher order of magnitude than the user delay costs.

4.2 *Provision of ramps*

Given the significance of the disruption and economic cost of the carriageway closures, further work was carried out to try to find some innovative way to determine if these periods could be shortened. However, it is recognised that where these Demag expansion joints have been replaced elsewhere in Europe, longer periods of closures were required and as noted the Demag joints on Forth were thought to be the biggest in Europe.

In addition, Flint and Neill consulting engineers, were asked to carry out a peer review of the work carried out to date and following information received from Flint and Neill, a project team of officers from FETA, and designers Atkins, held a series of meetings and determined that it would be feasible to design and construct temporary ramps over the expansion joints and thus allow traffic flow to continue during replacement work.

The notion of ramping over the joints had been considered earlier as the benefits to minimising traffic delays were evident. However, the difficulty was providing access to the joints for replacement when the ramps were erected. This was solved without the need to utilise barges in the river by extending the span of the ramps past the main tower legs to allow access from the cycletrack/footways to the side.

Erection of the ramps would require carriageway closures to erect and dismantle the structures. However, it did appear feasible to erect a ramp over both sets of joints at each tower within a carriageway during the same closure. The initial discussions indicated that most of the fabrication of the ramps could be carried out off site and the erection and dismantling operations would each require an estimated four days over a long weekend carriageway closure at either end of a period of at least 8 weeks to carry out replacement of the joints on one carriageway. On completion of the replacement works on the first carriageway, the ramps would be dismantled and re-erected on the other carriageway.

The ramps would be significant structures some 80 metres long and would be two lanes wide albeit these lanes would be narrower than in the permanent layout (Figure 8). The headroom under the ramps for working was a minimum of 1.8 metres. The existing bridge deck would have to be strengthened locally to accommodate the ramp at each tower and it is likely that the strengthening could limit the weight of vehicle using the ramps to 3.5 tonnes. Other factors such as headroom below the tower cross bracing, additional wind effects on the elevated ramp and containment issues precluded the accommodation of heavy goods vehicles. Enforcement of any restrictions would need careful consideration and it was likely that a 30 mph restriction would be required.

It should be stressed that any restrictions on the use of the bridge by heavy goods vehicles would apply only to one carriageway at a time. There would be no restrictions on the opposite carriageway. Heavy goods vehicles make up around only 6% of the traffic on the bridge and the split northbound and southbound is approximately 50/50. Therefore, although restricting heavy goods vehicles from using either the northbound or southbound carriageway for a period would have an

Figure 8. Virtual reality model of ramps.

effect on industry, only around 3% of the bridges traffic would be affected by the restriction. The fact that most traffic can cross meant that the diversion routes would not be as congested.

Although the ramps would maintain two lanes in each direction on the bridge it was likely that the very nature of the ramps would tend to reduce the throughput of vehicles, certainly during the first few weeks.

As the ramps are elevated off the carriageway it would be necessary, for the period that they are in place, to review FETA's operational procedures during high winds.

The cost of providing the ramps and temporary strengthening of the suspended span deck to accommodate them was estimated to increase the cost of the contract by between £1 and £2 million. However, despite that increased estimated cost, the FETA Board accepted the recommendation that given the benefits from savings in user delay costs (although notional) and, as important, the significant reduction in the level of disruption to the public, the ramps should be included in the project.

4.3 *Other deck joints*

It was proposed to refurbish or replace the joints in the viaduct carriageways using the carriageway closures provided for the 'Demag' joints. However, adopting the ramps has led to a review of this proposal. Atkins have concluded that although the viaduct joints are the originals, and therefore are also almost 45 years old, because their range of movement is less than the 'Demag' joints, they have suffered less wear and tear and there is no immediate urgency for refurbishment or repair. It is now recommended that these joints continue to be monitored and inspected and their refurbishment or replacement be programmed to be carried out during future carriageway closures, possibly during future resurfacing work.

5 PROCUREMENT OF CONTRACTING SERVICES

5.1 *General*

Tender documents based on a quality/cost ratio of 80/20 were issued in September 2008 for the project which now comprised of the following main elements.

- The replacement of all eight roller shutter joints within both the carriageways of the suspended spans with a similar joint with the same movement range but improved details.
- The replacement of all eight adjacent sliding plate joints within the two cycletracks/footways with a similar joint.
- Provision of two new permanent access platforms to allow improved inspection and maintenance of the roller shutter joints below the carriageway. The design of these elements was also to be the contractor's responsibility.
- The provision of two temporary access ramps (mini bridges) to allow vehicles under 3500 kg to cross a carriageway whilst the joints were being replaced. The design of these elements was also to be the contractor's responsibility.
- The strengthening of the suspended span orthotropic deck and truss to accommodate the loading from the temporary ramps.

The cost of the contract works including the deck strengthening and temporary bridges to carry traffic over the work areas was estimated to be £8.7 million. This sum excludes design and site supervision costs as well as the cost of ancillary works. The total project cost based on an £8.7 million tender was estimated to be £10.3 million.

However, the most economically advantageous bid submitted by Balfour Beatty was for the sum of £13,753,93 and this was some £5 million above that estimated. A significant sum of around £6 million in the tender price was for the provision of the temporary bridges and deck strengthening to accommodate the ramps.

The contract could not be awarded until the issue of funding was resolved and the Authority explored ways of bridging the funding gap.

6 REVIEW OF PROJECT

6.1 *General*

However, during this period in December 2008, the Scottish Government announced the details of the Strategic Transport Projects Review in the Scottish Parliament and within that announcement was the timescale for the building of a second bridge across the Forth adjacent to the existing Forth Road Bridge. Following the Minister's announcement and the commitment to a definite programme for the construction of the Forth Replacement Crossing, a review of the project was carried out to determine whether or not the replacement of the joints could be deferred until 2016.

The advantages in deferring the work until 2016 were substantial. It would be possible to shut one carriageway at a time in order to carry out the work with minimal disruption, as traffic would be diverted to the new bridge thus negating the need to provide the extensive temporary works required if the joints were replaced in 2010 (the earliest date for a site start). This would result in a saving to the public purse of over £6 million.

Given the above, a review of the project was commenced to determine whether or not it would be possible to delay replacing the joints until 2016.

It was known that the joints had reached or were reaching the end of their service life and required to be replaced as there were concerns over their reliability and the consequences of their failure in service.

Therefore, the brief for the review has focussed on the safety of bridge users and potential disruption to traffic between 2010 and 2016 when the new bridge is scheduled to open.

It was also clear that if following the review, the Authority determined that replacement of the joints could not be delayed then action on solving the funding shortfall would recommence immediately.

The review was carried out by a team of senior officers from FETA and Atkins, the Authority's Consulting Engineer for the project. Flint and Neill, Consulting Engineers, acting as peer reviewer for the project, were asked to extend their brief to audit the review.

6.2 *Failure mode effect and analysis*

The review was carried out using failure mode and effect analysis (FMEA) techniques. This type of analysis is more commonly used in the aeronautical and process industries than in construction but it was felt that it was the best means of identifying the likelihood and consequences of failure of the various components that make up the joints. Once the failure mode and consequence of that failure of the individual components had been identified an analysis to determine what mitigation could be put in place was then carried out.

To assist in the above one sliding train was removed by FETA staff from the joint at the north tower on the northbound carriageway for inspection during a weekend closure on 16/17 January 2009 (Figure 9). This inspection provided further information to allow benchmarking of the condition of the joints to be established for the FMEA. A workshop with both consultants and FETA staff was carried out during that weekend to enable a decision to be reached as soon as possible.

The object of the review was to determine not only possible failure modes but whether or not it was possible to put measures in place in order to continue to maintain the high operational safety and service levels of the bridge until replacement could be carried out in 2016.

The FMEA confirmed that there are a number of key components of the joints that have a high risk priority number (RPN). The RPN is an index that enables risk to be prioritised. The probability of failure, and the severity or impact on the users if that failure occurred, combined with the likelihood of detection are all evaluated to determine the final RPN for each component.

Figure 9. Plate train being removed.

It should be noted that a high RPN does not automatically mean that there is a high probability that a component will fail. A high RPN could be an indication that a component has only an average probability of failure occurring but that the outcome of that failure is severe and the possibility of detection by current inspection frequencies and methods is low.

6.3 *Findings of the review*

The findings of the review are detailed below.

- The review team were in agreement with the earlier study that the joints had reached the end of their service life. However, it was recognised that the decision to replace the joints had been taken without the certainty of the timescale for the Forth Replacement Crossing (FRC).
- Given the certainty now provided by the Scottish Government on the timescale of the FRC, the review team concluded that it would be possible to delay the replacement of the joints until 2016 subject to the following:
  - Inspection and monitoring levels would have to be increased significantly.
  - The installation of the permanent access system which formed part of the replacement contract should be installed as soon as practicable to aid inspection both now and in the future and would also be used to replace the joint when the FRC is opened.
  - Key components such as pins and springs would have to be replaced and in some cases modified to improve performance.
  - Temporary failsafe devices would have to be installed as a precaution in case of failure of critical components (Figure 10).
  - The decision to defer the joint replacement should be reviewed annually or following any significant component failure or adverse inspection finding.

The inspection, monitoring and the work involved in installing the temporary failsafe devices would involve carriageway closures. The inspection work would include lifting out each one of the 48 expansion trains from the eight separate joints between now and 2016. However, the experience gained on Saturday 17 January 2009 has led to the conclusion that this work could be done during long Friday night to Saturday morning and/or long Saturday night to Sunday morning carriageway closures which would minimise disruption to traffic.

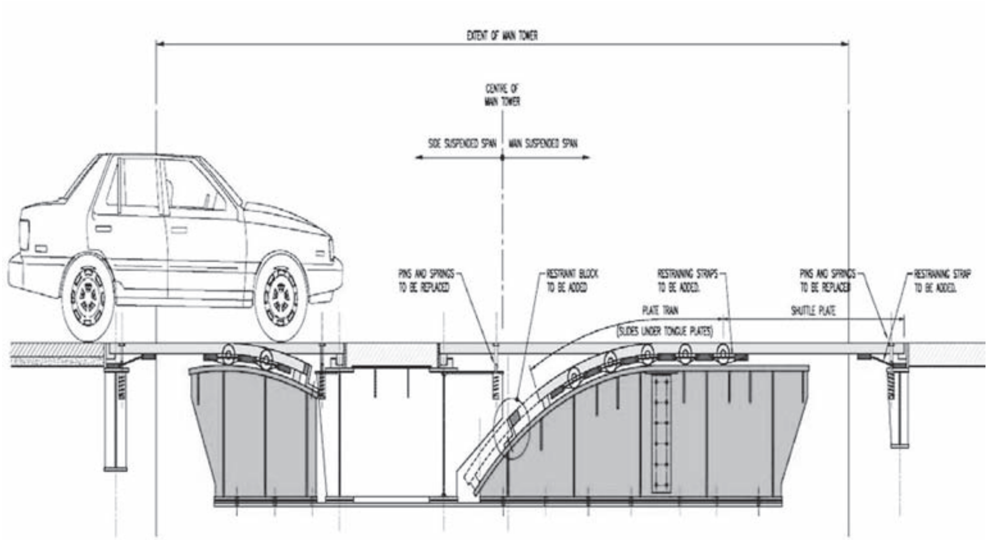


Figure 10. Section through joint showing proposed failsafe devices.

The cost of the installation of the temporary failsafe devices and the additional inspection and monitoring work has been estimated at between £150,000 to £250,000 as a total sum over the period 2009 to 2016. This cost can be compared with the £6 million plus cost that can be saved if the works are delayed until the opening of the Forth Replacement Crossing. The installation of the permanent access system ahead of the joint replacement is relatively cost neutral and is estimated to be between £500,000 and £600,000.

These measures will make it possible to maintain current operational safety levels for bridge users until 2016.

There are other residual risks that have been identified. These include risks to operational service levels and unforeseen risks that future movements and articulations that cannot be predicted cause failure. In addition, there is the risk that the Forth Replacement Crossing does not open in 2016.

– In extending the service life of the joints the risk that a component fails obviously increases. The mitigation measures will minimise the safety risk to users to an acceptable level. However, following any failure event there will be a risk that current operating service levels are reduced as remedial works would necessitate closures of a carriageway. It is difficult to quantify the duration of the carriageway closures that may be required but they would be expected to be measured in days rather than weeks. This risk will be significantly reduced by the recommended increase in inspection frequency and scope and by the early intervention of replacing pins and springs, a finding of the FMEA.
– The recent inspection of the joints confirmed that, increasingly, they are not behaving in a manner that was envisaged when they were designed. Given this, it is not possible to predict with any real accuracy future unknown movements and articulations of the joints as they continue to wear with age. Therefore, there is a risk that such unknown future movements and wear will result in the need to replace a major component such as a train prior to 2016. This would be very difficult to do without replacing other adjacent trains and would likely result in the need to replace the whole joint, which in turn would likely lead to the decision to replace all of the joints.
– It is expected that the recommended increase in the scope and frequency of the inspections will enable adequate, advance warning of the need to replace the joints before traffic restrictions would be required. However, funding would have to be arranged with the government and a

new contract to replace the joints put in place with the required temporary works to minimise disruption. It is estimated that it would take between 15 and 18 months to complete this work.
– The risk that there is a delay to the opening of the Forth Replacement Crossing in 2016 has to be considered. FETA does not have ownership of that risk. Therefore, a review of the decision to delay the replacement of the joints should also be carried out if there is any significant delay to the Forth Replacement Crossing project.

7 CONCLUSIONS

The main expansion joints on the Forth Road Bridge have been in service since the bridge opened in 1964 and in that time traffic volumes and the weight of vehicles crossing the bridge have increased significantly.

The joints are of the rolling plate type whereby a series of plates slide over fixed curved girders. There are eight joints in total and they allow movement of the 1006 metre long main span and 408 metre side spans and are thought to be the largest of their type in Europe.

The joints are now considered to have reached the end of their service life and a decision was taken to replace them. It was deemed unacceptable to close traffic lanes for the period required for the work to be carried out. Therefore, an innovative scheme to construct temporary ramps over the joint locations to allow traffic to pass over the replacement works was devised. However, a combination of a high tender price, due in the main to the high cost of the temporary works required to keep the traffic moving, and the announcement of a proposal to build a new bridge across the Forth to be open in 2016 led to a review of the project to determine if the service life of the joints could be extended until the new bridge opened. In extending the life of the joints until the new crossing was opened would lead to a saving of some £6 million.

The review was carried out using failure mode and effect analysis (FMEA) techniques. This type of analysis is more commonly used in the aeronautical and process industries than in construction but it was felt that it was the best means of identifying the likelihood and consequences of failure of the various components that make up the joints. Once the failure mode and consequence of that failure of the individual components had been identified an analysis to determine what mitigation could be put in place was then carried out.

The review concluded that the service life of the joints could be extended by increasing inspection and monitoring and by installing failsafe devices without reducing the current high operational safety levels on the bridge. However, it was recognised that there was a risk of a reduction in operational service levels which had to be balanced against the potential cost savings.

REFERENCES

Andrew, A.S., General Manager & Bridgemaster and Colford, B.R., Depute General Manager, Forth Estuary Transport Authority, *Forth Road Bridge—Maintenance Challenges, 5th International Cable Supported Bridge Operators Conference NY City 2006.*

Colford, B.R., Chief Engineer & Bridgemaster, *Forth Road Bridge—Maintenance and Remedial Works. Proceedings for the institution of Civil Engineers, Bridge Engineering (6) September 2008.*

The Forth Road Bridge Proceedings of the Institution of Civil Engineers, 1965 (Republished in 1967).

Chapter 33

Reinforcement design of the Filetto Bridge on the Santerno river near Bologna, Italy

G. Gasparini, S. Silvestri, T. Trombetti & C. Ceccoli
Department DISTART, University of Bologna, Italy

ABSTRACT: The aim of the project work here presented is to investigate the seismic behavior of the Filetto Bridge (an historical prestressed reinforced concrete bridge near Bologna in Italy) according to the rules of the new Italian Seismic Design Code. 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 solutions which provide the structure with the best performances are (i) the insertion of elastomeric isolators among the deck and the piers and (ii) the use of highly resistant carbon fiber strips in order to restore the loss of post-tension effect of the beams.

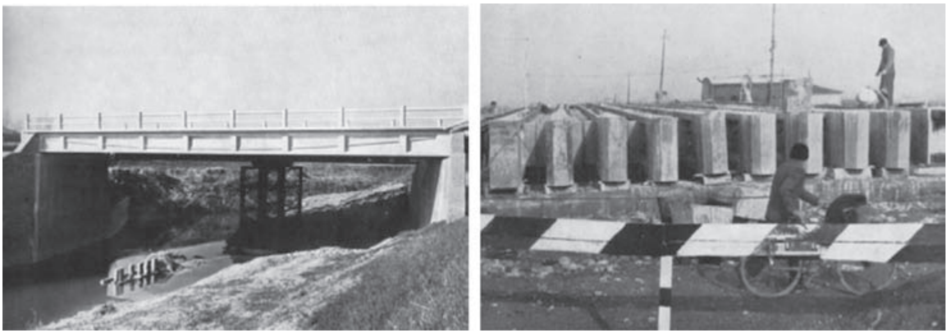
1 THE FILETTO BRIDGE

1.1 *The construction: Some historical information*

The Filetto Bridge was built about in 1950, to overpass 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. Figure 1 shows the geographical location (an aerial view) of the bridge.

This bridge has a peculiar characteristic: together with the Vallesella Bridge in Domegge near Belluno and the Samoggia Bridge in Lavino near Bologna (Figure 2), designed by Carlo Pradella (1905–1982), one of the most well-known engineer after the second war, it is the first post tensioned reinforced concrete bridge built in Italy (Siviero 2004).

Figure 1. Geographical location of the Filetto Bridge.



(a) (b)

Figure 2. The Samoggia Bridge near Bologna (a) Overall view; (b) Under construction.

The prestress and post tension techniques of reinforced concrete elements and structures were studied in the thirties by Eugene Freyssinet (1879–1962); in the historical chronicles retained in the Fontanelice town hall archive it's reported that Giuseppe Rinaldi, the engineer who designed the Filetto Bridge, who took out a patent for the headings of the prestressed cables (Figure 3), was helped in the use of the prestress reinforced concrete technique by an assistant of Freyssinet himself.

Figure 4 Shows some images of the bridge as it was just after the completion of its construction (about 1955).

1.2 *The structural design and state of the art*

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.

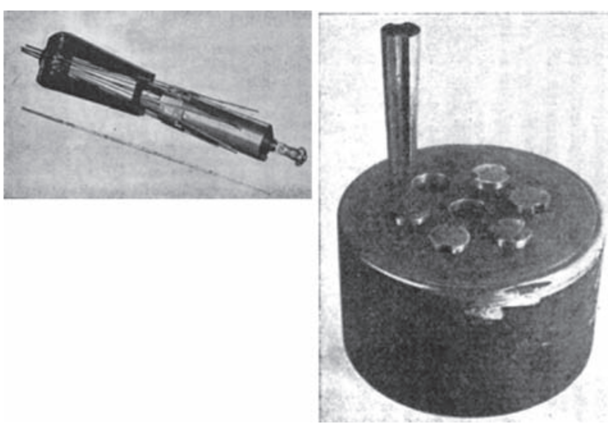


Figure 3. Rinaldi patent for headings (1950).

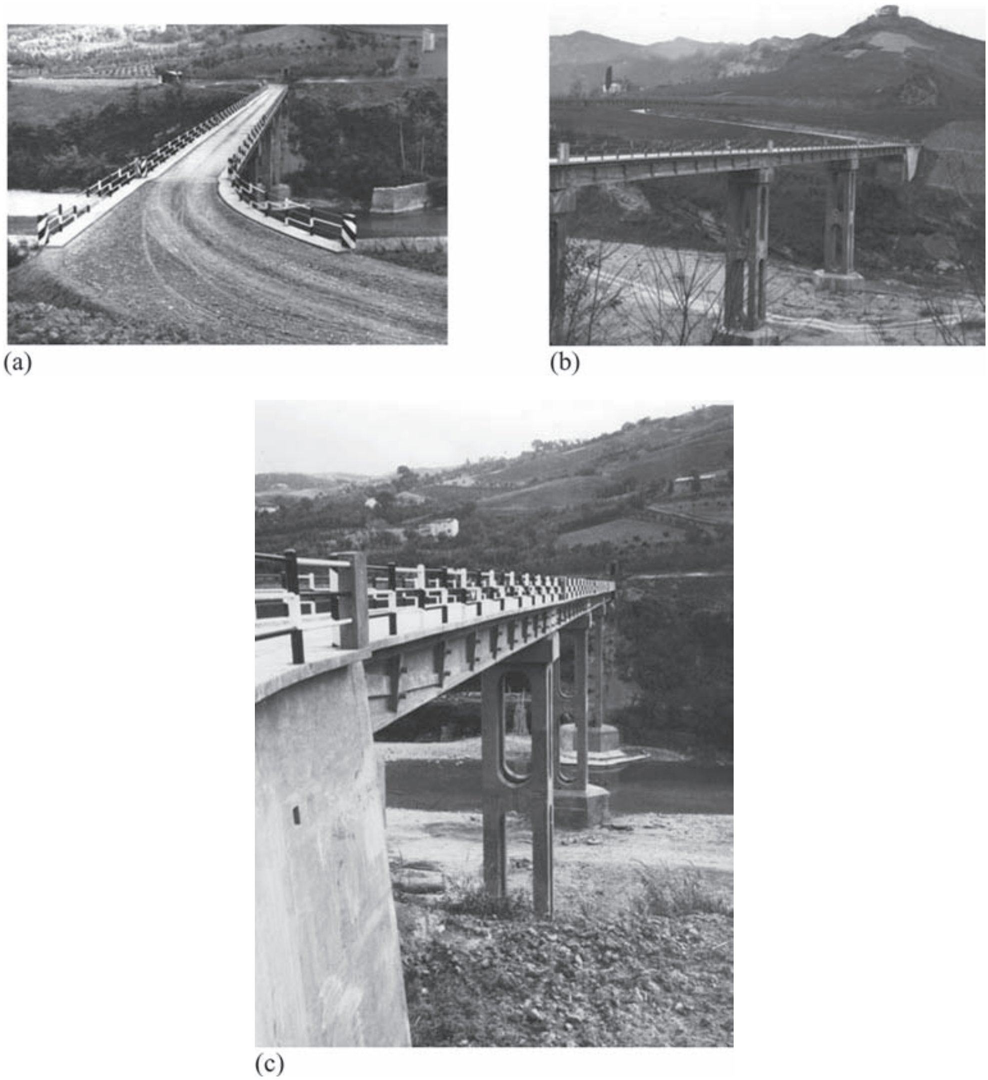


Figure 4. Historical photographs: (a) The gravel roadway leading to the bridge path; (b) Overall image; (c) The bridge piers. These images, contained in the historical archive of Fontanelice town, are kindly provided by the town hall technicians.

The bridge was, presumably, built considering the following steps during the construction:

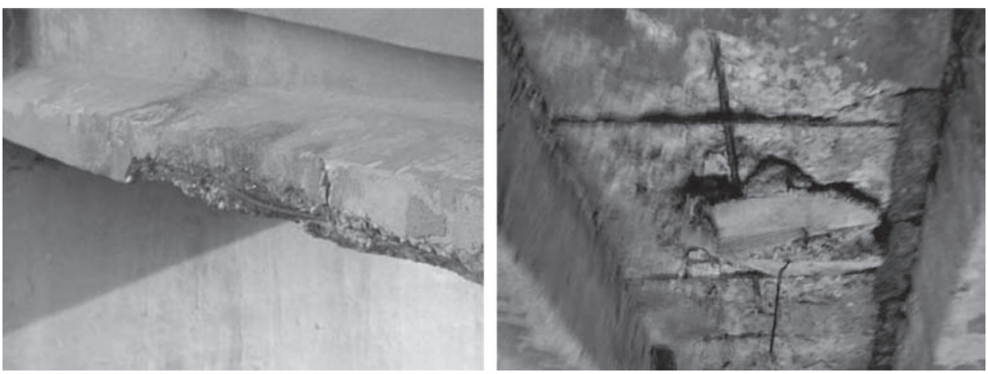
– realization of the longitudinal beams in the manufacturing plant, with prestressed straight steel cables located at their intrados;
– casting in situ of the longitudinal beams, realization of the transversal beams, each other at a distance of 3,0 m and consequent prestressing of these beams with steel cables located in the barycentric of each section;
– applying of the post-tension of the external cables (bounded by the transversal beams), presumably from only one head; the cables anchorage at the other head was, probably, realized in the manufacturing plant;
– realization of the floor reinforced concrete slab, asphalt and finishing works.

2 STATE OF THE ART

The state of the art of the bridge highlights some peculiarities: the reinforced concrete floor slab of the bridge deck got thinner (the reinforced asphalt thickness was higher than the reinforced concrete slab), with the concrete cover is completely expelled and several reinforcement steel bars are rusty and oxidized (Figure 5).

In the fifties, when the scientific and technological progress was at its beginning, several "poor" and recycled materials were used for different scopes: for example, to protect the headings of the transversal prestressed cables in the anchorage zones of the Filetto Bridge were used tomato cans (Figure 6) which managed very well. The external longitudinal cables, otherwise, realized with harmonic steel wires (ϕ5 mm each wire), weren't enough protected, so that there happens to be a serious loss of resistance for the whole beams. Most of the external cables lost their concrete protection (Figure 7a), in the way that they've been subjected to severe temperature conditions (rain, frost, humidity), becoming completely unusable (Figure 7b).

The sliding bearings located on the piers, maybe, are a little bit too sliding (Figure 8a): they were realized with steel rollers between two steel plates, so that the deck beams were simply leaning over the piers (Figure 8b). The steel rollers allowed the displacement of the beams over the piers, due to the variations in temperature, the shrinkage and so on: these bearing were simple and very effective, besides that during the past earthquakes they came out of the rails (Figure 8c).



(a) (b)

Figure 5. Reinforced concrete floor slab deterioration: (a) Expelled concrete cover, (b) Rusty steel bars.

(a) (b)

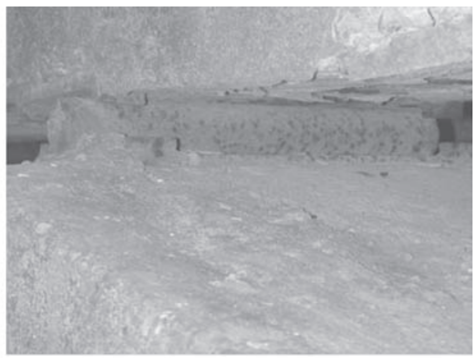
Figure 6. (a), (b) Use of tomato cans to protect the headings of the transversal prestressed cables.

(a)

(b)

Figure 7. (a), (b) External longitudinal prestressed cables exposed to severe temperature conditions effects.

(a)



(b)

(c)

Figure 8. (a) Sliding bearings; (b) Steel rollers between two steel plates; (c) Derailed rollers after earthquake.

An underwater inspection allowed to avoid the danger of an possible undermining of the bridge foundations due to the several cycles of the rising/lowering water level of the Santerno river, during about a hundred years (the foundation level appeared to be at about 6,0 m from the base of the piers plinths). Figure 9 shows the part of the foundation that comes out form the river.

(a) (b) (c)

Figure 9. (a), (b), (c) Piers foundation system.

3 THE MONITORING CAMPAIGN AND EXPERIMENTAL INSPECTIONS

At the end of June 2008, a wide monitoring campaign to control (i) the physical and mechanical characteristics both of the materials and of the structural elements of the bridge, and to evaluate the stress and strain trend in the elements themselves.

3.1 *Georadar inspection*

Using the georadar (an instrument whose functioning is based on reflection and propagation of electromagnetic pulses through materials with variables dielectric characteristics), both the elevation structures, the foundation system and the soil have been investigated, obtaining what follows:

- the reinforcement of the floor reinforced concrete slab is formed by 10 longitudinal steel bars/meter and 6 transversal steel bars/meter;
- 4 prestressed steel cables are located at the intrados of the deck beams;
- the shear reinforcement is constituted by 3 stirrups/meter, with concrete cover equal at about 2–3 cm;
- absence of piles under the piers and abutments foundation system.

3.2 *Investigations on materials*

In order to evaluate the concrete characteristics and the position of the steel prestressed and not prestressed bars, the following inspections have been performed: (i) ultrasonic measurements and sclerometric tests (combined), (ii) pull-out tests, measures of the carbonation level with the colorimetric method, electromagnetic investigations. Some limited destroying verifications allowed "to calibrate" the not destroying tests.

The results obtained are summarized as follows:

- the mean values of resistance (computed on 57 tests) for the beams are about equal to 361 daN/cm^2 (span 1), 365 daN/cm^2 (span 2), 321 daN/cm^2 (span 3), 315 daN/cm^2 (span 4); only 5 measurements give values between 224 daN/cm^2 and 251 daN/cm^2 and 6 measurements give values between 255 daN/cm^2 and 300 daN/cm^2. Excluded these 11 last measurements, regarding some zones with emphasized degradation, the total mean value is about 346 daN/cm^2;

- as regards the piers, it appears correct to assign a mean value of resistance of the concrete equal to 240 *daN/cm*^{2};
- pull-out tests have substantially confirmed the results obtained with combined method (ultrasonic measurements and sclerometric tests);
- the colorimetric method test has allowed to prove that the carbonation depth turns out to be among 1 and 5 cm;

As regards the reinforcement steel bars, the electromagnetic investigations, validated by appropriate small demolitions, have allowed the following evaluations:

- the steel prestressed straight cables located in the lower wing of every beam turns out to be formed by 4 cables with an external sheath of diameter equal at about 32 mm, (presumably obtained each with 12 or 16 wires ϕ5 mm);
- external to the beams (on both sides of the web) there are 3 post-tensioned steel cables, each with outside diameter equal at 27 mm, formed by established by bundles of 9 wires ϕ5 mm; in the rectilinear section these cables are located at 25 cm from the beam intrados;
- the 2 prestressed cables of the transversal beams, also turn out to be formed by bundles of 9 wires ϕ5 mm and are located substantially at the barycentre of the beam section.

4 THE REINFORCEMENT PROJECT

4.1 *Reference Seismic Code and preliminary analysis results*

The linear dynamic analysis of the response behaviour of the whole structural system (bridge deck-isolators-piers) has been performed on the basis of the prescriptions provided by the Italian Seismic Code (Ordinanza del Consiglio dei Ministri n. 3274) with special reference to what reported for the planning of bridges endowed with seismic isolation, that is the criteria and rules for the reinforcement and consolidation of existing bridges by inserting an isolation system between the deck and the piers/abutments, in order to improve the dynamic response of the whole bridge when subjected to seismic actions. Preliminary analysis showed, in fact, that the existing structure was 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 existing supporting devices couldn't support the drifts between the pier and the deck.

4.2 *Structural solutions*

In order to restore the capability of the Filetto Bridge of sustain the horizontal forces due to seismic actions, a specific reinforcement design has been provided by Studio Ceccoli e Associati, one of the most well-known structural engineering office in Bologna; the fundamental works which provide the structure with the best performances are:

- complete demolition of the superior reinforced concrete 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);
- realization of structural joints on the abutments to avoid the effects of thermal expansion on the slab;
- reinforcement of the reinforced concrete beams of the decks with the insertion of highly resistant carbon fiber strips, in order to restore the loss of post-tension effect.

The uplifting and replacement of the existing bearings between the deck and the piers appears the fundamental improvement element under the seismic point of view, and characterizes the whole design. Such technique allows to obtain an adequate safety level of the structure from the effects of the seismic events, by reducing the actions. By inserting seismic isolators among the deck and the piers, in fact, it's possible to considerably increase the period of vibration of the structure, avoiding the zone of the response spectrum characterised by high values of acceleration. The isolators used for the reinforcement design of the Filetto Bridge are elastomeric bearings, characterized by low horizontal stiffness (about equal to 500 daN/cm, that is 0,50 kN/mm) able to guarantee the uncoupling of the horizontal motion of the structure from the ground motion, and by high vertical stiffness (about equal to 500000 daN/cm, that is 500 kN/mm) to allow to adequately bear the vertical loads.

4.3 *Analysis conducted*

The following parameters has been considered for the identification of the design response spectrum used in the dynamic analysis of the structure:

- bridge of ordinary importance (importance factor: $\gamma_I = 1$)
- foundation soil category: A (rock soil)
- seismic zone: 2 (medium level of seismicity)
- equivalent viscous damping coefficient (considering the whole structural system bridge deck-isolators-piers): $\xi = 15\%$
- damping coefficient: $\eta = 0{,}707$

The reduction of the bridge seismic response has been obtained by increasing its fundamental period of vibration (thanks to the insertion of the isolation system), in order to have smaller response accelerations.

The Italian Seismic Code specifies that, in alternative to the use of the standard and codified form of the elastic response spectrum associated with the specific values of Peak Ground Acceleration (PGA = α_g) provided for each different seismic zone, it can be used specific response spectrum for the site considered, characterized by the different probability of occurrence for each limit state. In conclusion the following parameters and definitions have been considered (Table 1):

- Nominal life (ordinary importance bridges): $V_N > 50$ years
- Use Class: II (ordinary crowding)
- Use Coefficient: $C_u = 1{,}0$
- Reference Life: $V_R = V_N\, C_u = 50$ years

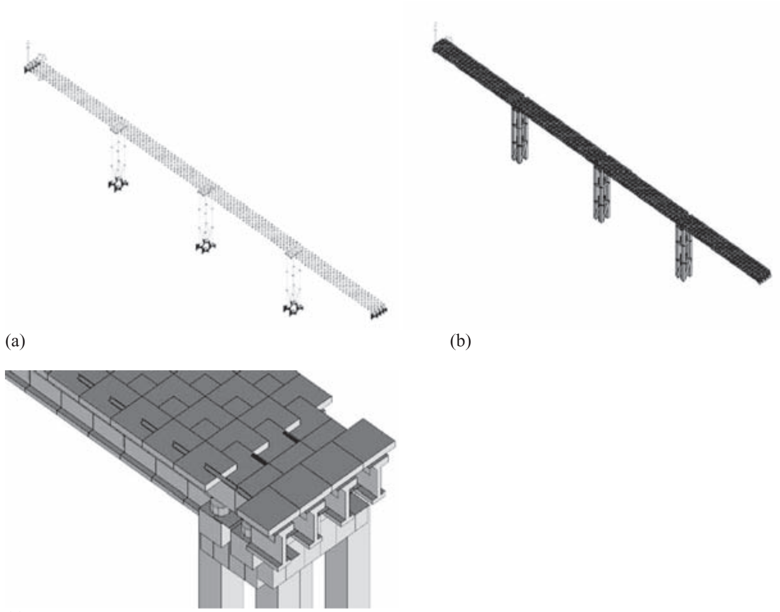
It is necessary furthermore to specify the characteristics of the isolators (obtained from a linear elastic analysis):

- horizontal stiffness: 500 daN/cm = 0,50 kN/mm
- vertical stiffness 500.000 daN/cm = 500 kN/mm

A linear dynamic analysis has been led on a computational FEM model which represents the bridge deck, the isolators and the piers (Figure 10).

Table 1. Parameters used for the linear dynamic analysis.

Limit state	Return period (T_R)	PGA (a_g)	Maximum value of the amplification factor for the response spectrum (F_0)	Period of vibration corresponding to the constant value of the response spectrum (T_c*)	Probability of occurrence during the reference life $(P_V)_r$
SLO (Operational limit state)	30 years	0,692 g	2,41	0,26 sec	81%
SLD (Damage limit state)	50 years	0,880 g	2,40	0,27 sec	63%
SLV (Life safe limit state)	475 years	2,068 g	2,47	0,30 sec	10%
SLC (Collapse limit state)	975 years	2,585 g	2,52	0,31 sec	5%



(a) (b)

(c)

Figure 10. (a), (b) Computational FEM model; (c) Particular of the connection between bridge deck (beams), isolators and piers.

4.4 *Results obtained*

The fundamental period of vibration with the isolation system turns out to be equal to 1,48 sec, approximately the double of the period of vibration of the structure without the isolators (about 0,75 sec).

Figure 11 shows the six modes of vibration of the structure endowed with the isolation system (deformed and undeformed shape); Table 2 summarizes the fundamental values of the vibration periods.

The main results obtained from the analysis performed are the following:

– the accelerations are considerably reduced;
– the horizontal action on each isolator laying at the top of the piers is equal to: $H_{\max-iso-pier} = 2500\ daN = 25\ kN$
– the maximum lateral displacement of each isolator laying at the top of the piers is equal to: $\delta_{max-iso-pier} = 50\ mm$
– the horizontal action on each isolator laying at the top of the abutments is equal to: $H_{\max-iso-abut} = 3000\ daN = 30\ kN$
– the maximum lateral displacement of each isolator laying at the top of the piers is equal to: $\delta_{\max-iso-abut} = 60\ mm$
– the maximum horizontal action at the top level of the piers is equal to: $H_{\max-pier} = 20000\ daN = 200\ kN$

The maximum forces on each of the 4 columns of a single pier, considering the verification for an Ultimate Limit State (ULS), are equal to:

$N_{\max} = 126000\ daN = 1260\ kN$ (compression)
$N_{\max} = -5000\ daN = -50\ kN$ (traction)
$M_{\max} = 18000\ daNm = 180\ kNm$ (bending moment)

These values allow the verification to be fully satisfied.

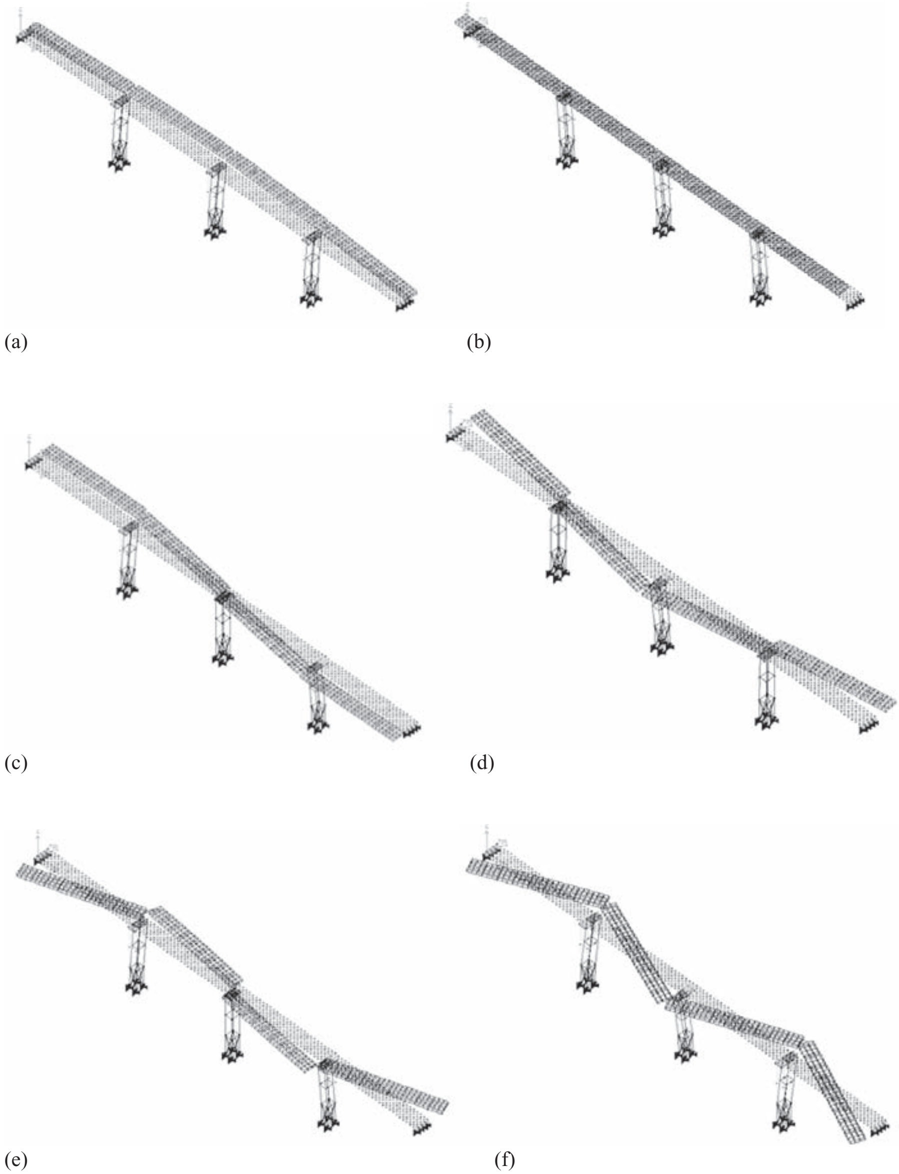


Figure 11. Modes of vibration, undeformed shape (light grey) deformed shape (dark grey); (a) 1 mode, (b) 2 mode, (c) 3 mode, (d) 4 mode, (e) 5 mode, (f) 6 mode.

5 REALIZATION OF THE REINFORCEMENT DESIGN WORK

Step 1: realization of the ford (Figure 12a), even if the river doesn't always agree (Figure 12b).
Step 2: putting up the scaffolding (Figure 13). The scaffolding comes to be hung on the bridge itself. This step requires many days, but the advantage that derives from this construction is remarkable (possibility to work under the bridge deck, where are provided the most difficult operational phases).

Table 2. Modes of vibration, periods and total excited mass.

	Frequency Hz	Period sec	Spectral Acc. g	Excited mass (Xg) daN	%	Excited mass (Yg) daN	%
1	0.675	1.481	0.099	0.18	1.73e–05	7.055e+05	68.2
2	0.683	1.465	0.100	7.437e+05	71.9	0.17	1.63e–05
3	0.740	1.352	0.108	4.41e–06	0.0	0.0	0.0
4	0.871	1.148	0.127	1.18e–05	0.0	2.747e+04	2.7
5	1.097	0.911	0.160	0.0	0.0	0.0	0.0
6	1.429	0.700	0.209	0.0	0.0	104.63	1.01e–02
7	1.833	0.546	0.268	0.0	0.0	0.0	0.0
8	2.215	0.451	0.324	0.0	0.0	698.70	6.76e–02
9	2.499	0.400	0.365	5.74e–05	0.0	2.43e–05	0.0
10	2.769	0.361	0.366	8.90e–05	0.0	0.0	0.0
11	2.773	0.361	0.366	5.13	4.96e–04	1.53e–06	0.0
12	2.805	0.357	0.366	1.370e+05	13.2	1.86e–03	0.0
13	2.975	0.336	0.366	2.40e–03	0.0	1.382e+05	13.4
14	3.015	0.332	0.366	9.82e–06	0.0	9.97e–05	0.0
15	3.045	0.328	0.366	2.88e–05	0.0	1597.49	0.2
Results				8.807e+05		8.736e+05	
Percentage				85.18		84.50	

Figure 12. The river ford.

(a) (b)

Figure 13. (a), (b) The scaffolding.

Figure 14. Abutment reinforcing.

Step 3: reinforcement and consolidation of the abutments (Figure 14).
Step 4: the uplifting of the bridge deck, using hydraulic jacks (Figure 15).
Step 5: the insertion of the seismic rubber bearing isolators (Figure 16).
Step 6: the positioning of the predalles (Figure 17a) and the realization of the "kinematic lines" (Figure 17b): the kinematic lines connect the deck spans each other, but not to the abutments; they bear both compression and traction stresses.
Step 7: the casting of the reinforced concrete floor slab (Figure 18).
Step 8: reinforcement of the reinforced concrete beams of the decks with the insertion of highly resistant carbon fiber strips (Figure 19a), in order to restore the loss of prestressing effects. The fiber have been painted the same color of the bridge (Figure 19b).
Step 9: realization of the seismic joints (Figure 20) of about 15 cm between the deck and the abutment.

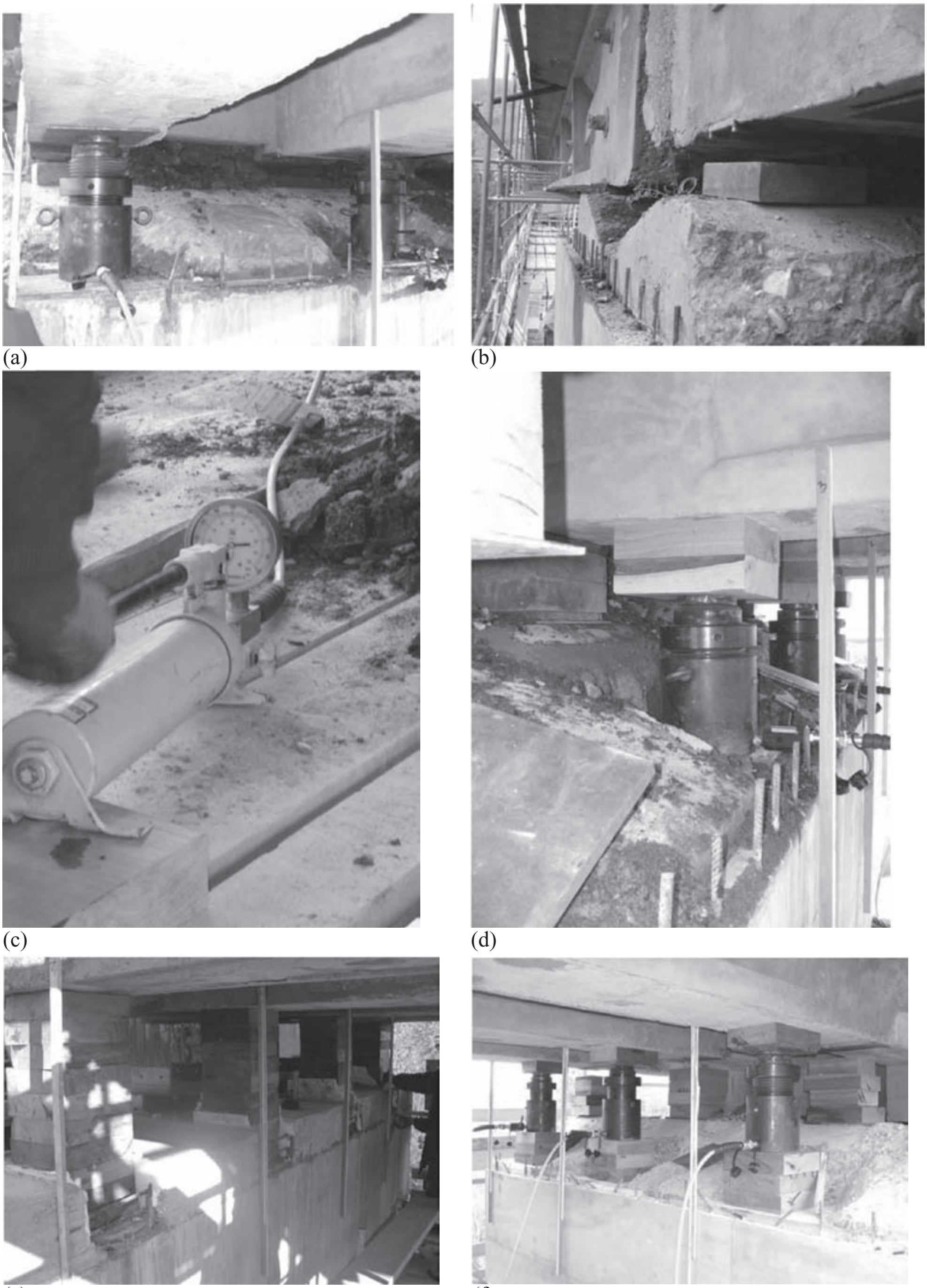
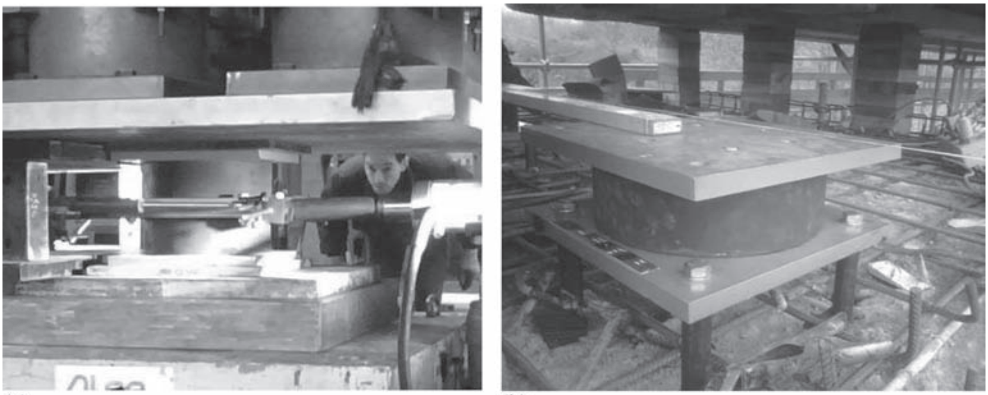
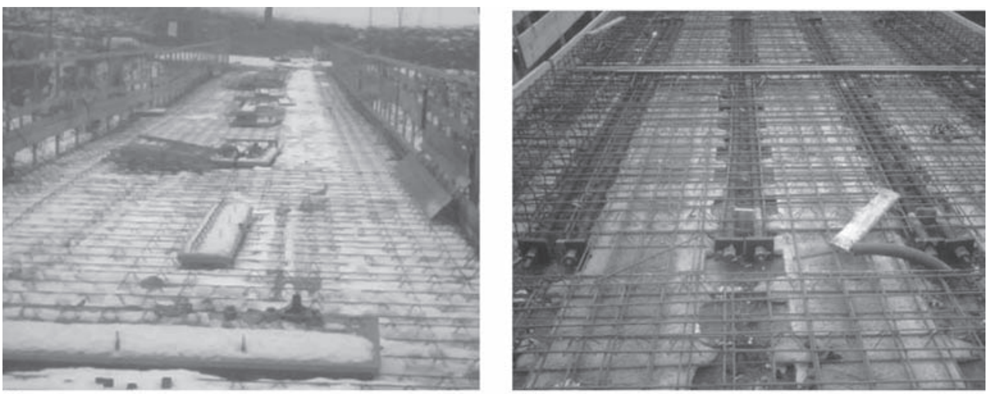


Figure 15. (a), (b), (c), (d), (e), (f) The bridge deck uplifting with hydraulic jacks.



(a) (b)

Figure 16. (a), (b) Elastomeric bearings.

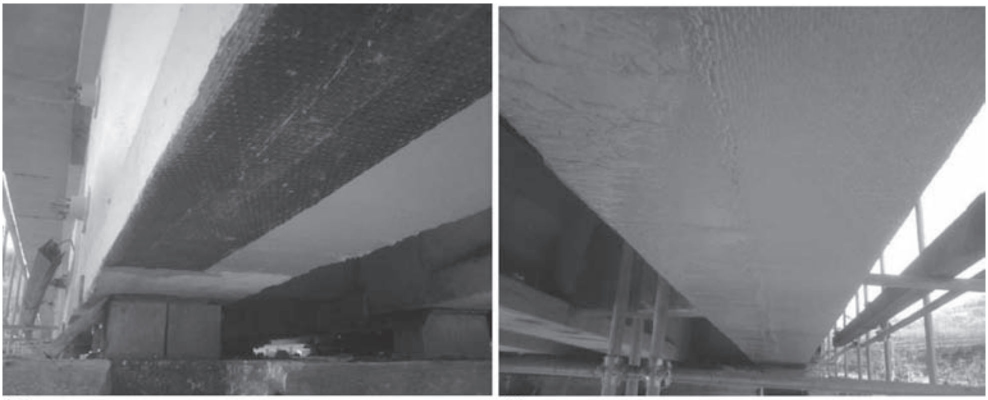


(a) (b)

Figure 17. (a) Positioning of predalles; (b) Kinematic line.

(a) (b)

Figure 18. (a), (b) Casting of the reinforced concrete slab.



(a) (b)

Figure 19. (a), Highly resistant carbon fiber strips; (b) Painted fiber.

(a) (b)

Figure 20. (a), (b) Seismic joint.

6 CONCLUSIONS

The analytical static and dynamic investigations carried out on the Filetto Bridge showed that the seismic behavior of the original existing structure didn't fulfill the prescriptions and the performances requested by the new Italian Seismic Design Code, mainly because of (i) the benefit loss of the post-stress due to the traction stresses in the deck, and of (ii) the deterioration of the supporting devices between the pile and the deck.

The reinforcement project takes into account some structural solutions that improve the seismic behavior of the bridge: the main interventions provided (i) the demolition of the superior r.c. slab of the whole deck and the realization of a new slab with lightweight aggregated concrete, (ii) the substitution of the actual restraints with elastomeric/rubber bearings, (iii) the realization of structural joints on the abutments to avoid the effects of thermal expansion on the slab, (iv) the 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.

ACKNOWLEDGMENT

The authors gratefully thank the structural engineers, Gilberto Dallavalle and Franco Baroni, Studio Ceccoli e Associati, Bologna, Italy, who planned and directed the reinforcement design and its construction at the Filetto Bridge.

REFERENCE

Siviero E. Zampini I. 2004. *Carlo Pradella e i primi ponti italiani in c.a.p.*, Proceedings of the 15th CTE Congress, Bari 4-5-6- November 2004.

Chapter 34

Retrofit and replacement of Dumbarton railroad bridges, California, USA

K.S. Go
HNTB Corporation, San Jose, California, USA

S. Treyger
HNTB Corporation, Bellevue, Washington, USA

M. Jones
HNTB Corporation, Santa Ana, California, USA

S.J. Hill, B. Susanto & W. Yang
San Mateo County Transit District, San Carlos, California, USA

ABSTRACT: The Dumbarton Rail Corridor is located in the southern region of the San Francisco Bay Area, connecting east and west bay. The Corridor is approximately 20.5 miles in length and includes a 4.5 mile segment that crosses the southern portion of the San Francisco Bay. The crossing of the Bay includes two bridges: the Dumbarton Bridge and Newark Slough Bridge, approximately 7550 and 422 feet long, respectively. Portions of these Bridges have timber and steel truss bridge sections that are approximately 100 years old, and each has a 100-year old swing span. The existing movable bridges have not been operational since the mid-1980's and portions of the existing timber bridges were damaged in a fire in the late 1990's. The Dumbarton Rail Corridor (DRC) Project proposes to provide commuter rail service by improving the existing rail infrastructure in the DRC Corridor. In order to reopen the Dumbarton and Newark Slough Bridges to accommodate the planned Dumbarton commuter rail service, several activities were performed in order to help determine if the bridges should be retrofitted or replaced. They include: Inspections (Underwater, Above Water), Concrete Coring of 100-year old large diameter caissons, Condition assessment of existing steel and concrete structures, Service Load Rating Analyses, Movable Bridge evaluation, Seismic Analyses, Constructability Review and Cost Estimates.

1 INTRODUCTION

The Dumbarton Rail Corridor is located in the southern region of the San Francisco Bay Area, connecting east and west bay. The Corridor is approximately 20.5 miles in length and includes a 4.5 mile segment that crosses the southern portion of the San Francisco Bay. The crossing of the Bay includes two bridges: the Dumbarton Bridge and Newark Slough Bridge, approximately 7550 and 422 feet long, respectively. Portions of these Bridges have timber and steel truss bridge sections that are approximately 100 years old, and each has a 100-year old swing span. The existing movable bridges have not been operational since the mid-1980's and portions of the existing timber bridges were damaged in a fire in the late 1990's.

The Dumbarton Rail Corridor (DRC) Project proposes to provide commuter rail service by improving the existing rail infrastructure in the DRC Corridor. In order to reopen the Dumbarton and Newark Slough Bridges to accommodate the planned Dumbarton commuter rail service, several activities were performed in order to help determine if the bridges should be retrofitted or replaced. They include:

- Project Specific Structure Design Criteria
- Inspections (Underwater, Above Water)
- Concrete Coring of 100-year old large diameter caissons
- Service Load Rating Analyses
- Condition assessment of existing steel and concrete structures
- Movable Bridge evaluation
- Geotechnical evaluation
- Seismic Analyses
- Constructability Review
- Cost Estimates

2 BACKGROUND

The existing Dumbarton and Newark swing steel truss bridges and piers were constructed in 1910. The original construction included: ballasted deck timber trestle at the west approach; steel deck girders and steel through truss with a swing span at the channel crossing; and ballasted deck timber trestle at the east approach. While the entire structure has a single railroad track, the steel spans were built for two parallel tracks. Sections of the approach spans have been modified and rebuilt in the 1960s and 1970s with precast concrete construction. In 1980s portions of the west timber trestle were destroyed in a fire.

The engineering team evaluated the original concrete piers and steel trusses to determine the feasibility of retrofit/rehabilitation of the structures for the proposed Dumbarton Rail service. See Figures 1, 2 and 3 for location and configuration of the existing bridges.

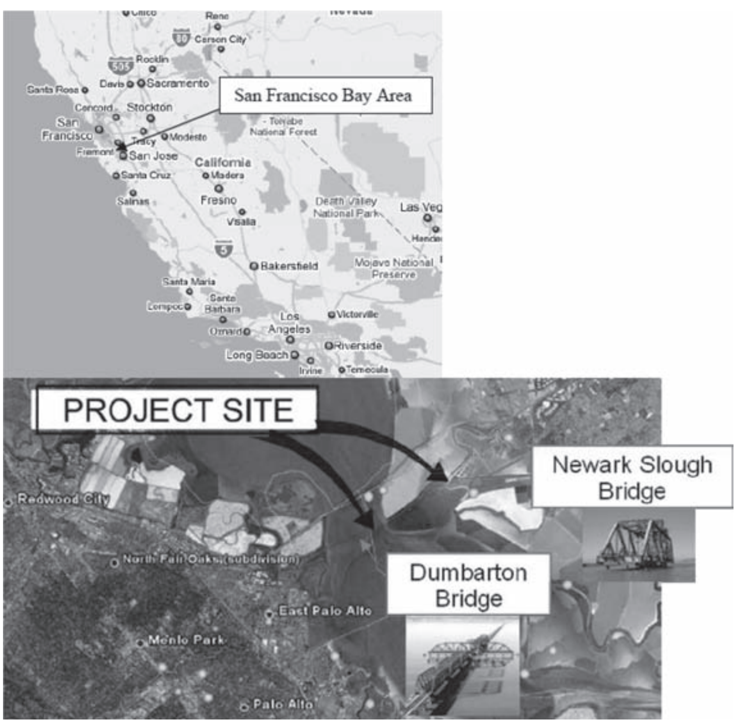


Figure 1. Location map.

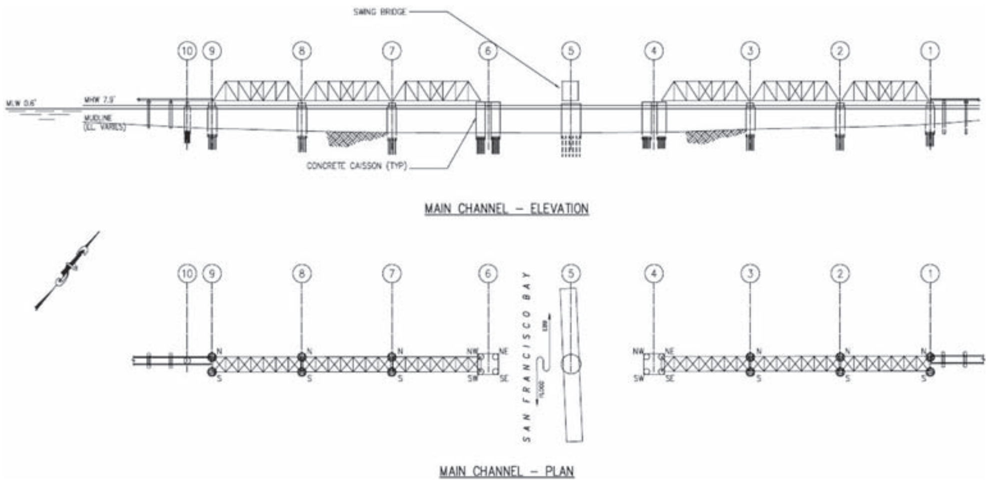


Figure 2. Dumbarton main span—swing span and steel trusses (Section 4 to 9).

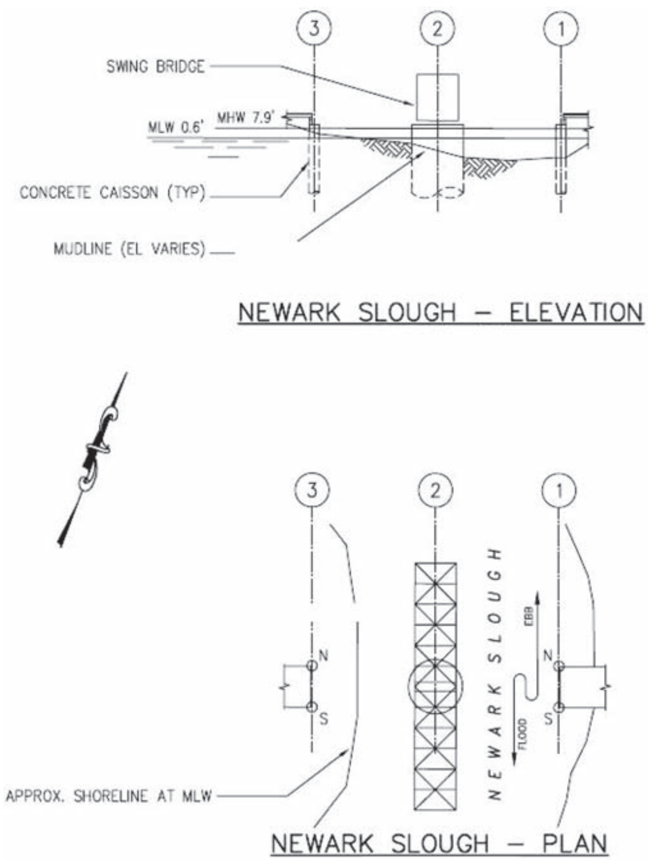


Figure 3. Newark Slough swing span.

2.1 *Constraints*

Construction in the San Francisco Bay posses several constraints. These constraints include water quality, biological, protection of endangered species, mammals and considerations of marine, bird and other wildlife breeding and nesting seasons. In addition, working on existing structures that are approximately 100 years old also imposes certain constraints.

2.2 *Work plan*

The work plan (Figure 4) below presents the relationships between the different activities and how each activity relates to another in order to meet the project objective of reactivating the Dumbarton Rail Corridor on the existing bridge.

3 INSPECTIONS

From November 2007 to January 2008, HNTB engineers and inspectors and Halcrow underwater divers-inspectors conducted inspections of the existing bridges and its supporting piers. CTL Group concrete specialists obtained concrete coring samples of the existing concrete caissons.

3.1 *Caisson inspections*

Over several days in November and December 2007, a three-person experienced engineer-diver team conducted detailed above-water and underwater inspections of 22 concrete caissons supporting the spans at the Dumbarton Rail Bridge and 5 concrete caissons of the bridge at the Newark Slough. These existing caissons were built as part of the original steel trusses construction in 1910. All caissons exhibit a zone of corrosion in the steel casing, typically ranging from 1-ft above the Mean Low Water (MLW) elevation to 10-ft below the MLW elevation. Approximately 60% of the steel in that zone has moderate to severe corrosion with areas of exposed concrete. The condition of the exposed concrete varies widely, from minor surface deterioration, to large voids with soft concrete and exposed timber piles (see Figures 5, 6 and 7).

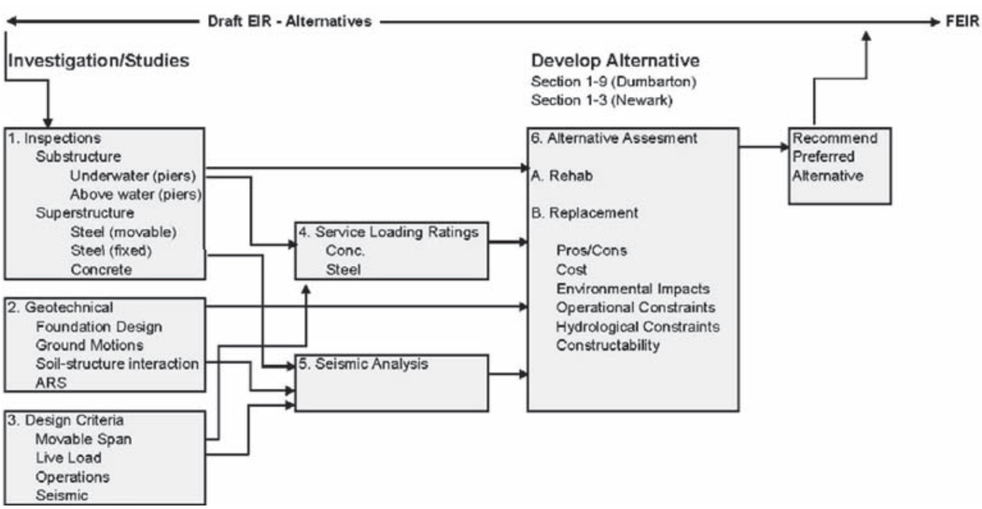


Figure 4. Work plan for retrofit and replacement of Dumbarton railroad bridge.

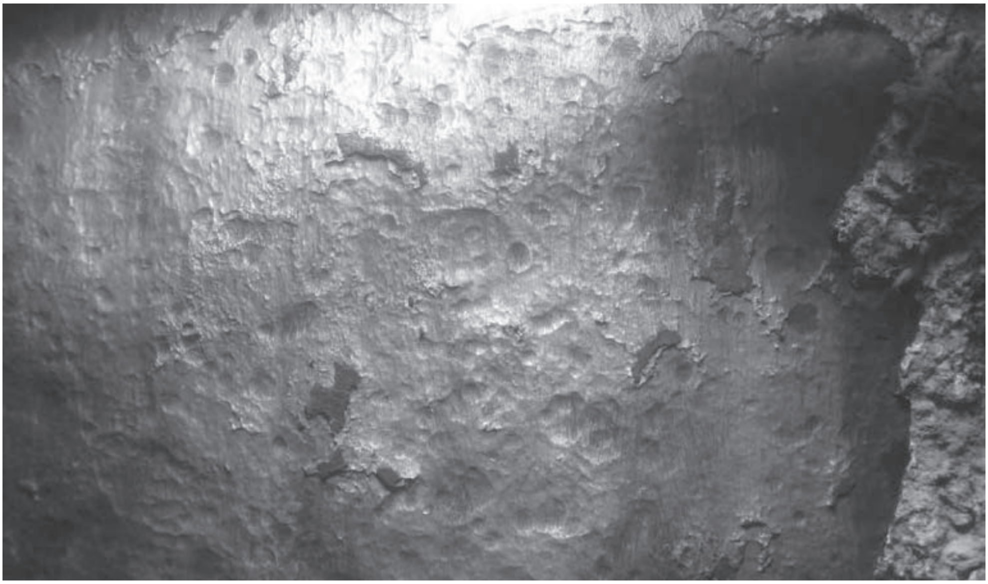


Figure 5. Dumbarton pier 6, Northwest caisson, SE Quadrant, 9.5-ft below MLW. Typical steel in 1-ft above to 10-ft below MLW, typical condition of steel shell.

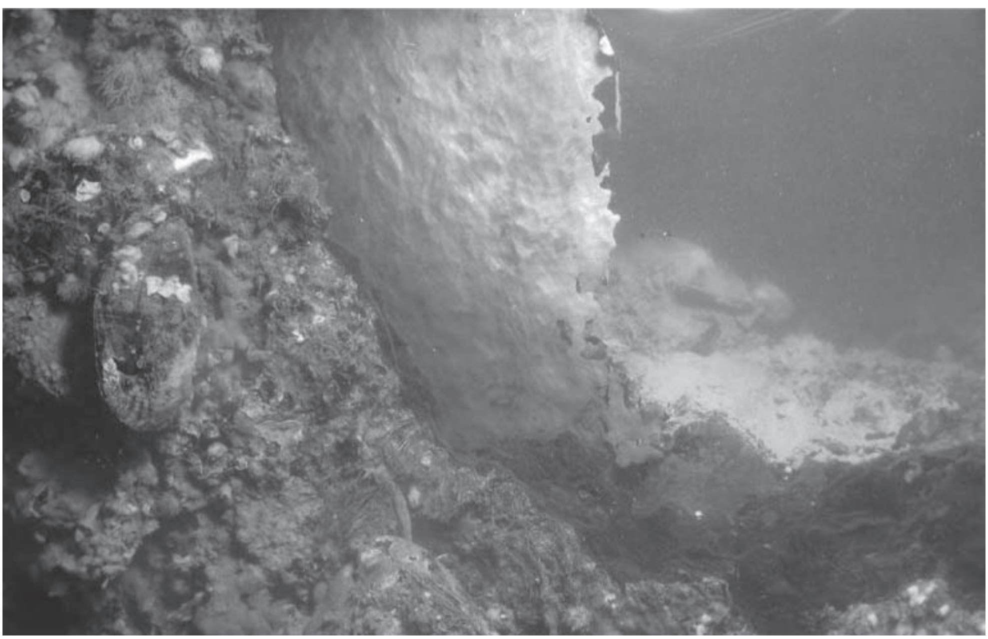


Figure 6. Dumbarton pier 6, Northwest caisson, SW Quadrant, 6.5-ft below MLW with large void (28'W × 2.3'H × 5'D) in caisson.

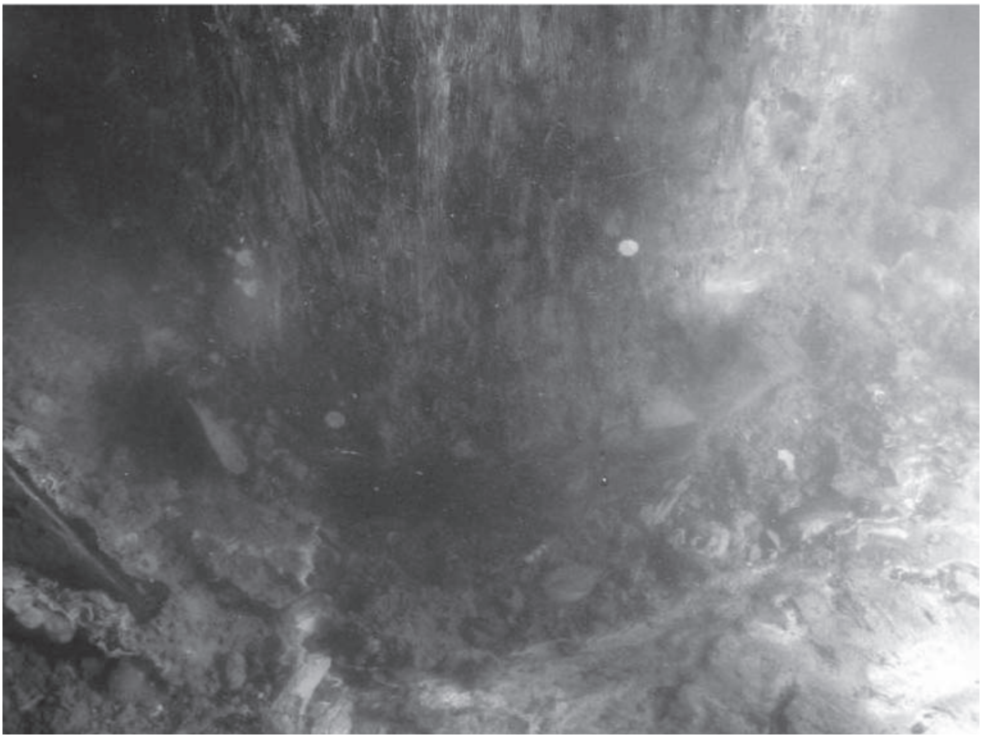


Figure 7. Dumbarton pier 6, Northwest caisson, SE Quadrant, large spall with exposed timber pile.

Overall structural integrity of the original caissons is affected by large areas of spalling with deep voids at the caissons, particularly the voids with exposed timber piles. Since a majority of the steel shells at the caissons are still intact, it is impossible to determine the overall extent of the deterioration of the concrete.

3.2 *Steel trusses and swing bridges inspections*

The Dumbarton and Newark Swing Bridges are open deck swing spans which opened to traffic in 1910. The span lengths for Dumbarton and Newark swing spans are 310-ft and 182-ft, respectively. Six 180-ft span steel trusses approach to the Dumbarton swing span from east and west. The structures have not seen any rail traffic since the mid 1980s.

Light to moderate blanket rust is present on almost all truss members as the protective paint system has failed. Pack rust is found under most connection plates, top surfaces of top and bottom chords and end posts. The floor system (floorbeams and stringers) and cross girders have extensive corrosion and severe section loss in several locations. The severity of the section loss and pack rust found in the floor system and connections warrants their replacement. All of the secondary members (laterals) and their connection gusset plates have substantial section loss and severe corrosion which warrants replacement of these members in their entirety. (See Figures 8, 9, 10 and 11 for bridge conditions).

3.3 *Concrete coring*

In January 2008, structural concrete specialists took concrete core samples from the concrete caissons that support the original steel trusses. A total of nine deep concrete core samples were taken with depths ranging from 16 to 72 feet from the top of caissons.

Figure 8. Dumbarton bridge—typical failed paint on truss member.

Figure 9. Dumbarton bridge—typical corrosion and pack rust of steel truss member.

Figure 10. Dumbarton swing span—view of floor beam.

Figure 11. Newark swing span—view of floor beam and pier top.

Concrete conditions were remarkably consistent for all cores drilled. Concrete in upper portions of the caissons (upper 12 to 14 feet), extending to approximately the tidal fluctuation zone consisted of good quality concrete (See Figure 12).

Concrete samples extracted from the tidal fluctuation zone (region at water line and below) were less sound and of lower quality. Cement paste was very soft and often washed away from the core surface, leaving exposed aggregate (See Figure 13). This condition is consistent with a concrete mix of very high water to cement ratio. In several locations, sample lengths consisting of

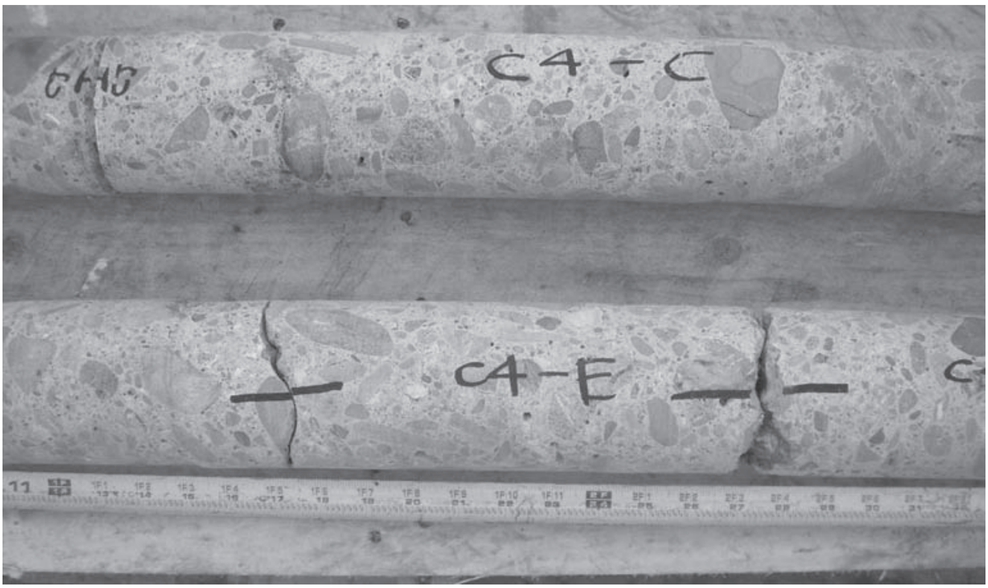


Figure 12. Typical concrete core samples, upper portions of piers.

Figure 13. Typical concrete sample from tidal fluctuation zone.

only cement paste were extracted (See Figure 14). This condition is consistent with segregation of the mix during original concrete/cement placement.

Below the tidal fluctuation zone, concrete quality was poor, typically disintegrating to rubble during coring. At the main span center pier core hole, concrete quality was poor from the tidal fluctuation zone and below (See Figure 15 and 16).

Figure 14. Concrete sample showing high level of cement paste.

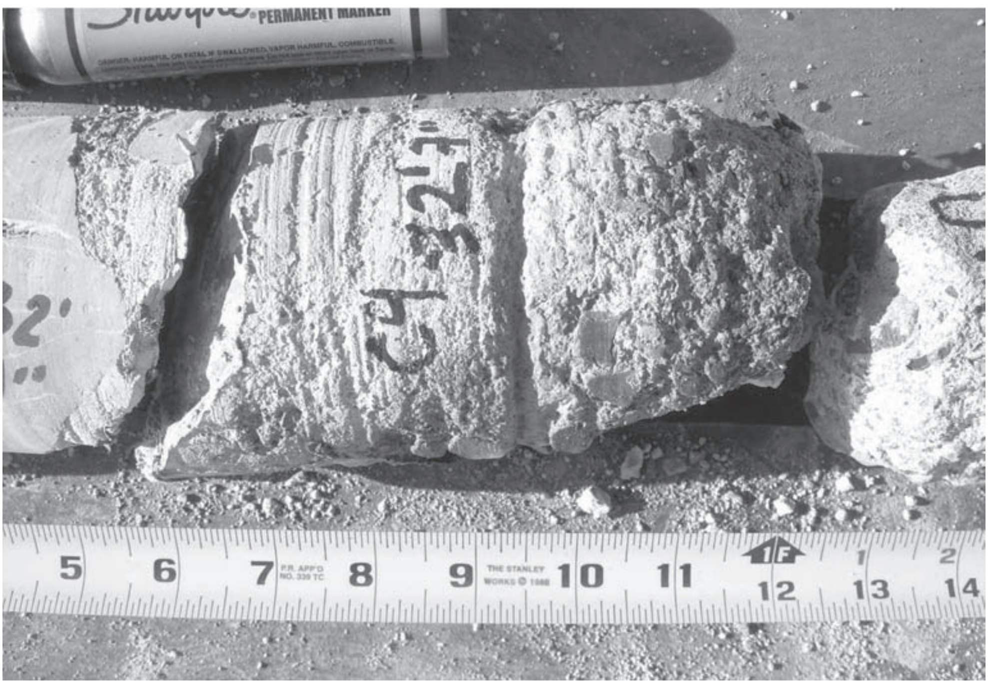


Figure 15. Poor quality concrete, typical of samples extracted below the tidal fluctuation zone.

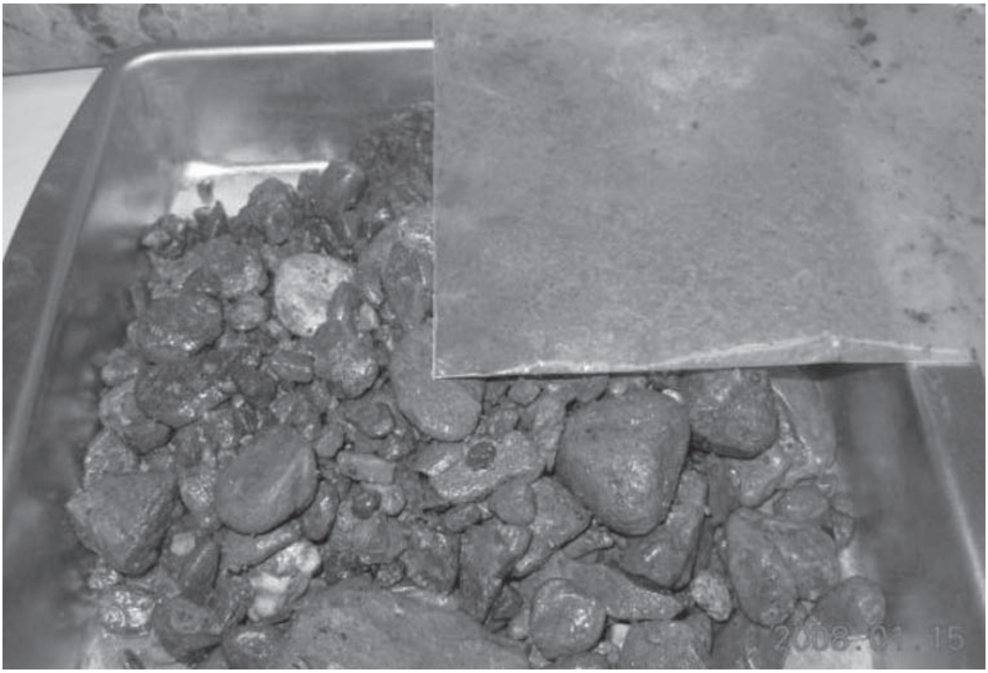


Figure 16. Rubble—Aggregate is very clean, with very little cement paste on surface. Typical of samples extracted below the tidal fluctuation zone.

Based on preliminary observations, it is believed that the poor quality of the concrete in the lower portions of the caissons is due to washout of the cement paste during original construction. Historical ASCE papers (Schneider 1913) describing the bridge construction indicate that concrete was simply dumped into the caissons in still water conditions, as was customary at the time.

4 LOAD RATING ANALYSES

A two dimensional computer model was created to complete a load rating evaluation for the six Dumbarton steel through truss spans and swing span. The analysis and modeling was performed using a proprietary program called T187, a structural analysis computer program developed by HNTB. The analysis focused on the vertical load resisting members within the truss and floor system. The result of the examination contains load ratings for the top and bottom chords, hangers, diagonals, floorbeams and stringers.

Using information from existing drawings and field measurements, member section properties and dead loads were calculated with Excel spreadsheets. Member section properties were adjusted to account for effects of corrosion in the bridge members. These modified section properties were incorporated in the computer model for the dead and live load analysis. Results of the computer analysis were used to carry out the load rating evaluation, which was done in conformance with the 2007 American Railway Engineering and Maintenance-of-way Association (AREMA) Manual.

The new rail corridor is expected to be a single track designed for Cooper E-80 loading. Results from the load rating evaluation demonstrated that all vertical load resisting truss and floor system elements can safely support a Cooper E-80 loading, with the exception of the stringers and end floorbeams. Their reduced load rating is primarily due to section loss of the members from corrosion. However, the stringers and all floorbeams exceed their fatigue category stress range.

For this reason, it is recommended that the stringers and all floorbeams be replaced if the existing steel through trusses were reused for the new rail corridor. The main truss members can be reused.

The load rating of the existing concrete spans indicate that they are capable of supporting a Cooper E-72 train loading, which they were originally designed for.

5 SEISMIC ANALYSES

5.1 *Seismic design criteria*

Project specific performance-based design criteria (Criteria) are under development for the Dumbarton Bridge. The principle design code reference is the 2007 edition of the AREMA Manual for Railway Engineering [AREMA 2007]. Criteria stipulated that the structure must comply with specified performance levels for two distinct seismic levels, an upper level seismic event with a mean return period of 1000 years that corresponds to the Survivability Limit State in AREMA and a lower level event with a mean return period of 91 years that corresponds to the AREMA Serviceability Limit State. At the time of writing this report Criteria was incomplete, but it is anticipated that Criteria will eventually specify the following:

- Allowable damage levels for each seismic demand level
- Material strain or force limits for each damage levels
- Drift and/or residual displacement limits
- Required analysis assessment procedures
- Methodology to consider seismic hazards

5.2 *Seismic analyses of Dumbarton main channel structures*

The existing Dumbarton Rail Corridor Bridge substructure is comprised of four unique construction types, designated as Segments 1 through 4 (See Table 1 below). Segment 1 consists

Table 1. Division of Dumbarton Bridge into four segments by bent types.

				Model properties	
Segment	Bents	Length (ft)	Description	No. of nodes	No. of elements
1	1–60	1770	[2]New 30 ft concrete box girder spans with 42 in. precast/prestressed concrete 3-pile bents	3709	3768
2	61–89	900	Existing 30 ft concrete box girder spans with 24 in. precast/prestressed concrete 4-pile bents	2967	3163
3	90–142	2340	Existing 45 ft concrete box girder spans with 48 in. precast/prestressed concrete 2-pile bents	3327	4119
4	143–159	1587	Existing navigational channel with superstructure replaced and new intermediate bents added	4439[1]	4469[1]
1, 2, 3 & 4	1–159	6597	Segments 1, 2, 3, and 4 combined into global model	14,459	15,123

[1]Properties of Retrofit Option 1.
[2]Subsequent to configuring retrofit models, spans for Segment 1 were increased to 60 ft. Four-pile super bents were added so that the longitudinal stiffness of the 30 ft and 60 ft spans remained essentially identical, thus not affecting the retrofit assessment.

of timber trestles; Segment 2 consists of 4-pile concrete bents; Segment 3 consists of 2-pile concrete bents; and Segment 4, the navigational channel, is constructed with large diameter concrete caissons.

Seismic finite element models were prepared incrementally using FE ADINA computer program recognizing that the Dumbarton Bridge is comprised of four unique segments as described in Table 1. For each segment, stand-alone models of individual bents were first prepared. These stand-alone models were useful for a number of reasons including checking modeling techniques used for global models, performing pushover analyses, and performing parametric studies of alternate retrofit strategies. Additionally, since superstructure flexural continuity was not provided across bents, the transverse responses of the stand-alone models are representative of the transverse response of the global bridge.

Global models only considered the portion of the structure extending from the West Abutment to the easterly end of the navigational channel, or the end of Segment 4 shown in Table 1. The total length of this portion of the structure is 6597 ft.

5.3 *Recommended seismic strategy*

The recommended seismic retrofit strategy for each of the four segments is as follows:

Segment 1, Timber Trestles—The existing timber trestles will be replaced with new construction.

Segment 2, 4-Pile Concrete Bents—These existing bents have numerous deficiencies including insufficient displacement capacity, inadequate bent cap strength, shear deficiencies in concrete piles, plastic hinges requiring retrofit below the mudline, and deficient shear keys. In consideration of these numerous deficiencies, replacement of the existing 30’ spans with longer spans is recommended.

Segment 3, 2-Pile Concrete Bents—Retrofit is recommended at Segment 3. The existing 48 in. hollow PC/PS concrete piles have sufficient displacement capacity to avoid retrofit to the piles below the mudline, with the exception of inserting isolation casings over the piles at selected locations. To increase ductility of piles at the connection to the pilecap in the transverse direction, a composite wrap is recommended. However, the composite wrap can be economically placed above the water level. Bent caps have sufficient strength to respond as capacity-protected members. The recommended retrofit at the bent cap is the addition of shear keys to ensure adequate transfer of forces between the superstructure and substructure.

Segment 4, Navigational Channel—Two retrofit options are recommended for further study. Option 1 requires the replacement of the superstructure, but retrofits the existing large diameter caissons with new piling and a concrete shell around the full circumference of the caissons. Additionally, to avoid the need to match the existing 180 ft superstructure spans, new intermediate bents will be added between caissons to reduce spans other than the moveable span to a maximum length of approximately 92-ft. Option 2 calls for the full replacement of all substructure and superstructure elements.

Of the two retrofit options, Option 1 provided the advantage of stiffening the entire structure and permitting the retrofit and reuse of Segment 3. This was due to the much greater stiffness of the retrofitted caissons. Since there are a number of considerations other than seismic performance that will influence the selection of replacement or rehabilitation of the existing substructure at Segment 4, it is recommended to continue studying both options and defer the selection of the preferred option until the regulatory agencies have commented on the two options.

6 PROPOSED SCOPE OF FUTURE RETROFIT OR REPLACEMENT

6.1 *General*

The summary of the proposed scope of the work in order to bring the existing bridge to current AREMA standards is based on the engineering studies completed to date. The studies included inspections, condition assessment and load ratings, seismic analyses, constructability review, and cost estimates. The work has been subjected to extensive peer reviews by SAMTRANS engineering staff and consultants. This scope of work could change in the future, based on continuing refinements, engineering studies and meeting regulatory agency requirements. At the time of completing this paper, the current proposed structure retrofit or replacement options are summarized in the Table 2 and highlighted below.

Section 1 & 11—Replacement of a 1,770 foot long section of timber trestles with concrete box girders.

Section 2—Replacement of a 900 foot long section of concrete spans with concrete box girders.

Section 3 and 10—Seismic retrofitting of sections of existing concrete box girders.

Sections 4 to 9—Replacement of existing truss spans with new concrete bridge sections. The work proposed for Section 7 includes replacing the existing swing span with a new movable bridge (Bascule or Swing).

6.2 *Substructures*

The superstructures in Sections 4 to 9 are supported on 100 year old pairs of caissons, as follows: One 13' Diameter, Twenty 18' Diameter (ten pairs) and One 40' Diameter.

There are two options for providing substructure support for the new superstructures in Sections 4 to 9 as depicted on Figures 17, 18 and 19. Option 1 includes retrofitting the caissons and installing 21 large diameter piles. Option 2 includes removing the existing caissons and installing new large diameter Cast-in-Steel-Shell (CISS) piles.

Both Options 1 and 2 are recommended to be further analyzed during the next phase of design. Both options are being carried through the environmental clearance phase.

Table 2. Dumbarton Bridge scope as of April 8, 2009.

SECTIONS	**1 and 11**	**2**	**3 and 10**	**4 through 9**	
Option Name	--------------	--------------	--------------	**Option 1 (Retrofit)**	**Option 2 (Replace)**
Existing Structure	Timber Trestle (portions burnt)	PC/PS Box Girder Spans on 4-Pile-Bents	PC/PS Box Girder Spans on 2-Pile-Bents	Main Channel Movable, Steel Thru Truss, and Transition Spans on Large Diameter Caissons	Main Channel Movable, Steel Thru Truss, and Transition Spans on Large Diameter Caissons
Proposed Structure	New Concrete Spans	Replace with Concrete Spans	Seismic Retrofit Existing Structures	Replace Superstructure with New Movable Bridge and 92' Concrete Spans - Retrofit 11 caissons (1-40', 9-18', 1-13') - Leave 9 caissons in place and remove 2 - Add: Seven new intermediate supports	Replace Superstructure with New Movable Bridge and 112' Concrete Spans - Remove All Caissons, - New Supports with Driven Piles
In-Water Construction Duration (With in-water restrictions)	Approx. 30~36 months in-water construction		Approx. 1~2 months in-water construction	Approx. 24~28 months in-water construction	Approx. 30~36 months in-water construction
Pile Driving **- Pile Type** **- # Piles** **- Length/Pile**	4' PC/PS pile 114 piles 110'/pile	4' PC/PS pile 36 piles 110'/pile	N/A	4.5' PC/PS, 3' steel, 4' steel piles 21, 120, 12 piles 125', 150', 150'/pile	6.0' CISS pile 54 piles 130'/pile
Total Piles / Length	114 piles/12,540 ft	36 piles/3,960 ft		153 piles/22,425 ft	54 piles/7,020 ft
Concrete in the Bay* - Existing **Change** **Total**	30 cubic yards + 513 cubic yards 543 cubic yards	188 cubic yards +43 cubic yards 231 cubic yards	1322 cubic 0 cubic yards 1322 cubic yards	14,959 cubic yards +10,373 cubic yards 25,332 cubic yards	14,959 cubic yards *decrease 11,447 cubic yards* *3,512 cubic yards*
Fill in the Bay* - Net Change	4200 cubic yards	1365 cubic yards	148 cubic yards	4271 cubic yards	1365 cubic yards

Note: PC/PS pile - Precast / Prestressed Concrete Pile
CISS pile - (Concrete) Cast-In-Steel-Shell Pile
"Concrete in the Bay" quantities include substructure from top of pile to mudline only (no superstructure included)
"Fill in the Bay" quantities include net change of substructure volume from mudline to bottom of pile only

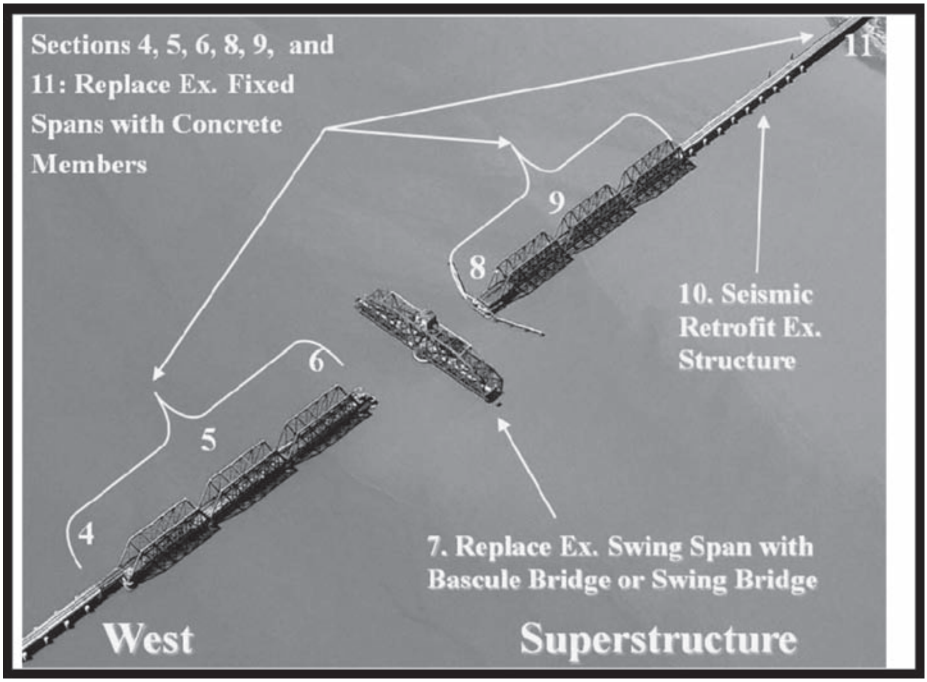


Figure 17. Proposed scope at Sections 4 to 11.

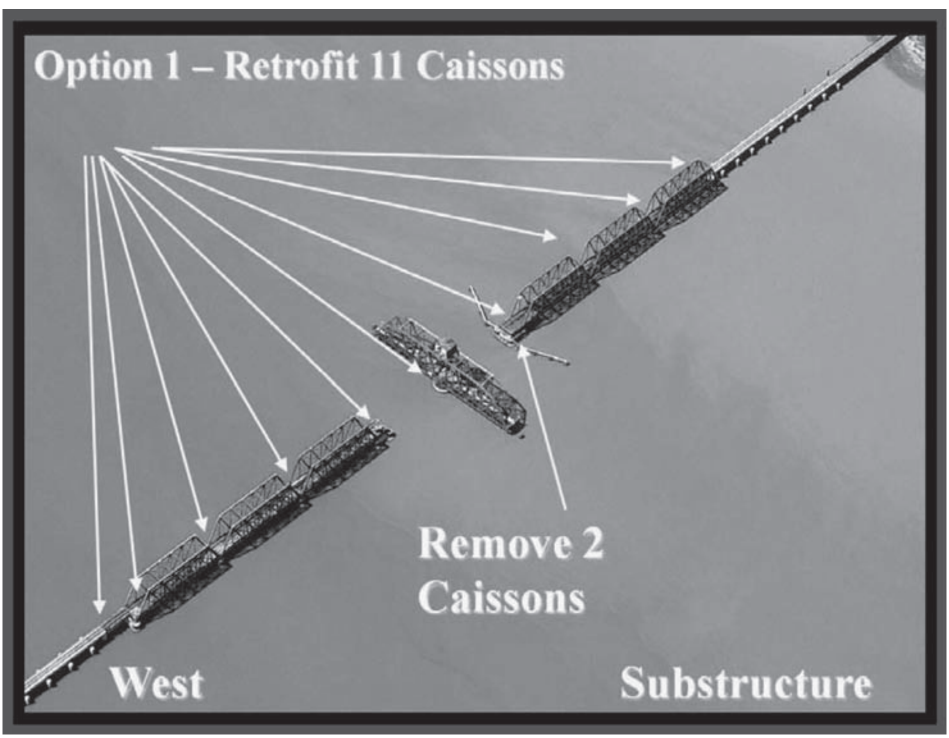


Figure 18. Option 1 (Retrofit) at Dumbarton main channel substructure.

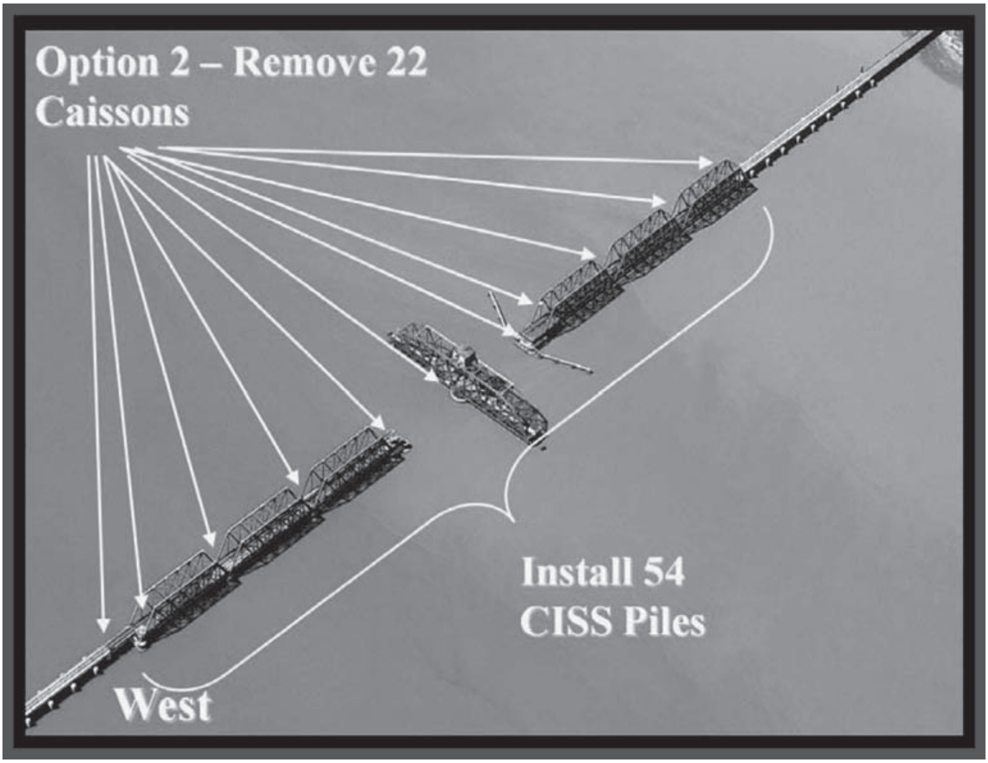


Figure 19. Option 2 (Replacement) at Dumbarton main channel substructure.

7 CONCLUSIONS

The engineering work performed on the Dumbarton Railroad Bridges has helped SAMTRANS determine the scope, constructability, cost and challenges of retrofitting or replacing the Bridges. It has also helped the environmental scientists determine the environmental impacts of the proposed retrofit or replacement of the Bridges. SAMTRANS will be circulating a draft environmental impact statement and environmental impact report (DEIS/R) on the DRC project in summer 2009. The decision on retrofit or replacement of the Dumbarton Railroad Bridges will depend on several factors including project cost, environmental impacts, constraints and public and agency reviews and comments. SAMTRANS will identify a Locally Preferred Alternative (LPA) including whether the Bridges will be retrofitted or replaced by the end of 2009.

REFERENCES

American Railway Engineering and Maintenance-of-Way Association (AREMA) (2007). Manual for Railway Engineering, Lanham, Maryland.

Schneider, E.J. 1913. Construction Problems, Dumbarton Bridge, Central California Railway, Transactions of the American Society of Civil Engineers, March 19, 1913. Paper No. 1271, 1573–1623.

9 Bridge deck evaluation

Chapter 35

Understanding the limitations of exodermic bridge decks: A case study on the Kingston-Rhinecliff Bridge

W.J. Moreau
New York State Bridge Authority, Highland, NY, USA

ABSTRACT: The Kingston-Rhinecliff Bridge is a 5,200-ft long deck truss, located in Kingston county of the Hudson Valley in New York, USA. The bridge was opened to traffic in February 1957, and has gone through much extensive deck repairs over the years. In 1997, a project began to replace the deck with an HS25 rated system. Based on a cost benefit analysis, a concrete filled grid, known as Exodermic Deck, was the best solution. This paper focuses on outlining some of the misconceptions regarding the proper application and performance with Exodermic Deck system. The shear flow mechanism between the steel and the concrete deck will be discussed. The paper will outline retrofitting measures for the inadequate deck system applications to provide a long and durable life.

1 INTRODUCTION

Panelized deck replacement has become popular in the bridge industry due to the cost savings associated with standardized panels, the time savings associated with prefabricated construction and the enhanced performance of various proprietary bridge deck systems. This paper will focus on one such system, the Exodermic Deck system, outlining some of the misconceptions regarding its proper application and performance.

Within one year after completion of the deck replacement project at the Kingston-Rhinecliff Bridge a number of cracks had formed over the main steel bars of the Exodermic grid panel. Six months after the cracking started the concrete was delaminating completely in localized areas. Although the cracking had developed in 8 of the 21 approach spans, only one span was suffering complete delamination of the concrete.

A thorough review was made of the design and construction records including the weather at the time of concrete placement and curing, slump and cylinder test results etc., no abnormalities were noted. A Finite Element analysis was made of the Exodermic panel by a prominent bridge design firm and alarming results were reported. The stress at the interface of the steel grid and concrete deck were clearly above the modulus of rupture of the concrete thereby causing the failure. Unfortunately the manufacturer's literature utilized by our bridge deck design group is not accurate in reflecting the expected performance of this system.

This product went thru a major design change in the mid 1990's and the lab testing of the new product was limited at best. The cracks that formed in the lab test section were erroneously passed off as insignificant. Furthermore the manufacturer offers products scaled up from the test section that were never tested and are much more susceptible to delamination failures like the one we experienced. This paper looks at the development of the new Exodermic deck section, the shear flow mechanism that connects the steel to the concrete deck, and how inadequate deck applications can be retrofitted to provide a long and durable useful life.

2 BACKGROUND

The Kingston-Rhinecliff Bridge is a 5,200′ long deck truss with girder span approaches of 1,100′ and 1,470′ (Figure 1). It is located approximately 110 miles north of New York City and opened in

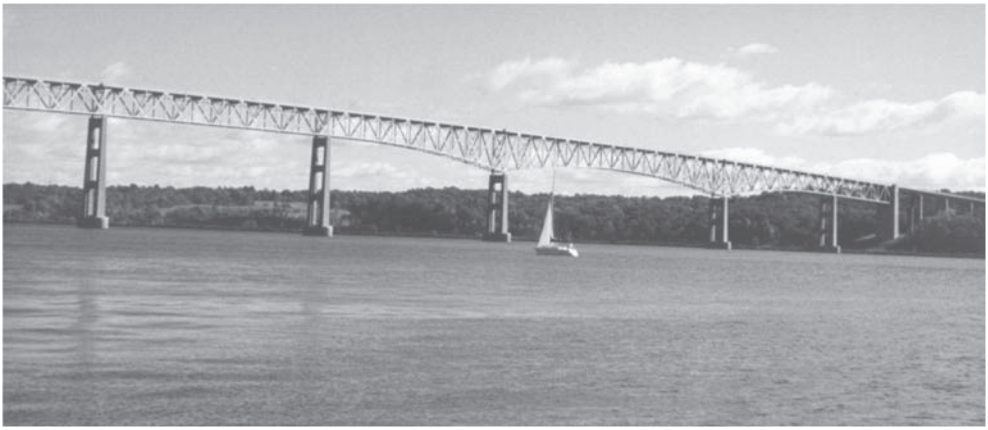


Figure 1. General elevation Kingston-Rhinecliff Bridge.

February of 1957. The steel superstructure was designed by David Steinman and the substructure was designed by his partner, Carl H. Gronquist.

The original 7-¼″ reinforced concrete deck had problems right from the start. During construction the screed for finishing the concrete was mounted on the fascia beams. These beams were smaller in size than a conventional roadway stringer and deflected under the loading of the screed. This produced a sag in the roadway profile at the mid-span of each 40′ panel. In 1958 it was decided to allow the contractor to pave the bridge with asphalt to improve the ride and settle the Bridge Authority's claim.

Unfortunately, this had the deleterious affect of holding moisture against the deck and potholes soon developed. Throughout the 1960's various deck repair patching materials were applied without success, even a cutting edge product from Shell Oil, Armor-Coat was ineffective as a durable repair.

In 1974 the entire overlay was removed, concrete deck repairs were performed, a Royston preformed sheet membrane was installed and a new 1-¼″ asphalt overlay was placed. Although this repair was more durable than its predecessors, patching began in the late 70's and the asphalt overlay was removed in 1984.

3 REHABILITATION

Once again extensive deck repairs were performed and this time a latex concrete overlay was applied. This system performed well, but the original slab was reaching the end of its useful life. During many of the deck repairs the reinforcing steel was found poorly distributed and corroded. The original deck design was based on a DOT design table and would rate only an HS17.8 with today's code analysis. HS20 has been the minimum design standard since 1944.

In 1997 a project began to replace the deck with an HS25 rated system. Various deck types were studied and the option of widening the bridge was evaluated. The original 36′ wide cross section was often not wide enough to perform maintenance activities while maintaining two lanes of traffic. It was determined to add two 5′ wide shoulders to the existing cross section and utilize concrete Jersey shaped edge barriers. Precast concrete panels, conventional filled steel grids, exodermic panels and orthotropic panels were studied (See Figures 2, 3). A concrete filled grid or an Exodermic system were the best solutions from our cost benefit analysis (N.H. Bettigole, P.C., 1991).

A contract was awarded in December of 1999 to the Cianbro Corporation of Pittsfield, Maine. Although the bid documents were based on a concrete filled grid, the Exodermic alternative was preferred by the contractor due to the more constructible longitudinal splice detail. The value engineering submission was accepted by the Authority using performance based criteria. Since

Figure 2. Placement of un-filled steel grid panels.

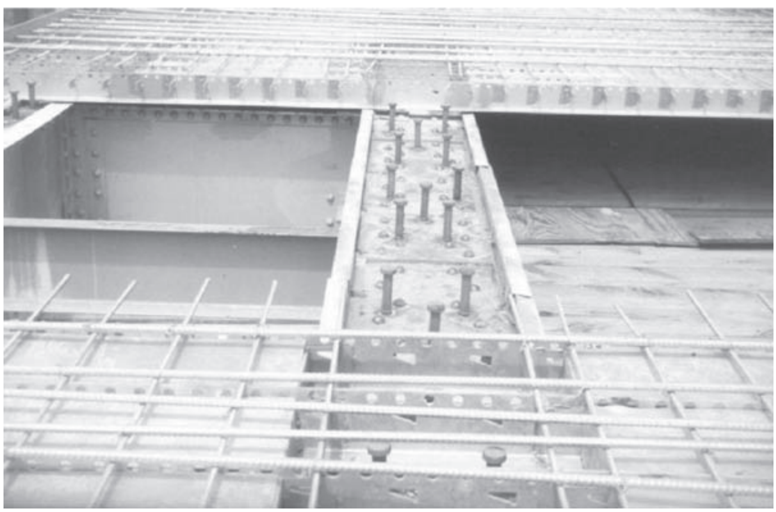


Figure 3. Steel grid panel and top mat reinforcement. (Note holes in transverse primary grid member. Concrete is placed from 4″ above the grid to 1″ below the grid thereby filling these holes and engaging the steel to the concrete).

this was a pre-engineered system, the "design" was performed by the manufacturer and an HS25 load rating performance level was specified. Construction began in 2000 and was completed at the end of 2002, one-third of the width was replaced the full length of the bridge each construction season. The final phase of construction completed the deck pours in August, earlier and under better weather conditions than the previous two seasons.

Cracking of the last construction phase was noticed in the spring of 2003. This last phase of construction was the north portion of the 1,100′ long west approach girder spans (Figure 4). This approach is curved along the majority of its length.

Figure 4. West approach girder spans.

While our first assumptions were that the curvature had something to do with the deck cracking, our analysis did not support this theory. The girder and stringer framing are tangential, creating a slightly larger overhang at mid-span than at the piers. Analysis of the Exodermic system found adequate reinforcement in the cantilever, but the anchor span for the cantilever was not adequately reinforced (Amman & Whitney, 2006).

4 ANALYSIS

A finite-element computer model was constructed of the deck system and the analysis indicated tension stresses in the bottom of the concrete will exceed the modulus of rupture in the span adjacent to the cantilever. Since the shear transfer mechanism between the steel grid and the concrete deck involves embedding the grid 1″ into the bottom of the concrete, a notch is formed in the concrete directly above the embedded steel web. The cracks in the deck were observed to be located directly above the primary members of the grid (Figure 5).

The William J. Rowley lab at Clarkson University did full scale testing of the Exodermic system (Higgins & Mitchell, 1997 and 1998), with emphasis on the shear transfer mechanism described above. Most tests were performed on simply supported panels with no cantilever or negative moment effects. Although the system was more than adequate in the test case, our application created conditions that were beyond the capacity of the Exodermic system. Furthermore, the manufacturer of our panels increased the spacing of the main members in the steel grid to 10″ vs. the 8″ spacing of the test panel. Although the 10″ grid spacing provides an adequate design assuming full composite action, the shear flow exceeds the panel's capacity in our application, cracks form above the embedded members and travel to the surface as delamination begins to occur (Figure 6).

Figure 5. Cracks, sealed and unsealed, forming above the main steel in grid panels.

Figure 6. Failed area of deck panel.

The reinforcement in the concrete slab is very light in the secondary direction, number 3 bars at 6″. The grid reinforcement in the secondary direction consists of ¼ × 1-½ steel bars at 6″. With the primary grid consisting of WT5 × 6 at 10″ centers and primary deck reinforcement being comprised of number 5 bars at 5″, the relative stiffness of the primary to secondary directions is on the order of 10 to 1. This does not create a grid nor does this system behave as a grid. Individual wheel loads create deflections of each WT member of the grid thereby causing shear failures during this deflection. Excessive strains in the bottom of the concrete form at the top of the embedded steel web and progress to the surface delaminating the concrete deck from the steel grid.

5 RETROFIT

Strain gauging was performed in the field on spans with limited cracking as well as the spans with the worst cracking (Figure 7). The predictions of the computer model were validated with excessive strains measured in the bottom of the concrete. While our consultants recommended removing the concrete, adding distribution steel to the grid and additional reinforcement within the deck and then recasting the concrete to insure a long lasting system, we were not enthralled with the idea of removing and replacing all the concrete so soon after the deck was replaced.

A prototype repair was conceptualized that would bolt on additional secondary reinforcement to the bottom of the grid to improve the wheel load distribution. The finite-element computer model predicted a modest improvement and one span of the bridge was retrofitted with three rows of L5 × 3 × 5⁄16 spaced at 28″ (Figure 8). The section modulus of this angle is comparable to the WT5 used in the grid. With the WT5 spaced at 10″ and the L5 × 3 spaced at approximately 30″, we have a grid relative stiffness on the order of 3 to 1. Even without additional distribution steel in the concrete deck reinforcement, we began to observe grid like behavior.

Once again strain gauging was performed on the retrofitted span and strains in the bottom of the concrete deck were reduced by a maximum of 70 percent. Strains in the new L5 × 3 indicated they

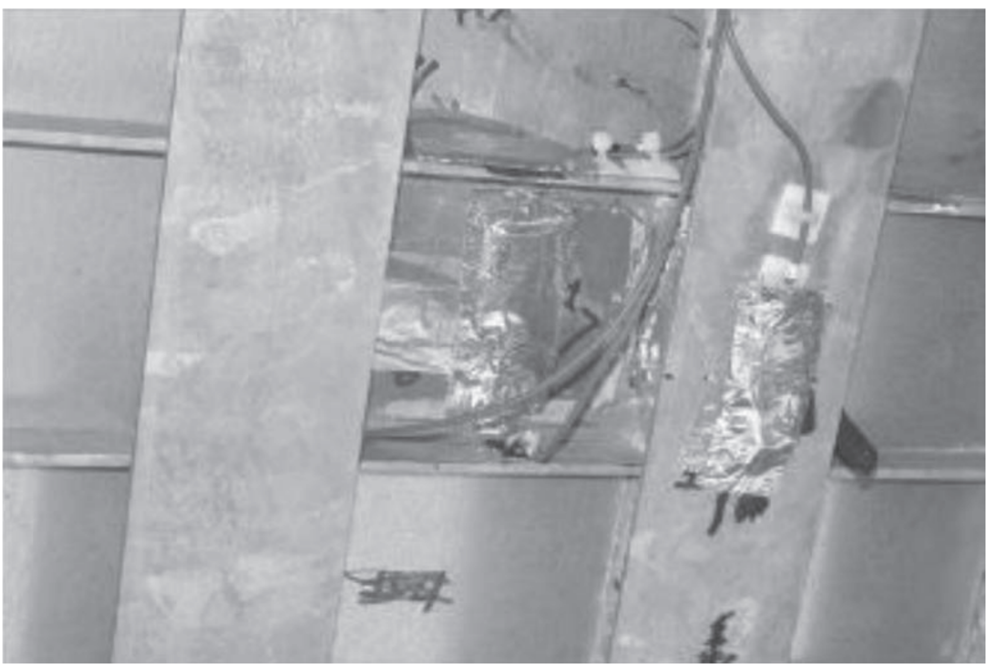


Figure 7. Strain gauge placement.

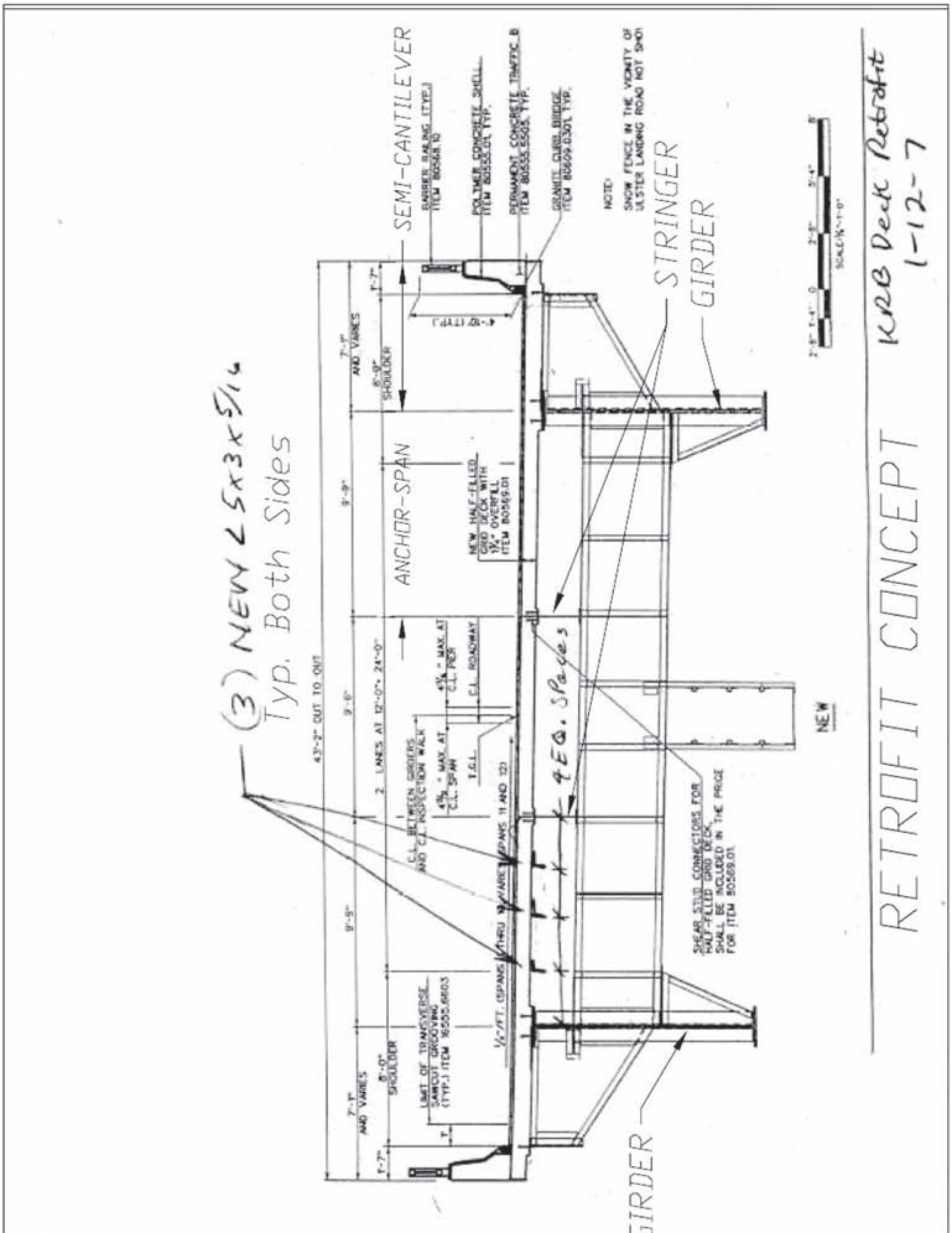


Figure 8. Retrofit concept with three rows of L5 × 3 × 5⁄16 spaced at 28″.

were very effective in attracting load. The retrofit was indeed capable of reducing the concrete strain to levels well below the modulus of rupture. It was determined the retrofit was a success and all approach spans adjacent to cantilevered sections have been retrofitted (Figure 9).

Strain gauge results of the un-retrofitted deck section showed that the measured strains were consistent with a partially composite system (Figure 10). Obviously the most actively cracked

Figure 9. Retrofit L5 × 3 × 5⁄16 installation.

Figure 10. Strain gauge placement.

sections of the deck were delaminating, but a huge majority of the deck was still behaving as a composite section.

The following sections were taken from the (Amman & Whitney, 2006) Finite Element Report prepared by Amman & Whitney, June 2006, on this study (Figures 11–13). The addition of the L5 × 3 not only improves wheel load distribution, but also lowers the neutral axis to improver stress/strain levels in the composite deck section.

Note that the neural axis for negative bending in the Longitudinal Section is nearly at the top of the steel grid, placing all the concrete deck in tension with only the transverse #3 bars at 6″ available to resist this load. After the retrofit, the neutral axis moves down such that the distribution

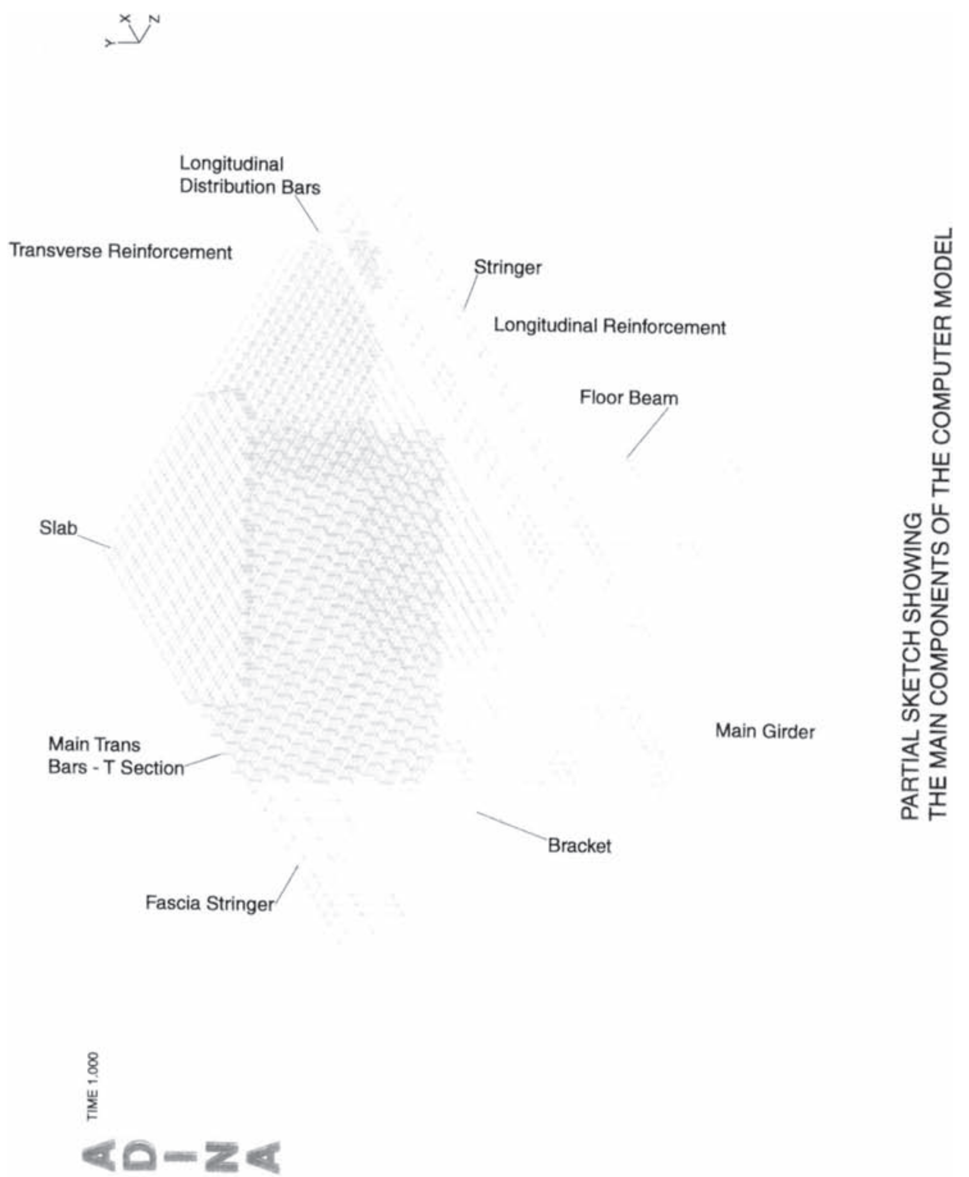


Figure 11. Partial sketch showing the main components of the computer model.

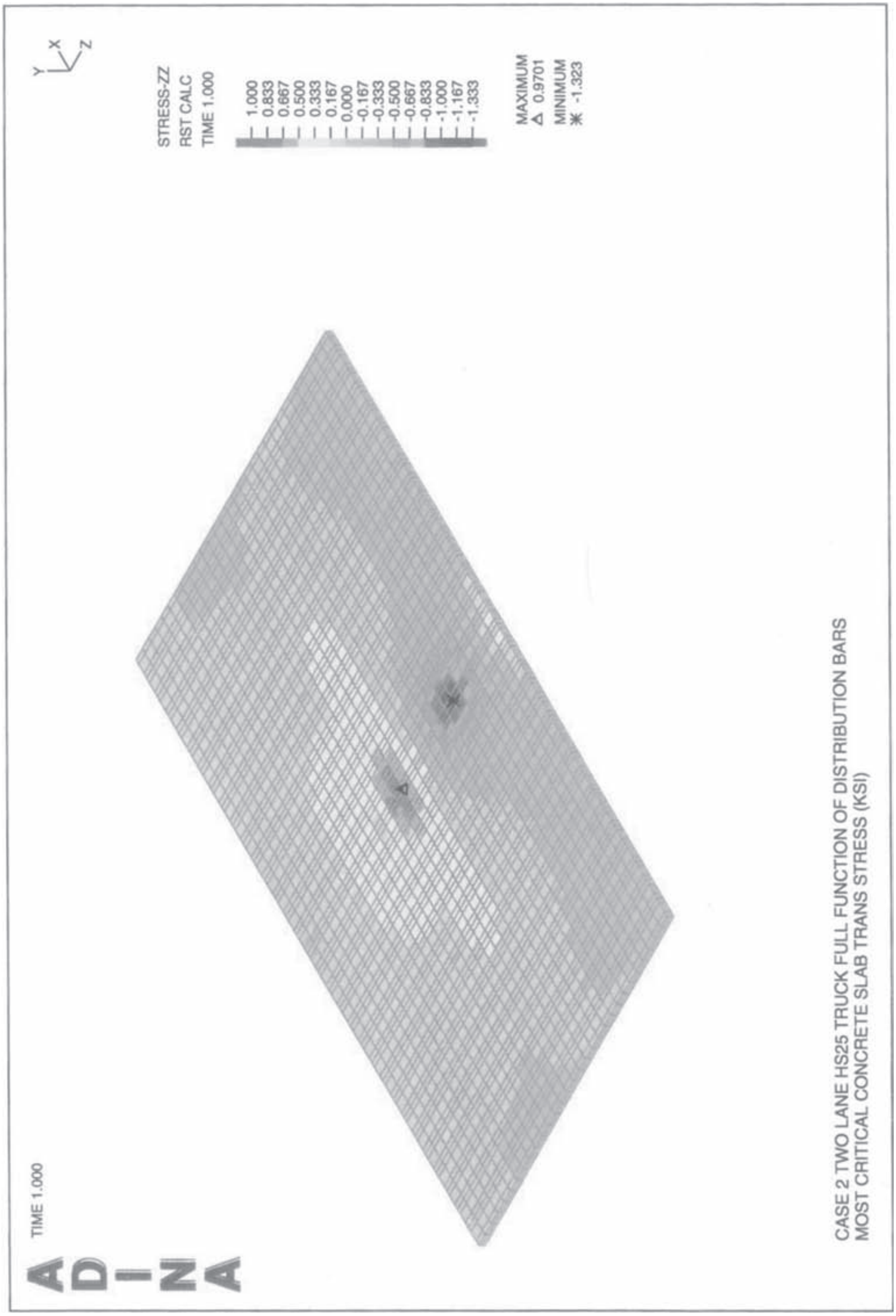
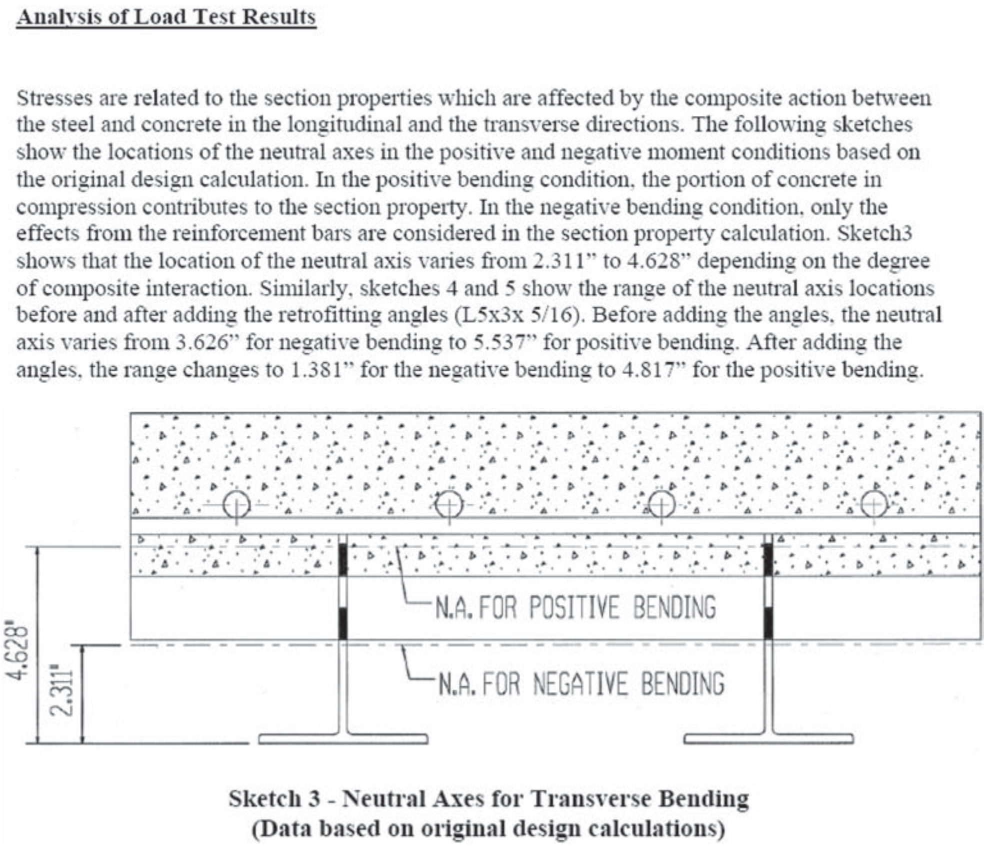


Figure 12. Case 2 two lane HS25 truck full function of distribution bars.



Analysis of Load Test Results

Stresses are related to the section properties which are affected by the composite action between the steel and concrete in the longitudinal and the transverse directions. The following sketches show the locations of the neutral axes in the positive and negative moment conditions based on the original design calculation. In the positive bending condition, the portion of concrete in compression contributes to the section property. In the negative bending condition, only the effects from the reinforcement bars are considered in the section property calculation. Sketch3 shows that the location of the neutral axis varies from 2.311" to 4.628" depending on the degree of composite interaction. Similarly, sketches 4 and 5 show the range of the neutral axis locations before and after adding the retrofitting angles (L5x3x 5/16). Before adding the angles, the neutral axis varies from 3.626" for negative bending to 5.537" for positive bending. After adding the angles, the range changes to 1.381" for the negative bending to 4.817" for the positive bending.

Sketch 3 - Neutral Axes for Transverse Bending
(Data based on original design calculations)

Figure 13. The locations of the neutral axes in the positive and negative moment conditions based on the original design calculation.

steel (¼″ × 1½″ bars) within the grid become effective in resisting the tensile load in addition to the rebars in the deck (See Figure 14).

A 6′ wide × 20′ long section of the worst cracked concrete was removed with hydro-blasting and re-cast. The number 3 bars were supplemented with number 4 bars at 6″ and ⅜″ diameter S-hooks were installed through the holes in the webs of the main grid members, every 12″. These S-hooks were tied to the top mat of the deck reinforcement prior to placing the concrete. The strain gauging of this proto-type repair also showed a significant reduction in concrete strains at the bottom of the deck. This repair will be used on the balance of the span to replace all delaminated concrete.

It has been decided that with the limited delamination that has occurred so far, the steel retrofit combined with a surface overlay will provide the useful life originally intended for the decks of the approach spans.

A thin asphalt overlay was installed to insure all the cracks were sealed. Nova-Chip was selected due to its proprietary hot applied tack coat which provides a tenacious bond between the concrete and the thin asphalt overlay, ¾″ (Figure 15). The effectiveness of this tack coat as a waterproofing membrane has been evaluated on our interstate bridge for the last three years. While the tack coat may not be as effective as some high performing waterproofing systems, it appears effective enough while the thin overlay dries.

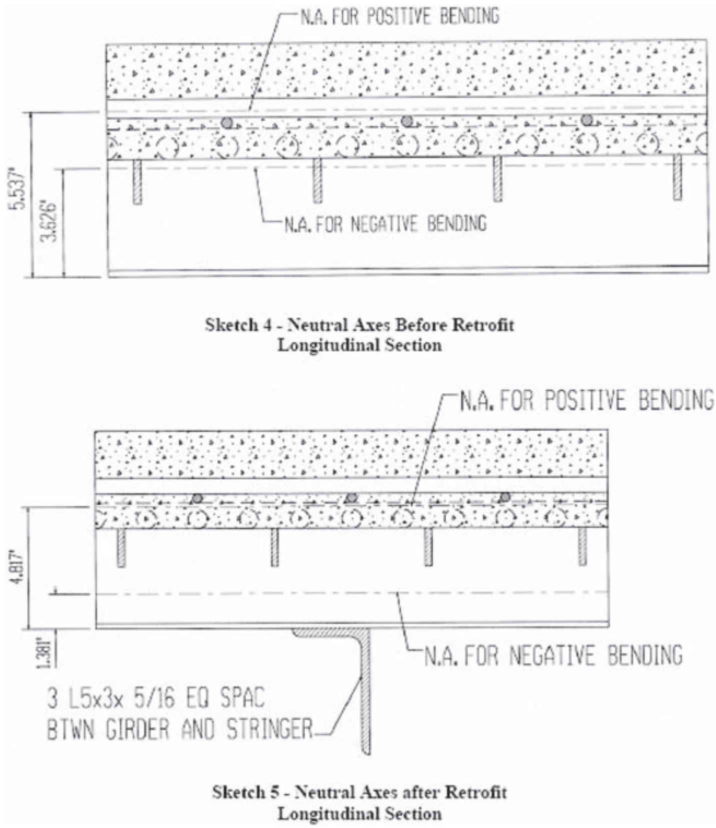


Figure 14. Neutral axes before and after retro fit longitudinal section.

Figure 15. Kingston-Rhinecliff Deck with Nova-Chip Overlay.

Based on this experience, I propose the AASHTO design committee review the code requirements for the design of exodermic deck panels. It seems apparent from this application that the shear transfer mechanism, that has been lab tested, does not extrapolate to grid panels that are ineffectively reinforced in the transverse direction. The code should require a minimum ratio of stiffness for the grid between the longitudinal and transverse directions. The exodermic system can not retain its composite nature with panels under-reinforced in the transverse direction. Distribution of the wheel load needs to be recognized as the primary design criteria for the transverse reinforcement. In our repair, I chose a minimum ration of (3) between the longitudinal and transverse section modulus. The current product literature does not protect against this misapplication.

For the reader's reference, I have appended to this paper the previous research performed on the second generation Exodermic deck panel.

CONCLUSIONS

The published section properties and performance data for exodermic deck panels do not reflect real world experience, especially with main bar spacings in excess of eight inches. The shear flow mechanism in the revised exodermic panel does not appear to be as durable as the mechanism in the original panel design.

Although our retrofit scheme has been successful in reducing the overstresses of the standard panel, I believe the current design guidelines and codes can give a misleading impression of the performance of an exodermic bridge deck panel. Clearly the use of the Grid section of the AASHTO code should require the steel grid panels to have a minimum ratio of strength between the main steel and distribution steel. Panels as lightly reinforced as ours clearly do not perform as a grid and excessive local deflections create cracking at the shear interface.

Based on this experience, I propose the AASHTO design committee review the code requirements for the design of exodermic deck panels.

REFERENCES

Amman & Whitney, *Kingston-Rhinecliff Bridge Eroding Study of the Exodermic Deck Full Scale Finite Element Computer Modeling*, 1996.

Bettigole, N.H., *Kingston-Rhinecliff Bridge Deck Replacement Study*, 1991.

Higgins, Christopher and Heath Mitchell, *Tests of a Revised Exodermic Bridge Deck Design*, 1997.

Higgins, Christopher and Heath Mitchell, *Fatigue Tests of a Revised Exodermic Bridge Deck Design*, 1998.

Chapter 36

Multi-level bridge deck evaluation combining Ground Penetrating Radar and Infrared Thermography methods

K.R. Maser & B.C. Miller
Infrasense, Inc., Arlington, Massachusetts, USA

ABSTRACT: Wisconsin DOT bridge maintenance engineers deployed nondestructive test methods on 183 bridge decks during 2007 and 2008 to more efficiently focus preventative maintenance and rehabilitation efforts. In addition to the usual visual inspections, Ground Penetrating Radar (GPR) and Infrared Thermography (IR) were used to determine bridge deck condition. For each deck a Level 1 evaluation was initially conducted to screen out decks in good condition, and a Level 2 evaluation was then carried out on decks that were likely candidates for future maintenance and rehabilitation. This multi-level approach utilized the strengths of each method to develop a cost effective way of assessing deck condition. The paper describes the methodology, the equipment used, the field data collection procedures, and the analysis procedures used to integrate the data.

1 BACKGROUND

Repair and replacement of deteriorated bridge decks and elevated roadways represents a major expense to many state highway agencies. An accurate description of deck deterioration conditions is necessary to efficiently focus preventative maintenance and rehabilitation efforts, and to make the best use of available funds. Such a description is often difficult to obtain, since the mechanisms of deterioration occur below the surface, and their manifestations are not readily seen in visual inspections. The limitations of visual inspections are compounded by the presence of overlays, where the top surface of the structural deck is no longer visible. The lack of reliable condition information has led to the replacement of sustainable decks (their "soundness" was revealed during demolition), and to the neglect of unsound decks until the resulting potholes and falling concrete require emergency response.

Ground Penetrating Radar (GPR) and Infrared Thermography (IR) are two technologies that have been applied to reinforced concrete decks to detect subsurface conditions and can be deployed globally over the entire deck with minimal disruption to traffic. They were first introduced for bridge deck condition assessment in the mid 1980s and are currently covered by two ASTM specifications (see references). Subsequent to their introduction, each technology has been developed and enhanced, by improvements in hardware, software, and understanding of the interpretation of data. In previous work there has often been a preference for one method over the other, but rarely have the two methods been used in combination, primarily due to cost. As discussed in this paper, the cost issue has been addressed by using a two-level approach to evaluation of the data.

2 DESCRIPTION OF THE APPROACH

Wisconsin DOT regional bridge maintenance engineers have utilized nondestructive testing over the past ten years for the evaluation of bridge decks. Each region has functioned relatively autonomously and has selected the testing methods that seem the most relevant to their needs. Some districts have used just IR or just GPR. Over the past two years, a two-level approach combining both IR and GPR was introduced and utilized by three of WisDOT's six regions.

2.1 *GPR and IR methods*

GPR data can be collected at normal driving speeds without lane closures and traffic control. GPR is equally effective on bare concrete decks as well as on decks with concrete and bituminous overlays. GPR data requires office processing to reveal deteriorated areas. The GPR results produce good quantity estimates and general locations, but are not as strong at precisely locating delaminated areas. While GPR is generally effective for deck slabs on girders, it has limitations with one-way slab decks. In addition to deck deterioration, GPR also provides detailed data on rebar depth and overlay thickness.

IR produces a visual delamination image immediately in the field that can be checked on the spot using sounding and cores, and thus provides good location accuracy. However, as a surface temperature method, its detection capability is depth limited. While it is very effective for non-overlaid decks, its effectiveness can be reduced in the presence of overlays. In these cases, infrared will clearly reveal the overlay debonding but may have limitations detecting rebar-level delamination if the rebar is deep. Also, IR data collection takes place at 5 mph and requires a moving closure, and solar radiation conditions must be adequate to produce the required temperature differentials.

From the description above, it appears that these two methods can complement one another. GPR rebar depth data is used for each deck to determine the depth limitations of IR detection. Where the rebar is too deep, IR is used to identify overlay debonding, while GPR is used to identify rebar depth deterioration. For decks that were not well suited for GPR, IR provides necessary deterioration information.

2.2 *Two-level evaluation strategy*

The two-level strategy separates the decks that are in good condition, and focuses the analysis on the decks that that are in poorer condition. For example, in the southwest region, if the deterioration levels are clearly less than 10%, then the no further analysis work is recommended. However, if there were indications that there was >10% deterioration, or if there was some uncertainty due, for example, to the presence of an asphalt overlay, the observed depth of rebar, or the underside condition, then the Level 2 analysis was recommended.

The Level 1 assessment involves collection of GPR and IR data, and a preliminary analysis for overall levels of deterioration. In some cases an underside visual inspection is included in Level 1. Based on this preliminary analysis, no further work is needed on many of the decks, while some are designated for a more detailed Level 2 evaluation. Level 2 involves detailed mapping of deteriorated areas, and specific quantity estimates for different levels of deterioration. In some cases, core samples are obtained for confirmation. The details of data collection and analysis are described in the following sections.

3 DATA COLLECTION

The data collection, described below, was the same for all WisDOT regions.

3.1 *Infrared thermography (IR) survey*

The IR survey was carried out according to ASTM D4788-05 using a high-resolution digital infrared camera and a color video camera operated from an elevated platform attached to a survey vehicle. Data was typically collected between 10 AM and 3 PM, to maximize the temperature differentials caused by debonding and delamination. Infrared and video data were displayed on live video monitors and recorded to digital media for later office processing. The survey covered one lane per driving pass, and the vehicle traveled at no more than 5 mph. A moving lane closure with two attenuator vehicles was typically provided for this survey. During the IR survey, delaminated and debonded areas revealed by the infrared data and observed in the real-time video were periodically checked with sounding to confirm the infrared observations.

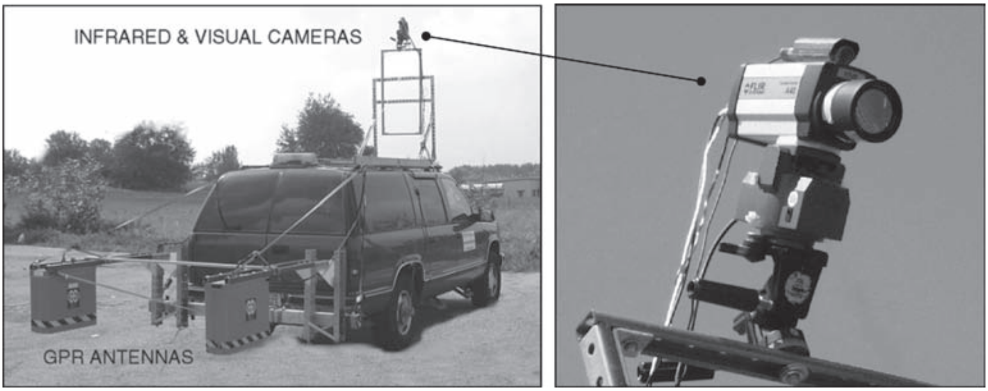


Figure 1. Survey vehicle equipped with 2 GPR antennas, 1 visual and 1 infrared camera.

3.2 *Ground Penetrating Radar (GPR) survey*

The GPR survey was carried out according to ASTM D6087-08 using twin air-coupled horn antennas suspended above the pavement surface. Multiple passes were carried out using a series of round trips across each deck. The radar data was collected on longitudinal lines spaced transversely at approximately 3 feet apart. The collection was carried out at normal driving speeds. The GPR survey requires dry pavement conditions, but otherwise is not temperature or weather dependent. Thus, the GPR survey work was carried out when solar conditions were not suitable for IR.

3.3 *Underside visual inspection*

The underside of the deck was visually observed, and areas of efflorescence, map cracking, rust staining, spalling, and exposed rebar were documented. It is understood that for water crossings, not all spans were accessible for underside inspection.

3.4 *Coring*

Cores were taken on some of the decks selected for Level 2 evaluation. Core locations were established based on Level 2 condition maps. The number of cores per deck varied from 3 to 8, depending on the size of the deck and the conditions revealed in the Level 2 analysis. Location maps and core coordinates were provided to the coring crew prior to commencement of coring operations for locating the cores in the field. All cores were 3 inches in diameter, and ranged in depth from 6 to 10 inches depending on the predicted rebar depth from the GPR data. The objective of the specified depth was to core 2 inches below the top rebar, so that rebar-level delaminations would be clearly revealed.

The core holes were examined to verify fractures and delaminations observed in the cores. After examination, each core hole was filled with rapid setting non-shrink grout. The cores were taken back to the lab, and their conditions documented and photographed.

4 DATA ANALYSIS

4.1 *Level 1 analysis*

A "Level 1" analysis was carried out for bridge decks surveyed with both IR and GPR. This analysis includes:

1. Review of the infrared data for estimated overlay debonding and rebar-level delamination quantities.
2. Review of the GPR data for estimated rebar depth and rebar-level condition.

3. Review of underside visual data.
4. Recommendation regarding the need for Level 2 analyses.

4.1.1 *Level 1 infrared analysis*

The IR data was collected in a series of passes across each deck. For a typical interstate deck with 2 lanes and left and right shoulders, the survey was carried out in four passes—one in each lane and one in each shoulder. Each pass covers a 12–15 foot width of deck. The survey results in a series of infrared images collected every foot of vehicle travel. After the survey, the center foot of each image is stitched together with the center foot of the next image, and so on, so that a single strip image is obtained for each pass. The strip image for each pass is placed next to those of adjacent passes to produce an infrared image of the entire deck, as shown in Figures 2 and 3.

The white blotchy areas on the IR image of Figure 2 indicate delaminations. These are "hot" spots where the surface temperatures are higher due to the thermal barrier produced by the delaminated area. The data in Figure 2 clearly shows >10% delamination. By contrast, the IR data in Figure 3 shows very little evidence of delamination.

4.1.2 *Level 1 GPR analysis*

The GPR data was analyzed to estimate rebar depth and condition. A typical 40-foot wide deck would have 13 lines of data, each representing a cross sectional slice of the deck at that offset. Figure 4 shows a sample of GPR data used for this evaluation. Note that due to the placement of a concrete overlay, the depth of the top rebar is over 6 inches below the deck surface. At this depth, the IR may not be able to reveal rebar-level delaminations.

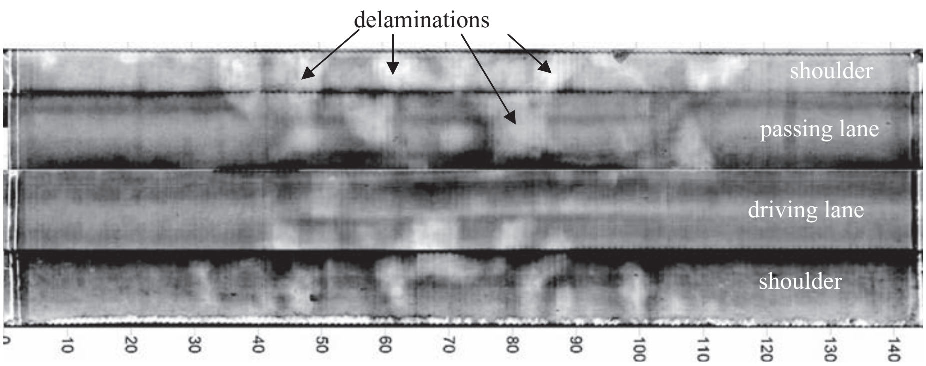


Figure 2. Sample IR deck image showing >10% debonding and delamination.

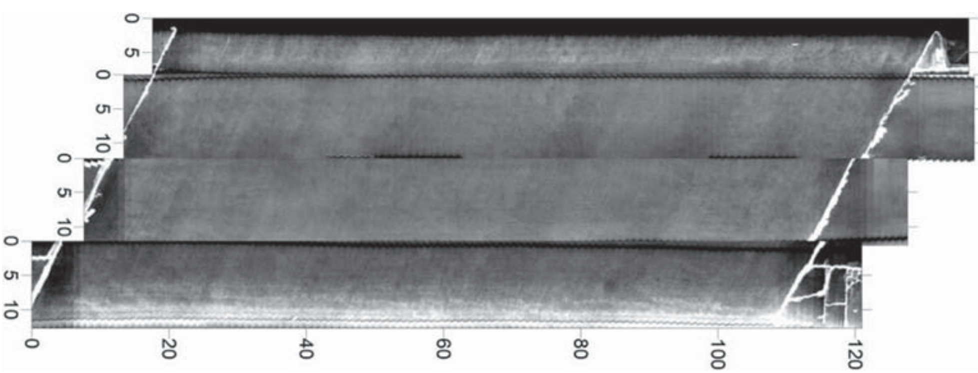


Figure 3. Sample IR image showing very little delamination or debonding.

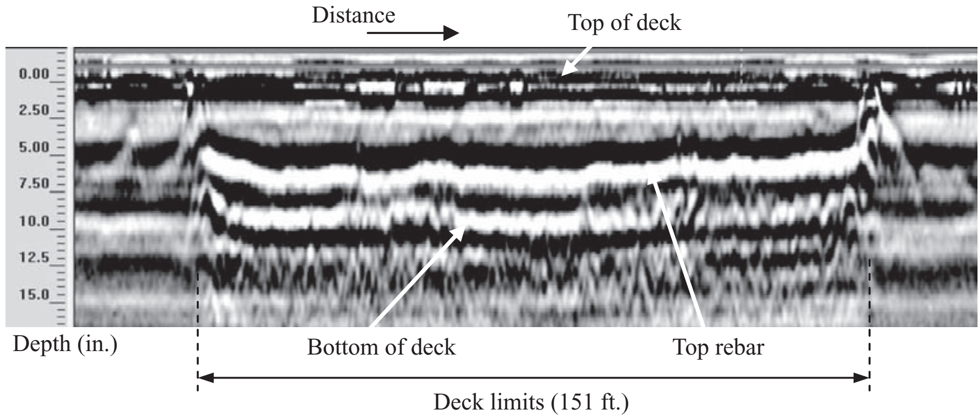


Figure 4. Sample GPR bridge deck data.

4.1.3 *Level 1 underside visual data analysis*

For some of the bridges, the underside of each bridge was visually inspected for evidence of cracking, efflorescence, spalling, rust-staining, and other signs of distress. These observations were mapped onto bridge plan sheets with supporting photographs. Figure 5 shows a photograph of underside conditions associated with deck delamination.

4.2 *Level 2 analyses*

The recommendation for a Level 2 analysis is based on the Level 1 IR, GPR, and underside analyses. If the deterioration levels are clearly less than the prescribed threshold, then the no further analysis work was recommended. However, if there were indications that deterioration, exceeded the threshold, or if there was some uncertainty due, for example, to the presence of an asphalt overlay, the observed depth of rebar, or the underside condition, then a Level 2 analysis was recommended.

Figure 5. Photograph showing efflorescence, spalling, and rust-staining.

4.2.1 *Level 2 deck condition maps*

The type of mapping (IR, GPR, or both) depended on the depth of rebar, type of deck, and the deck surface conditions. It was generally assumed that infrared detects overlay debonding and rebar-level delamination down to a depth of 4.5 inches. For decks that satisfied this condition, the mapping was done using the infrared data. Due to the presence of concrete overlays, many of the decks on this project had rebar depths from 5 to 6 inches. For these decks, it was assumed that rebar-level delamination might be missed by the infrared, and the GPR data was mapped to handle these conditions. In some cases, the presence of an epoxy overlay produced a mottled surface condition preventing the collection of useful infrared data. GPR was used for the analysis of these decks. A typical condition map combining the results of GPR and IR is shown in Figure 6.

The quantities determined from the detailed mapping of the IR and GPR data were and separated into "overlay debonding" and "rebar-level delamination" categories. The color scale for GPR indicates "severity", which is related to the combined effects of chloride contamination and

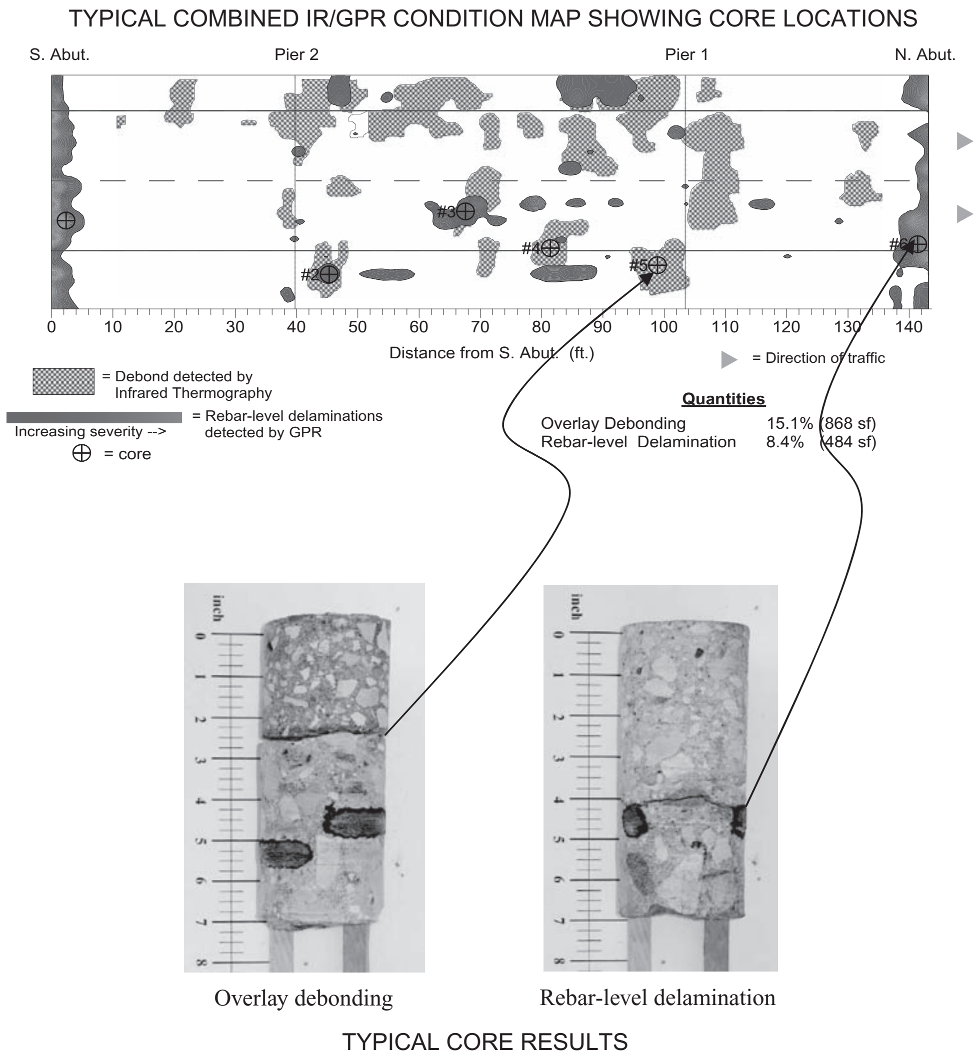


Figure 6. Typical combined IR/GPR condition map showing cores.

Table 1. Summary of core results.

Detected by	Total no.	Debonded no.	Delaminated no.	Correct %	False alarms %	Misses no.	Misses %
IR &/or GPR	105	25	61	83%	17%	5	5%
IR only	64	20	35	86%	14%	22	26%
GPR only	59	9	33	71%	29%	18	17%

Note that the 5 "misses" for the combined IR and GPR occurred when only one or the other method was used, so the benefit of both methods combined was not available.

rebar corrosion. When cores are used, their locations are shown on each map, along with their coordinates.

4.2.3 *Level 2 core results*

As indicated in Figure 6, cores were generally located to coincide with areas of deterioration predicted by either IR, GPR, or both. A small number of cores were taken areas showing no evidence of deterioration. Figure 7 shows typical core logs obtained on the project. Core results showed that the depth of top rebar predicted by the GPR is fairly consistent with rebar depths found in the cores. The thickness of the concrete overlay observed in the cores ranged from 2 to 4 inches and is significantly greater than the 1.5 inches specified in the plans. As a result, depths of top rebar on overlaid decks range from 4 to 6 inches. It is unlikely that rebar-level delamination at these depths could be detected by conventional chain drag techniques, a fact that underscores the value of the multi-level methods used on this project.

A summary of the core detection results is presented in Table 1. The table shows the method by which the core was selected, and the detection frequency of debonding and delamination. Note that there were 114 cores taken, so 9 cores were selected outside of areas indicated by IR and GPR. The table shows a high degree of accuracy in detecting debonding and delamination with IR and/or GPR. IR has the greater location accuracy, with 86% of the cores confirming correct readings vs. 71% for GPR. The table also shows "misses"; that is, defect conditions what were missed with one method but picked up with the other. Note that IR shows a higher percentage of misses compared to GPR. Of the IR misses, 18 of the 22 were delaminated. This confirms the earlier suggestion that IR has more difficulty with the deeper delaminations found in this project.

5 CONCLUSIONS

The use of Infrared Thermography (IR) and Ground Penetrating Radar (GPR) enabled the Wisconsin DOT bridge maintenance engineers to have a more comprehensive view of the condition of their bridge decks. The development of a multi-level condition assessment methodology has provided a more economical approach to deck evaluations since it minimizes analysis work on decks that are in good condition, and focuses on the decks that need the work. The combination of IR and GPR is very powerful, maximizing the capabilities of each method and compensating for the limitations. The core results collected as part of this project demonstrate a high degree of accuracy in locating delaminated and debonded areas, and provide a great deal of confidence in the overall deterioration quantities determined in this work.

ACKNOWLEDGEMENTS

The authors would like to acknowledge David Bohnsack, Greg Haig, and Tom Hardinger, Structures Maintenance Engineers for the Wisconsin DOT's Southwest, Northwest, and North Central

Regions, respectively, for their support and contributions to the overall direction of this work. The authors would also like to acknowledge Kris Olson and Paul Eggen of OMNNI Associates for their contributions to the under-deck evaluations and coring programs; and Adam Carmichael, and Laura McGrath of Infrasense for their contributions to collection and analysis of the GPR and IR data.

REFERENCES

ASTM 2005. "Standard test method for detecting delaminations in bridge decks using infrared thermography", Designation D4788-97, ASTM International, West Conshohocken, PA.

ASTM 2008. "Standard Test method for evaluating asphalt-covered concrete bridge decks using ground penetrating radar" Designation D6087-05, ASTM International, West Conshohocken, PA, 2005.

Carter, C.R., Chung T., Holt, F.B. and Manning D. 1986. "An automated signal processing system for the signature analysis of radar waveforms from bridge decks", Canadian Electrical Engineering Journal, Vol. 11, No. 3, 1986, pp. 128–137.

Manning, D.G. and Holt, F.B., 1980. "Detecting delaminations in concrete bridge decks", Concrete International, November, 1980, pp. 34–41.

Maser, K.R. 1990. "New technology for bridge deck assessment" Phase I and II Final report prepared for the New England Transportation Consortium, Center for Transportation Studies, Massachusetts Institute of Technology, Cambridge, Massachusetts.

Maser, K.R. and Roddis, W.M.K. 1990. "Principles of radar and thermography for bridge deck assessment", ASCE Journal of Transportation Engineering, Vol. 116, No. 5, Sept/Oct, 1990.

SHRP C-101. 1993. "Condition evaluation of concrete bridges relative to reinforcement corrosion—Volume 3: Method of evaluating the condition of asphalt-covered decks", Strategic Highway Research Program Report SHRP-S-325, Washington, D.C.

Chapter 37

The history and benefits of prefabricated grid reinforced concrete decks

P.M. Gase
The D.S. Brown Company, North Baltimore, Ohio, USA

M.R. Kaczinski
Bridge Grid Flooring Manufacturers Association, North Baltimore, Ohio, USA

ABSTRACT: Accelerated Bridge Construction (ABC) techniques are employed predominately to reduce on-site construction time, minimize traffic impacts, and improve work zone safety. Grid reinforced concrete bridge decks have been an economic, lightweight, rapid deck replacement option long before ABC became the vogue acronym defining the desired construction practice for the 21st century. The prefabricated modular nature of grid reinforced concrete deck naturally lends to reduced construction periods and offers the advantage of shortened work windows—nighttime or weekend—when traffic volumes are lower and the travelling public is less encumbered. In addition, the reduced weight translates into direct savings in the superstructure and substructure for new construction and can minimize superstructure rehabilitation and increase live load capacity of existing structures. A few recent projects accentuate the role of grid reinforced concrete decks as the solution to lightweight, rapid deck replacements.

1 INTRODUCTION

As engineers searched for a lightweight, high strength deck alternative, the first grid reinforced concrete decks emerged in the 1920's as tees butted together at their flanges and encased in concrete. In the 1930's, the fully filled I-beam Interlock decks were introduced. The steel grid consisted of specially rolled narrow beams (main bars) evenly spaced and connected with cross bars oriented normal to the main bars, evenly spaced and welded flush to the top of the main bar. In addition, round bar was placed in the lower section of the grid, oriented in the same direction as the cross bar. Thin gage sheet metal rests between the bottom flanges to support the full-depth concrete (Gilmore 1987) (see Figure 1).

In the 1950's, an intermediate flange was added at the center of the 5″ I-beam. This offered support for the form pans, which constitute the bottom of the concrete. This created a half-filled system that reduced the total weight of the deck without loss of spanning capacity (see Figure 2).

Exodermic™ decks (Unfilled Grid Decks Composite with Reinforced Concrete Slabs as defined in AASHTO LRFD Design Specifications) were introduced in the 1980's. It is a system of inverted WT shapes evenly spaced with distribution bars normal to and welded to the web of the tee. The tee webs penetrate 1″ into the bottom of a reinforced concrete slab and are made composite with the slab via closely spaced ¾″ diameter holes punched into the top of the grid (Bettigole, 1998) (see Figure 3).

Grid reinforced concrete bridge decks offer three major advantages over traditional reinforced concrete bridge decks. One is that they are lighter in weight. Grid reinforced concrete decks can weigh up to 50% less than traditional decks when span capacity is compared. For new bridge construction, this translates into a direct savings to the superstructure and substructure. For movable bridges, additional savings are realized in the counterweight, trunnion assemblies and drive machinery. For rehabilitated structures, specification of a lighter weight deck may reduce the amount of strengthening of the superstructure and result in an increase of live load capacity.

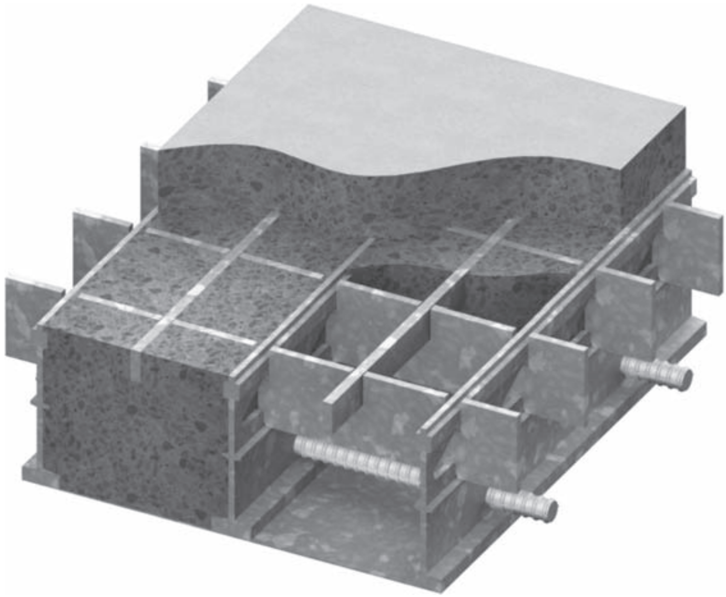


Figure 1. Fully filled grid reinforced concrete deck.

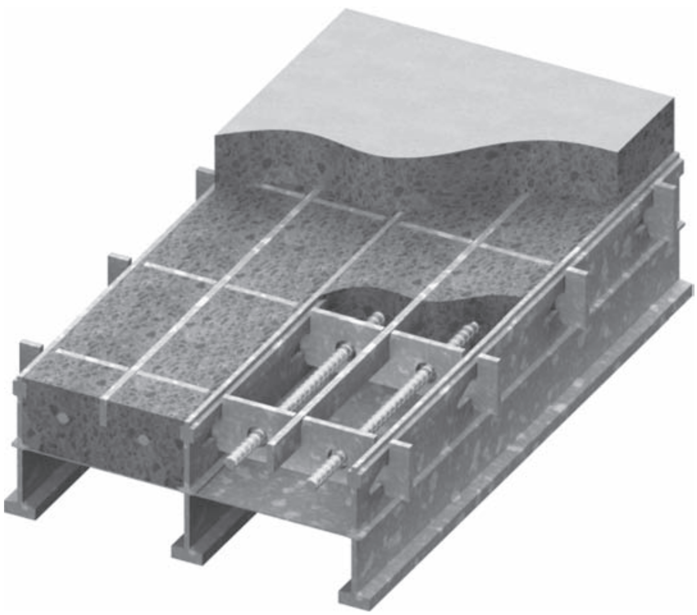


Figure 2. Partially filled grid reinforced concrete deck.

For some structures, the weight savings could also mean that a much needed widening of the deck is now possible.

The second advantage is their durability. There are many grid reinforced concrete decks, built in the 1930 to 1960 time period that are still in service today. The Bronx-Whitestone Bridge (1939) and the Mackinac Bridge (1957) are just a couple of high profile structures that still have their

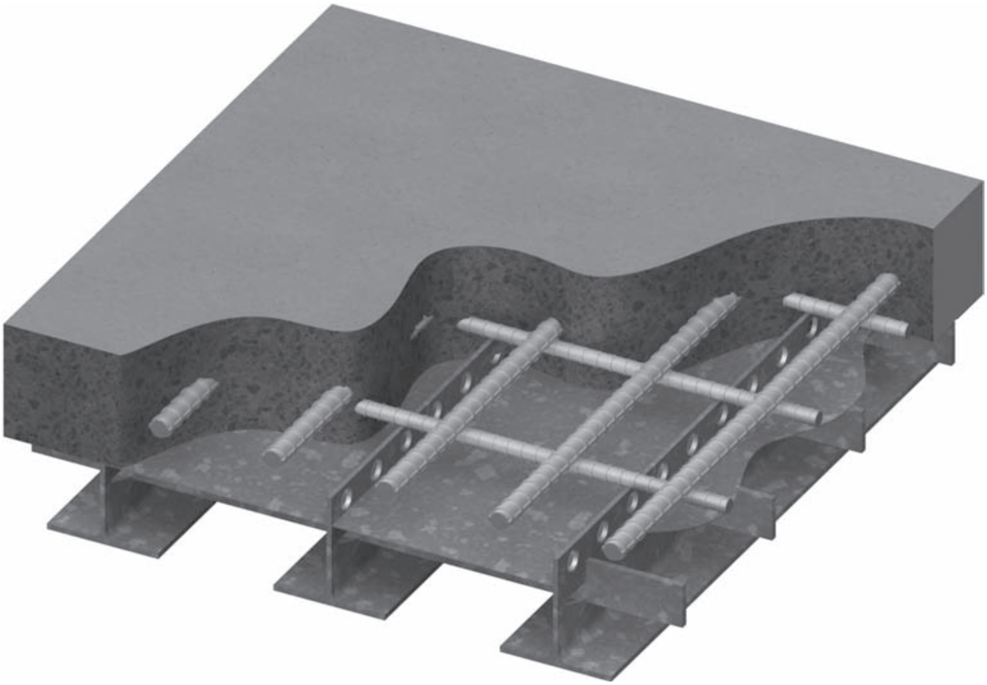


Figure 3. Unfilled grid deck composite with reinforced concrete (Exodermic™ deck).

original decks. Grid reinforced concrete decks are known to last twice as long as standard reinforced concrete decks. A perfect example of this is the Elizabeth Bridge over the Monongahela River in Allegheny County, Pennsylvania. The structure was built in 1950 with a grid reinforced concrete deck on the main span and a conventional reinforced concrete deck on the approach spans. Both deck types experienced the same traffic and were exposed to the same environmental conditions. In 1986, all of the approach ramp decks required replacement, whereas the main span only received a new overlay. There are several other mixed deck structures such as the Walt Whitman Bridge, the Thousand Islands Bridge, and the Homestead Bridge that have shown similar life cycles (BGFMA, 1994).

The third advantage is speed of installation. Grid reinforced concrete decks are modular by nature which when installed, can eliminate weeks or even months of field labor. For cast-in-place applications, the grid panel acts as its own formwork, leaving very little forming for the field. Precast panels also need very little forming and require only rapid-setting concrete closure pours over the supporting steel and between panels making them ideal for nighttime or weekend work when traffic is expected at the end of the work shift.

2 I-285 NEAR ATLANTA, GA

When it came time to replace the decks of two major bridges on I-285, with traffic volumes nearly 300,000 vehicles per day, the Georgia DOT (GDOT) Office of Maintenance wanted a rapid method of replacement that would prevent grid lock in Metro-Atlanta (Edwards, 2006).

GDOT approaches rehabilitation of bridge decks using four major construction methods: (1) bonded concrete overlays, (2) hydro-demolition and concrete overlay, (3) conventional deck replacement with high early strength concrete, and (4) pre-fabricated deck systems. Each method is evaluated based on the condition of the existing deck, impacts to traffic, and construction costs.

Prefabricated deck systems are considered when the cost associated with the impacts to traffic outweigh the additional material costs. While it is hard to justify the cost of prefabricated deck systems, anyone who commutes on I-285 in rush hour traffic understands the impact of lanes being shut down. The potential for accidents in construction zones and loss of revenue to companies shipping goods on these roadways must also be considered. GDOT contracted with J.B. Trimble, Inc. (JBT) for this fast track rehabilitation project. JBT is an Atlanta based consultant with extensive interstate deck replacement experience and practiced at premiering different technologies for bridge rehabilitation. Because of the condition of the deck, JBT recommended a prefabricated deck system utilizing a steel grid deck composite with a full-depth, overfilled precast slab. Although GDOT was unfamiliar with this type of construction and the costs exceeded that of conventional replacement, the ability to reconstruct the deck on weekends, thereby minimizing any disruption to traffic flow, justified the specification.

The bridge carrying I-285 over Cobb Parkway (US 41) is located approximately 1 mile west of the I-75/I-285 Interchange. Cobb Parkway is critical from a traffic perspective because it links major commercial areas in Cobb County. The bridge is 243′ long and 149′ wide, carrying four lanes in each direction. The superstructure consists of steel girders spaced at 6.5 ft. on center supporting a 6.625″ reinforced concrete deck slab.

The bridge carrying I-285 over Buford Highway (SR 13) is located approximately 1.1 miles west of the I-85/I-285 Interchange. Traffic on this bridge is heavily influenced by traffic conditions in the I-85 Interchange. Buford Highway is a vital link to commercial and industrial areas surrounding I-285 in DeKalb County. Two parallel structures span the intersection. The total length of each bridge is approximately 217 ft. The westbound bridge has girders spaced at 6.5′ on center supporting a 6″ concrete deck slab and carries six lanes. The eastbound side employs girders at 7.0′ topped by a 6.25″ deck and carries seven lanes. After widening in 1976, the bridge is now 197′ wide.

BGFMA member LB Foster supplied the 4¼ Inch Interlock steel grid panels to the prime contractor—The L.C. Whitford Co., Inc. (LCW) Georgia Division (see Figure 4). LCW specializes in accelerated bridge construction projects on the congested interstate system around Atlanta and is no stranger to the stringent demands of this type of work. Panels were then precast by LCW forces at their Alpharetta yard. Weekend work was confined to 56 hours—9:00 pm on Friday to 5:00 am on Monday, and the contract limited the number of available weekends to sixteen, but

Figure 4. Precast panel installation on I-285 near Atlanta, GA (Photograph courtesy of L.C. Whitford).

LCW was able to complete the 25,720 sq. ft. of deck replacement in eleven weekends. From GDOT's perspective, this project was considered a success largely due to the significant reduction in traffic impact. Because of this success, GDOT, JBT and LCW have since completed two other similar projects on I-75, and GDOT will continue to examine other sites for potential use of prefabricated grid deck systems.

3 MATHEWS BRIDGE IN JACKSONVILLE, FL

Designers and owners routinely select Exodermic™ decks for their light weight, design efficiency and the speed at which they can be placed and completed. When the home town design firm Reynolds, Smith & Hills, Inc. (RS&H) was given the task to replace the open grid deck on the channel span of the Mathews Bridge in Jacksonville, Florida, they had yet another reason for selecting an Exodermic™ deck—Ride Quality.

The Mathews Bridge, carrying Alternate US 90/SR 115 over the St. Johns River, is one of the most heavily traveled bridges in Jacksonville with average daily traffic at 74,000 vehicles per day. The bridge is a direct link for the populated east side to downtown for work, and recreation including the Jacksonville Jaguars stadium and the shops and restaurants at The Jacksonville Landing (Florida Engineering Society, 2009).

The bridge was originally built in 1952 with an open grid for weight savings, and there have been subsequent open grid deck replacements and rehabilitations. FDOT desired a closed deck system, which would protect the superstructure elements from moisture and debris and thereby extend the service life of the structure.

In 2006, FDOT awarded RS&H the contract to design a new closed deck system with the stipulation that the construction period be limited to 90 days. In addition to the required rapid replacement, the new deck would have to be weight conscious to replace the existing 18.5-lbs/sf open grid deck. A cast-in-place Exodermic™ deck with WT4 × 5 main bars spaced at 13.5″ and 4″ of lightweight reinforced concrete was specified to span new stringers spaced at 5′–10½″ designed to AASHTO LRFD Specifications vehicular design load (see Figure 5). The total weight of this

Figure 5. Bare Exodermic™ panels placed on Mathews Bridge (Photograph courtesy of RS&H).

Exodermic™ deck is 48.8-lbs/sf yet still providing 2″ of cover over the rebar. This translated into savings toward minimal required strengthening of the truss system and the existing floor beams.

Tampa based PCL Civil Constructors, Inc. was awarded the contract, and BGFMA member, Interlocking Deck Systems International (IDSI), supplied the Exodermic™ grid panels. The project was constructed in phases maintaining westbound traffic and detouring the eastbound traffic. The project was scheduled to be complete by the end of July, 2007, but because of the prefabricated nature of an Exodermic™ deck in which the deck acts as its own form work, PCL completed the project two weeks early much to the relief of the Jacksonville residents.

4 CONCLUSIONS

Grid reinforced concrete bridge decks have a long history of dependable service to the public and the industry. First appearing in the early twentieth century as an innovative lightweight design, they have been the preferred deck choice for many high profile structures such as the Verrazano Narrows Bridge, the Walt Whitman Bridge and the Brooklyn Bridge to name a few. Their durability has been demonstrated over time on numerous projects with many original decks still in service for more than 50 years. Grid reinforced concrete decks are also an effective alternative for projects that require accelerated replacement of existing bridge decks.

REFERENCES

Bettigole, Robert A., Exodermic Decks and Steel Bridges, *Modern Steel Construction*, November 1998.
BGFMA, "A Bridge To The Future," 1994.
Edwards, Garrick L., Steel Grid Prevents Gridlock, *The Georgia Engineer*, 2006.
Florida Engineering Society, January 2009.
Gilmore, Gene, Steel Grid Bridge Flooring: New Innovations to a Proven Modular Decking System, 1987.

10 Bridge history and aesthetics

Chapter 38

Christian Menn's recent bridge designs—Reducing structural elements to the simplest solution

E. Brühwiler
Ecole Polytechnique Fédérale de Lausanne (EPFL), Lausanne, Switzerland

ABSTRACT: The conceptual designs by Christian Menn of four landmark bridges are presented: 1) a 350-m span cable-stayed bridge with jointless deck girder, 2) a cable-stayed bridge with a single "spindle-shaped" pylon, 3) a bridge with an arch reaching high above the deck (both carrying a horizontally curved deck girder), and 4) a cable stayed bridge with three pylons monolithically connected to the deck girder. All of the original bridge designs are driven by the aim to optimize the flow of forces with the objective to reduce structural elements and their dimensions to a minimum.

The four bridges express technical efficiency by slenderness and transparency and emphasize the importance of understanding the functioning of the structural systems. A sound engineering concept is thus the solid basis for a far-reaching aesthetic quality and for the conceptual design of simple but elegant structures that offer a great crossing experience while respecting the functional requirements and providing technical performance.

1 INTRODUCTION

The art of bridge design consists in balancing economy and aesthetics against each other on a case by case basis to achieve the desired design objectives. Functional requirements comprising among others traffic loads and clearances, topography and geology, and consideration of state-of-the-art technologies need to be satisfied. The resulting bridge designs must be compatible with the environment and visually pleasing as an entity [Menn 1996, Menn 1998].

Albert Einstein's dictum stating that "Everything should be made as simple as possible, but not simpler" also applies to the technical-functional domain. In bridge design, reducing structural elements to the simplest solution is always a meaningful approach that often leads to original structural shapes with an aesthetic quality determined by natural science.

Following this approach, the conceptual designs for the upcoming construction of four landmark bridges are presented: a 350-m span cable-stayed bridge with jointless deck girder, a cable-stayed bridge with a single "spindle-shaped" pylon, a bridge with an arch reaching high above the deck (the last two carrying a horizontally curved deck girder), and a cable stayed bridge with three pylons monolithically connected to the deck girder.

The technical efficiency of all of these bridge designs is achieved by optimizing the flow of forces with the objective to reduce structural elements and their dimensions to a minimum. A sound understanding of the technical functioning of the structural systems with respect to the functional requirements is central. The result of this engineering design process are esthetically pleasing structures characterized by their bold slenderness and transparency.

2 CABLE-STAYED BRIDGE OVER THE LAKE OF GRIMSEL, SWITZERLAND

Plans to raise the Grimsel dam and lake require redrawing of the road over the Grimsel Pass. Close to the Grimsel Hospiz, the road has to cross more than 300 m across the artificial lake. Preliminary studies showed that it is advantageous to bridge the lake in one leap.

In 2005, Christian Menn designed a cable-stayed bridge crossing the lake with a single span of 352 m supported by two vertical 75-m high inversed Y-shaped pylons (Figure 1). The slender deck slab with a total width of 10.9 m and a thickness of 1.2 m is carried by two slightly inclined planes of stay cables that are fixed at the deck edges and provide torsional stiffness to the deck. The two planes of stay cables are fixed along the pylon needle and are bundled in one plane and back-anchored to the nearby rock.

The aesthetic expression of the bridge is characterized by the slender deck and pylons having a simple and pure shape as well as the semi-fan arrangement of the stay cables and their concrete anchorage blocks shaped like mountain crystals. This bridge structure is the result of a design process reducing structural elements to meet the given functional and environmental requirements.

Exposure to severe environmental conditions at 2’000 m altitude in the Swiss Alps necessitated the detailed investigation of three major environmental actions on the structure:

Wind speed measurements from the weather station at the nearby Grimsel Hospiz were available. These measurements were translated to the specific conditions of the bridge site which is somewhat protected from the wind in a valley-like location below the weather station.

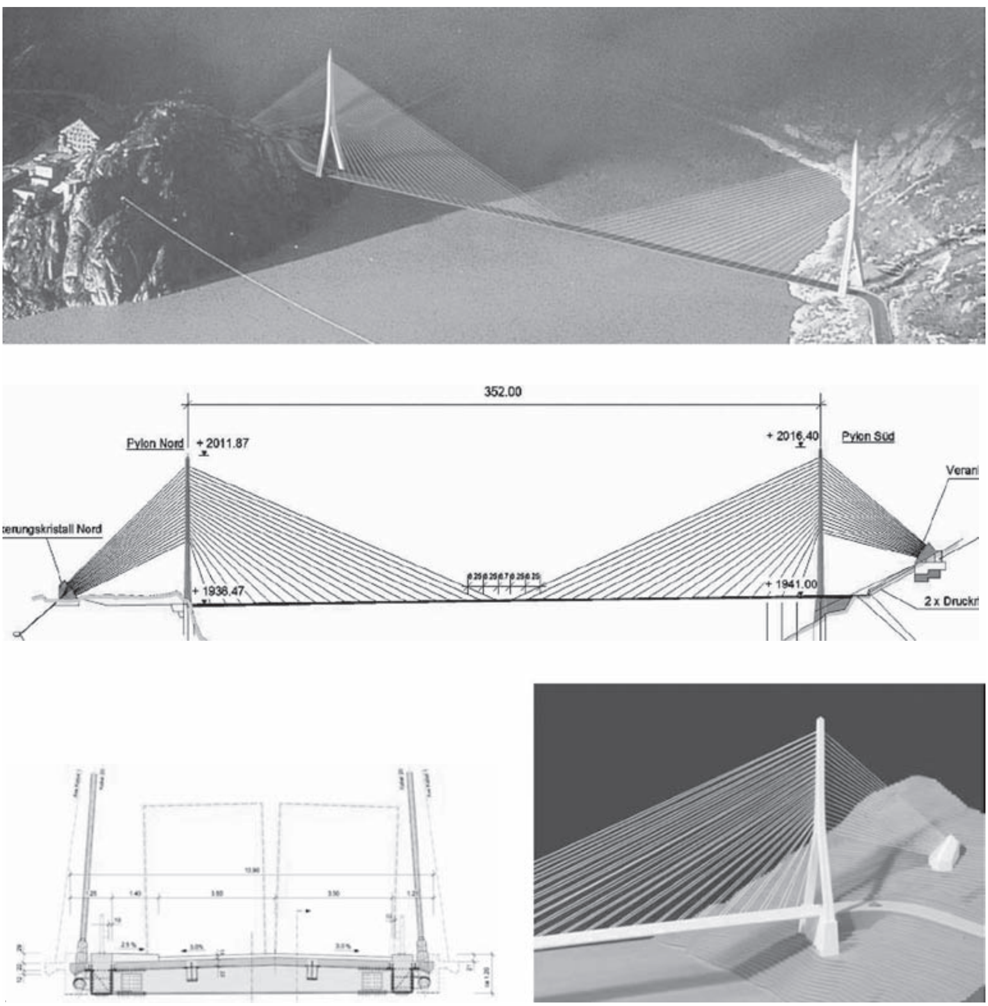


Figure 1. Cable-stayed bridge over the artificial lake of Grimsel.

The measurements revealed that the governing wind action is in the north-south direction almost along the bridge axis and the wind action in the east-west direction perpendicular to the bridge is relatively small. A maximum wind speed of 57 m/s was considered that lead to the design wind force of about 2 kN/m^2 (with 50 year return period) in the north-south direction. The cross section of the deck girder was developed based on aerodynamic investigations. Although the wind action perpendicular to the deck is relatively modest, studies of the aerodynamic behavior of the deck revealed that a "nose"-detail along the edges of the deck results in significant wind stability of the deck. Furthermore, the initially designed tapering of the deck width approaching the abutments has been proven not to be necessary for resisting the lateral moments, thus leading to a further simplification of the structure.

The deck slab is a composite steel-concrete construction rigidly fixed to the abutments located at the base of the pylons. It will be built from each pylon in segmental sequence to the mid-span where the two deck halves will be rigidly connected without a joint. In fact, the structural analysis revealed that the initially designed hinge at the mid-span is not necessary as the dilation of the jointless deck subjected a temperature variation between −20°C and +30°C is resisted by the structure. However, in order to limit built-in stresses in the restrained deck, the temperature at the time of connecting the two deck halves must be about 5°C.

The third environmental action to consider concerns the formation of ice on the stay cables when freezing rain accumulates on them to form icicles. However, this phenomenon is not significant since such icicles drop off due to their dead weight. Another phenomenon leading to an ice crust on the stay cables occurs when, at temperatures ranging from 0°C to −10°C, undercooled fog flows in laminar flow around the stay cables. In analogy with observed ice formation on high-voltage power lines, an ice thickness of 10 cm (corresponding to an additional load of 0.5 kN/m on the cables) is considered in the design.

Considering all these actions on the structure, the maximum vertical deflection of the optimized deck under design traffic load as the governing action was calculated to be 1/450 of the span, while the maximum horizontal deflection due to wind as the governing action is 1/400 of the span.

Being part of a major hydroelectric renewal project in the Grimsel area that is in the process of getting concession, the bridge project is awaiting its construction.

3 CABLE-STAYED BRIDGE IN ABU DHABI, UNITED ARAB EMIRATES

The Al-Sowah Island project in Abu Dhabi will comprise the Abu Dhabi Financial Centre, low density housing units and several high-rise buildings providing among others offices and retail areas. A three lane carriageway in each direction will link the offshore island to the main land. The roadway is curved with a radius of 1'000 m.

The design developed by Christian Menn in 2008 is a proposed cable-stayed structure with a single 118 m high pylon between two main spans of 205 m, bridging the waterway (Figure 2). The major design challenge is due to the curvature of the deck slab leading to significant torsional moments to be resisted by the structure.

The cable-stayed structure is characterized by the "spindle-type" pylon and a fan-shaped cable arrangement in one plane that follows the roadway curvature in the middle of the deck. Menn developed the spindle pylon when studying the optimal solution for cable-stayed bridges with very large decks in retrospect of his design of the Leonard Zakim Bunker Hill Bridge built in 2002 in Boston. The spindle pylon is composed of a needle for cable fixation and a bracing system which provides the high lateral resistance of the pylon to bending forces due to eccentric live load or horizontal curvature of the roadway deck. This enables in return a simple and slender cross section for the roadway deck, thus avoiding complicated and heavy box cross sections that are often necessary for cable-stayed bridges with central pylons.

The curved deck slab with a thickness of only 2.6 m is suspended from the pylon by one plane of stay cables. Since the stay cables fixed along the curved centerline of the girder transmit their force from their slab fixation along a straight line to the tower needle, significant deviation forces

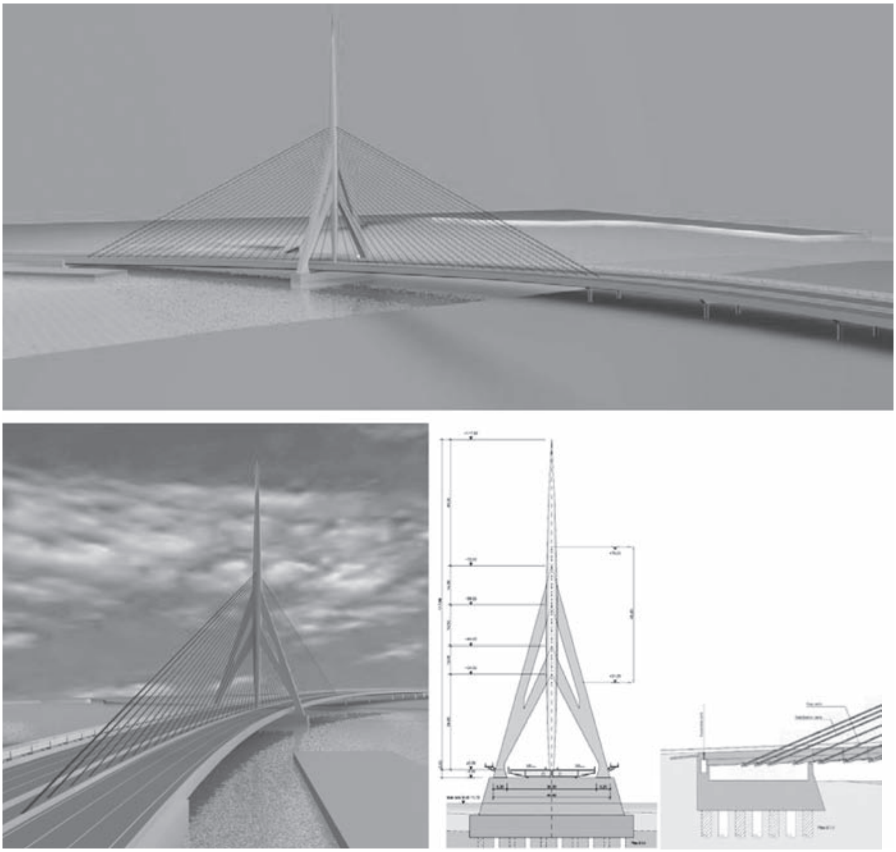


Figure 2. Cable-stayed bridge with curved deck in Abu Dhabi.

result (in particular from the longer stay cables with the largest eccentricity from the bridge axis) that act in the lateral direction to the pylon. The important lateral bending of the pylon is resisted by the chosen spindle form whose strong double-arm bracing provides the necessary lateral stiffness. Thus, the torsional moment due to the permanent loads (dead loads) of the curved deck is taken by the pylon's lateral resistance. The structural system is balanced for permanent loads and only small bending moments due to vertical and horizontal forces appear in the deck. Essentially only the (circular) torsion due to an asymmetric alignment of traffic load needs to be carried by the deck, thus allowing for relatively slender box cross section.

The first bearings adjacent to the cable-stayed spans, i.e. at the abutment and first pier respectively on each side of the bridge, are of outmost importance. On the one hand, the cable stayed system is stabilized and stiffened in the longitudinal direction by the top three stay cables that are anchored beyond the first deck bearings (Figure 2, bottom right). On the other hand, the horizontal bending moment in the deck due to the cable eccentricity with respect to the bridge axis is resisted by the rigid connection of the deck at these first bearings.

The bridge is in its final stage of design waiting for its construction in the near future.

4 ARCH BRIDGE IN ABU DHABI, UNITED ARAB EMIRATES

This bridge is part of a roadway extension to the Al-Sowah Island in Abu Dhabi. The roadway in the case of this bridge is curved with a radius of 900 m. The adoption of the curved deck is the result of a practical requirement and aesthetic considerations. The bridge carries a three lane roadway and a lightweight railway in each direction leading to an overall girder width of 42.1 m (including a central zone of 4.0 m for the arch and the vertical cable plane. In addition, a pedestrian lane of 2.5 m width is attached laterally on one of the inner edges of the curved deck.

The main design challenge was again the curved deck. Menn developed a structural system consisting of a 117 m high steel arch for a deck span of 170 m (Figure 3). The structure's main feature is this rather high arch with respect to the span which shall offer a great crossing experience and make this bridge a showpiece object. In addition, the aesthetic expression of the arch will be enhanced by a zinc-titanium cladding glittering in the sun.

The arch shape follows the girder axis and thus a spatial curve, i.e. the arch is always tangential to the girder axis. As a consequence, the resultant arch force is eccentric with respect to the arch base at the abutments that leads to significant out-of-plane second order effects.

The torsional moment due to the permanent loads (dead loads) of the curved deck is taken by the two inclined outer cable planes and the deck is suspended from the arch in the middle by a

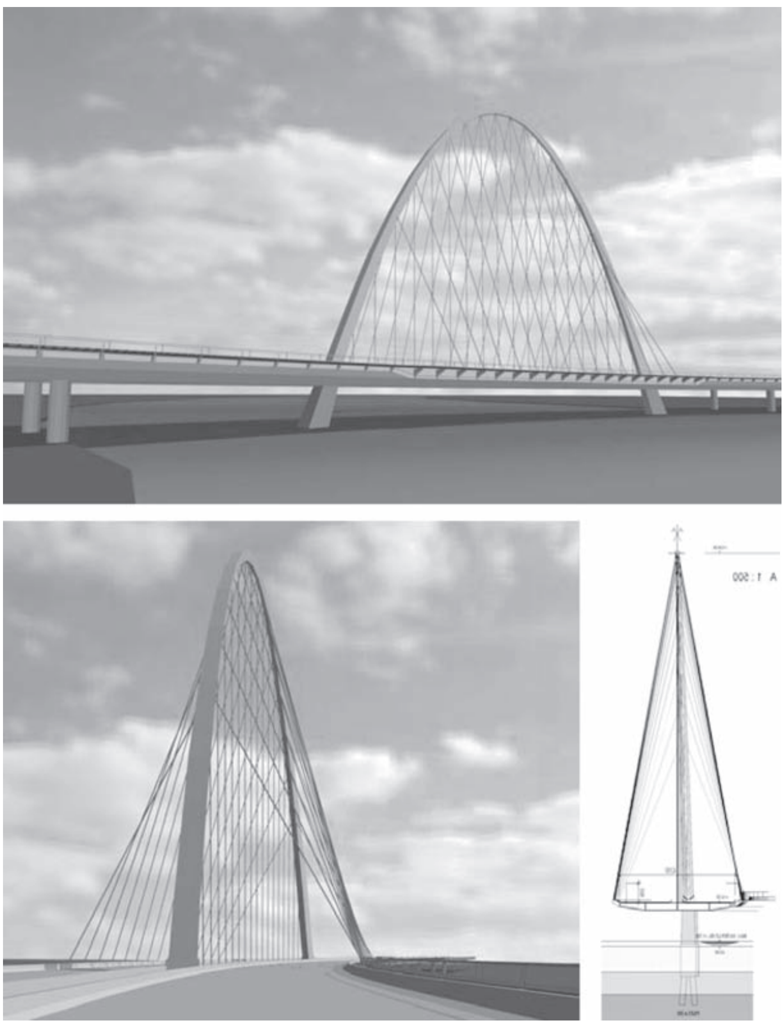


Figure 3. Arch bridge with curved deck in Abu Dhabi.

vertical cable plane. The difference in the forces between the outer cable planes is counteracting this torsional moment. In addition, the outer cable planes stabilize the arch. In this way, the structural system is balanced for permanent loads. In order to optimize this balancing, some ballast (concrete) is further added to the inner edge zone of the deck.

Bending and torsional moments are resisted by the cross section of the box girder deck which is rigidly fixed to the short and thus stiff stub-like piers in the prolongation of the arch. The torsional moment due to asymmetric traffic load is resisted by the box section while the asymmetric traffic load along the bridge axis is carried by the deck's bending resistance and the stiff piers forming the arch abutments.

Overall, the present structural system can be seen as a new generation of the well-known deck-stiffened arch bridges developed by Robert Maillart. Vertical loads are suspended from the arch by means of the cables, while the arch transfers torsional and bending moments to the deck girder which itself is rigidly fixed to the stiff piers.

Similar to the previous bridge, the project is in its final design stage waiting for its construction to begin in the near future.

5 PEACE BRIDGE OVER THE NIAGARA RIVER, BUFFALO, USA

The existing beautiful multi-span steel-arch bridge built in 1927 over the 700 m wide Niagara River between Buffalo (USA) and Fort Erie (Canada) needs to be expanded to accommodate for future traffic needs. Contacted by the Peace Bridge Authority, Christian Menn developed between 2001 and 2006 several cable stayed designs for a "signature" bridge. Bridge design competitions were held and finally in 2006 Menn's design of a cable-stayed bridge with two spindle-pylons and a slender roadway deck with a main span of 500 m, aligned parallel to the existing bridge was selected for construction (Figure 4).

However, environmental considerations resulted in a strong recommendation to drop the bridge design with 173-m (567-ft.) high pylons. In fact, a review has determined that the height of the two pylons would have unacceptable impact on migratory birds, and it was requested to review the design in favor of a lower profile bridge. Furthermore, it was decided that the new bridge shall replace the existing one.

To meet these new requirements, in 2008 and 2009 in collaboration with Linda Figg from the Figg Engineering Group, Menn developed a structural system comprising a three pylon

Figure 4. Peace Bridge—Companion bridge design.

cable-stayed bridge structure with a harp shaped stay cable arrangement to carry a roadway with a total width of 20 m (65 ft.) (Figure 5). The pylons with two needles vary in height, the highest being 72-m (236-ft.) high, for spans of 114 m (374 ft.) –245 m (805 ft.) –268 m (880 ft.) –137 m (450 ft.). The highest tip of the structure is 84 m (276 ft.) above the water level.

The main idea of the design is to provide three stiff triangular cable-stayed structures with the pylon and the deck girder forming a rigid node. The top cables transmit significant forces from the deck to the abutments where the deck girder has to be fixed. These main cables are anchored to the deck almost next to each other, such to avoid high stress concentration in the deck at mid-span due to differential cable forces.

The problem of stabilizing a cable-stayed structure with several spans in the longitudinal direction had to be addressed. Rather than linking the adjacent pylon tips by a horizontal cable or cross bracing two neighboring pylons, stabilization may also be obtained by a series of rigid pylon-to-deck girder systems. This is an efficient solution for bridges with rather short pylons and relatively short overall length such as the one in the present case.

The "nose-like" detail added to each edge of the deck improves the aerodynamic behavior of the structure and also emphasizes the deck girder slenderness and the rigid cross formed by the pylon and the deck.

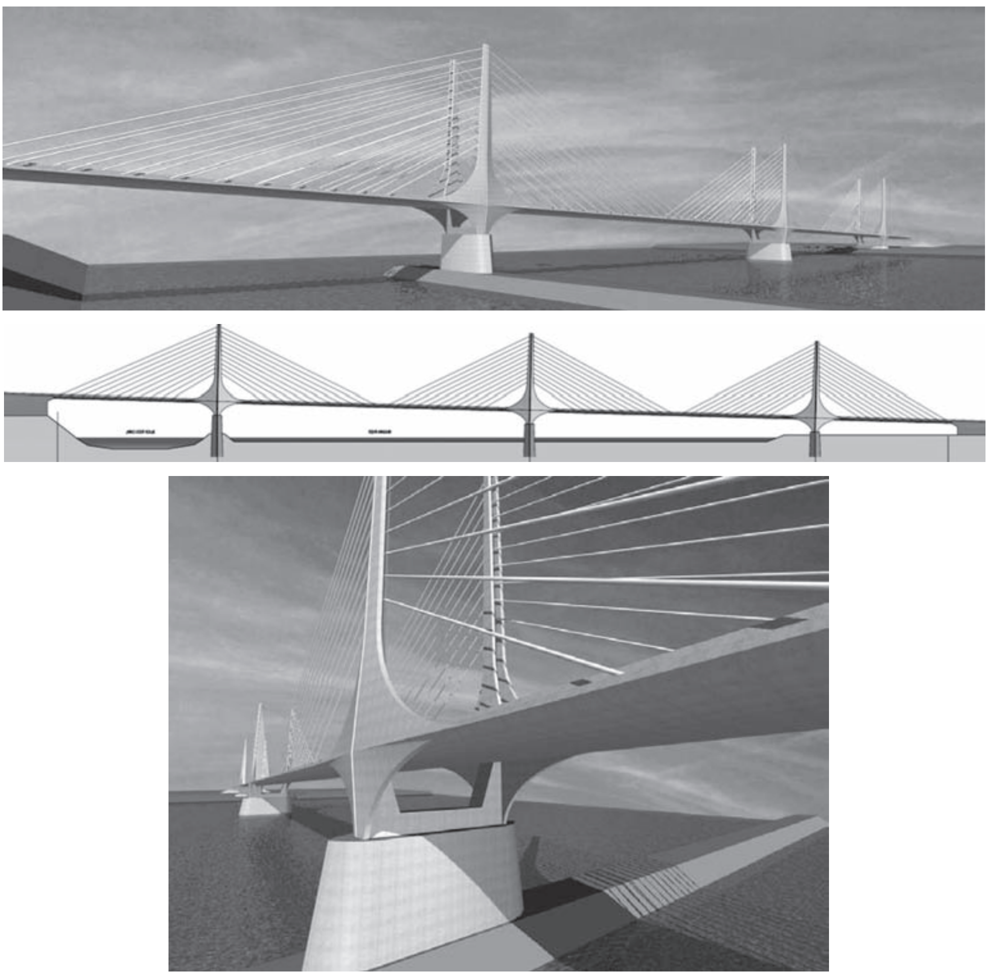


Figure 5. Peace Bridge—most recent design for a replacement bridge.

The final bridge design is now with the Figg Engineering Group. The decision to begin construction in 2010 has not yet been taken.

6 CONCLUSIONS

The four presented designs of landmark bridges express technical efficiency with their accent on slenderness and transparency. They emphasize the importance of understanding how the structural systems function.

A sound engineering concept is the solid basis for a far-reaching aesthetic quality and for finding simple yet elegant structures offering a great crossing experience while respecting the functional requirements and providing technical performance. Guided by the basics of mechanics and natural sciences, this approach continues to be very efficient and valuable, in particular nowadays, when architect-led bridge designs (often based on a metaphoric idea) have produced structures that are excessively expensive to build and maintain.

The presented bridge designs are the result of the exclusive engagement of Christian Menn in bridge engineering. They result from his more than 50 years of continuous experimentation in the conceptual structural and aesthetic design of bridges.

This paper shows that bridge design is founded on creativity. The art of structural engineering is characterized by innovation and imagination with the objective to improve the environment through structural art.

The art of structural engineering should be appreciated again and be given much more importance in the education of structural engineers.

ACKNOWLEDGMENTS

The author expresses his gratitude to Christian Menn for giving insight into his bridge designs over many years. As a student, the co-author of the extended version of Menn's book on concrete bridge design (Brühwiler and Menn, 2003), a colleague and a friend, the author has gotten and still gets preferential access to Menn's art of bridge engineering.

The detailed designs of the four bridge structures are performed by the Christian Menn Partners AG consisting of Christian Menn and three Swiss engineering companies (and their principal engineers), namely, Bänziger Partner AG (Dr. Werner Brändli), Dr. Deuring + Oehninger AG (Dr. Martin Deuring) and Walt + Galmarini AG (Carlo Galmarini). The author acknowledges their contribution to this paper in providing drawings, computer animations and detailed design information.

REFERENCES

Brühwiler, E., Menn, C. 2003. Stahlbetonbrücken (Reinforced concrete bridges). 3rd, updated and extended edition, Springer Editors Wien New York, 551 S (in German).

Brühwiler, E. 2007. Christian Menn's Brückenentwürfe in den USA (Christian Menn's bridge designs in the USA), *Beton- und Stahlbetonbau*, Vol. 102 (2007), No. 7, S. 477–483 (in German).

Menn, C. 1996. The Place of Aesthetics in Bridge Design, *Structural Engineering International*, Volume 6, No. 2, pp. 93–95.

Menn, C. 1998. Functional Shaping of Piers and Pylons, *Structural Engineering International*, Volume 8, No. 4, pp. 249–251.

Chapter 39

A study of the evolution of arch forms: Eiffel's Maria Pia Bridge and Ammann's Bayonne Bridge

A.P. Thrall & D.P. Billington
Princeton University, Princeton, New Jersey, USA

ABSTRACT: Gustave Eiffel and Othmar Ammann each designed great two-hinge arch bridges that have been named landmarks by the American Society of Civil Engineers. Eiffel's Maria Pia Bridge exhibits the crescent form with an aesthetic of lightness. Ammann's Bayonne Bridge is of the spandrel from and is influenced by an aesthetic of monumentality. Referring to archival research and technical analyses, this paper will show that each is a major work of structural art by adhering to the follow tenets: efficiency (minimum material consistent with satisfactory performance and assured safety), economy (competitive construction cost consistent with minimal maintenance requirements), and elegance (aesthetically striking consistent with efficiency and economy). Then it will present a quantitative comparison between the crescent and spandrel forms of two-hinge arch by optimizing each with respect to member cross-section. This study found the crescent form to be more efficient based on a lower self-weight after optimization.

1 INTRODUCTION

1.1 *Two great, two-hinge arches: The crescent and spandrel forms*

This paper seeks to present some results from detailed studies of two major metal bridges, neither of which have been treated in detail in major structural engineering journals prior to 2008.

Gustave Eiffel (1832–1923) and his company G. Eiffel & Cie, designed and built the Maria Pia Bridge over the Douro River in Porto, Portugal (completed in 1877) (Figure 1). It was the first of Eiffel's great arch bridges. This arch is of the crescent form, meaning that it is deep at the crown and narrows to nearly points at its hinges. Its aesthetic can be judged as light. It is a two-hinge arch bridge with a span of 160 meters (525 ft) and a rise of 42.935 meters (141 ft) (Seyrig 1878). Construction began in January 1876, ended on October 31, 1877, and on November 4, 1877, King Luiz I and Queen Maria Pia inaugurated it. They honored Eiffel by naming him Commander of the Order of the Conception (Eiffel 1879).

Othmar Ammann (1879–1965) designed and supervised the construction of the Bayonne Bridge spanning over the Kill van Kull River between Bayonne, NJ and Staten Island, NY (completed in 1931) (Figure 1). This arch is an example of a spandrel form in which the arch is deepest at the hinges and becomes progressively less deep toward the crown. The deepening at the hinges suggests an image of a fixed arch and aims toward an aesthetic of heaviness and monumentality. The Bayonne Bridge is a two-hinge arch with a span of 504 m (1652 ft) and a rise of 81 m (266 ft) (Ammann 1928). The Port Authority of New York and New Jersey began planning for the bridge in the 1920s and received authorization from Congress in 1925 (Ammann 1930). Ultimately, it came in $3 million under budget (compared to an estimated initial cost of 16 million 1931 dollars) and was opened ahead of schedule with a dedication ceremony on November 14, 1931 (Anonymous 1981, Anonymous 1931).

1.2 *Definition of structural art*

A structure can be considered structural art if it can be shown to be efficient (minimum material consistent with satisfactory performance and assured safety), economic (competitive construction

Figure 1. Maria Pia Bridge (left) and Bayonne Bridge (right). Photograph of Maria Pia Bridge courtesy of Kyrsten Bréa. Photograph of Bayonne Bridge from Library of Congress—Historic American Engineering Record (Janberg 2009).

cost consistent with minimal maintenance requirements), and elegant (aesthetically striking consistent with efficiency and economy). These qualifications act as scholarly guidelines through which one can frame a discussion of a structure. However, they should not serve as black and white boundaries for structural art. Rather, structural art is a continuum on which designs can be judged, with major works following all three guidelines. In this regard, all bridges designed with aesthetic motivation and with an efficient form can be structural art. The degree to which they are economic and elegant makes certain bridges stand out as major works. Christian Menn wrote that "The optimization of economy and elegance requires more than the craftsmanship component of engineering. It requires creativity, fantasy, and sensitivity to visual form. These talents collectively constitute the art of engineering" (Menn 1996). Menn thus identified the key to great structural art in the engineer's play between economy and elegance within a discipline of efficient form.

1.3 *Outline*

The goal of this paper is first to show that each two-hinge arch is a major work of structural art. Each is deemed efficient based on the ratio of the stress in each member to the allowable stress, thereby illustrating the degree to which each designer minimized the use of material. Economy will be considered through a design competition in the case of the Maria Pia Bridge and through a discussion of final costs for the Bayonne Bridge. The aesthetics of each bridge are not quantifiable, but the fact that the Bayonne Bridge was named a National Historic Civil Engineering Landmark and that the Maria Pia Bridge was named an International Historic Civil Engineering Landmark by the American Society of Civil Engineers (ASCE) testifies to their elegance (ASCE 2009). The paper ends with a more general quantitative comparison of the crescent and spandrel arch forms. The results indicate the behavior of each type of arch and provide some sense of the relative efficiency of these two forms.

2 EIFFEL'S MARIA PIA BRIDGE

2.1 *Technical analysis*

Seyrig wrote a paper entitled *Le pont sur le Douro à Porto*, in which he described his design, method of analysis, and the results of his calculations (Seyrig 1878). This document provided the structural plans and served as the primary resource for the analysis referred to here and detailed elsewhere (Thrall 2008). Seyrig used the force method of analysis to find a horizontal reaction of 341,000 kg (752 kips) under the permanent dead load, which includes the self-weight of the arch, the deck, and the pillars. We reproduced these calculations and found a result that agreed.

We then performed full finite element analyses to consider the efficiency of the crescent form. These analyses first confirmed the horizontal reactions found by Seyrig, thereby verifying his results and validating our models. These analyses found a minimal amount of bending in the arch under dead load. Figure 2 shows the shape of the arch (the solid line) where the horizontal axis is the horizontal coordinate and the vertical axis is the vertical coordinate of each node. The dotted line shows the moment (divided by 100 for scaling) found through the finite element analysis and added to the vertical coordinate of the arch. Therefore when the moment is zero, the plots exactly superimpose, a positive moment is above the arch line and a negative moment is below. The moments are relatively small and follow the basic form of Eiffel's arch. A sample calculation of the stress due to bending shows that these moments do not pose a problem. The bending stress at the outermost fiber at the node with the highest moment (circled in Figure 2), is 406,000 kg/m^2 (0.577 ksi). In his paper on the Garabit Viaduct, Eiffel takes the allowable stress of iron to be 6,000,000 kg/m^2 (8.53 ksi) (Eiffel 1889). Therefore the stress due to bending even at the largest point of moment is effectively negligible. Our calculations for the axial force in each member allowed us to also make a judgment on the efficiency. Using derived member sizes and axial forces calculated by a finite element program, this study determined the axial stress in each member of the upper and lower chords under dead load and a uniformly distributed live load. These calculations only included the arch and therefore did not consider the deck-arch interaction. A numerical efficiency can be found by dividing this stress by the allowable stress (where a value of 1 indicates ideal efficiency). Under dead load and a uniformly distributed live load, the average efficiency of the upper chord was 0.87 and the lower chord was 0.77. These results suggest that the members are highly efficient consistent with safety. The efficiency under live load and the minimal bending under dead load shows this bridge to be of an efficient form.

2.2 *Maria Pia as a great work of structural art*

The technical analyses described here show that the Maria Pia Bridge is efficient both through minimal bending and by considering axial stresses in each member. Within this discipline of

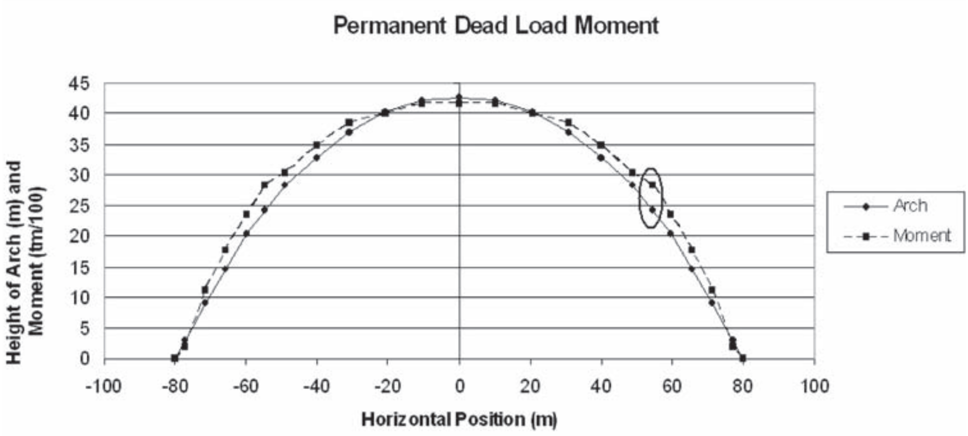


Figure 2. Permanent Dead Load Moment for the Maria Pia Bridge. The solid line shows the shape of the arch where the X axis is the horizontal coordinate and the Y axis is the vertical coordinate of each node. The dotted line shows the moment found using a finite element anlaysis (divided by 100 for scaling) and added to the vertical coordinate of the arch. The circle indicates the point of highest moment.

efficiency, the bridge also exhibits a play of economy and elegance. This structure can be deemed economic based on the design competition that led to its construction. The Royal Portuguese Railroad Company opened a competition for the Maria Pia site on May 1, 1875. Four companies submitted six proposals for the site, including Eiffel's design. Eiffel's proposal was 31 percent less in cost (per linear foot) than the next lowest design and was ultimately chosen (Seyrig 1878). In 1990, the ASCE awarded this bridge the distinction of becoming an International Historic Civil Engineering Landmark—a testament to its elegance. This is Eiffel's only bridge to receive this award (ASCE 2009). Furthermore the engineers at G. Eiffel & Cie clearly expressed their aesthetic intentions for the bridge. As documented and translated by Bréa, Seyrig discussed the play of economy and elegance in his paper on the Maria Pia Bridge, stressing that aesthetics are as important as economy (Bréa 2005). Overall, the bridge exudes a sense of lightness and elegance within a discipline of efficiency and economy.

3 AMMANN'S BAYONNE BRIDGE

3.1 *Technical analysis*

A full technical study of the Bayonne Bridge has been published by the authors in 2008 (Thrall & Billington 2008). Here, we will briefly summarize the results to make a case that the Bayonne Bridge is a work of structural art. In this research, we reproduced Ammann's original calculations and verified these against a full finite element analysis. We found members to be efficient under the maximum design load. A study of one stage of construction found low stresses in members, revealing a consideration of safety during erection. These results suggest an efficiency within Ammann's choice of spandrel form.

3.2 *Bayonne Bridge as a great work of structural art*

The technical analysis referred to above shows that Ammann designed the Bayonne Bridge to be efficient. From the minimal use of material in his design and the fact that the bridge came in well under budget, we can argue that the bridge is economic. Ammann's aesthetic motivations in bridge design were revealed in his 1918 paper on the Hell Gate Bridge. He wrote that, "An elaborate stress sheet, worked out on a purely economic and scientific basis, does not make a great bridge. It is only with a broad sense for beauty and harmony, coupled with wide experience in the scientific and technical field, that a monumental bridge can be created" (Ammann 1918). This paper also indicates Ammann's preference for the spandrel form for its aesthetic of monumentality. The Bayonne Bridge received the first award for the most beautiful steel bridge from the American Institute of Steel Construction in 1931 and was awarded the distinction of being named a National Historic Civil Engineering Landmark by the ASCE in 1985 (Anonymous 1981, ASCE 2009). Such motivation by the designer and these awards start to indicate the elegance of the bridge. Based on this evidence of efficiency, economy, and elegance, we can conclude that the Bayonne Bridge is also a major work of structural art.

4 A COMPARISON OF THE CRESCENT AND SPANDREL FORMS

4.1 *Early comparisons of forms*

Ammann grappled with the difference between the crescent and spandrel forms in his 1918 paper on the Hell Gate Bridge. His sketch (Figure 3) illustrates the two different forms situated at the Hell Gate site (Amman 1918). See Thrall & Billington (2008) for a full discussion of the choice of form related to the Hell Gate Bridge and how this led to Ammann's choice of the spandrel form for the Bayonne Bridge.

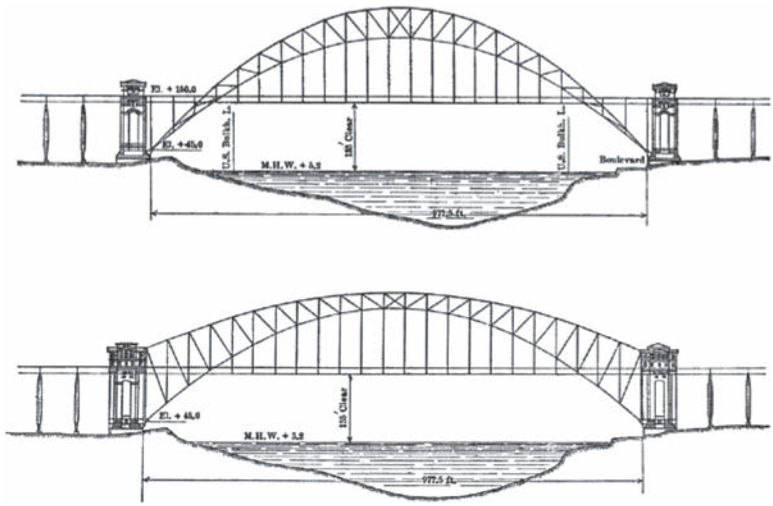


Figure 3. Sketches of the crescent form (top) and spandrel form (bottom) as they appear in Ammann's paper on the Hell Gate Bridge (Ammann 1918).

4.2 *Force distribution comparison*

To consider the difference in the two forms of two-hinge arches, we first performed a preliminary study using two finite element models to show the difference in the distribution of forces in the crescent and spandrel forms. The crescent model is the same as that used in the analysis of the Maria Pia Bridge. The spandrel model implemented the same member sizes and geometry for the lower chord. The geometry of the upper chord followed a parabola based on three points: the crown (the same depth as crescent model) and the two ends (scaled from Figure 3). Additional web members were added at the hinges to complete the model. Each two-dimensional model assumed pin connections of all members and used pin supports for each hinge.

When analyzed under their own self-weight, the horizontal and vertical reactions of each form differ (see Table 1). The "advantage of the crescent" refers to the difference between the reactions of each form divided by the magnitude of the crescent value. The horizontal reactions were essentially the same for both forms, with just a 2.6 percent advantage for the crescent form. The vertical reaction for the crescent form showed a 9 percent advantage which can be attributed to the material necessary to have the two additional vertical members and the 4 extra diagonal members at the hinges. This suggests that additional material would be necessary for the spandrel form, and therefore reduce its economy. However, this is not a conclusive study since both models used member sizes designed for the crescent form. Presumably when designing the spandrel form, an engineer would size the members according to the force in each and therefore make a more economic design. This will be further discussed in the next section.

Table 1. Reaction comparison.

Form	Horizontal reaction (kg (kips))	Vertical reaction (kg (kips))
Spandrel	163590 (361)	223856 (494)
Crescent	159502 (352)	205428 (453)
Advantage of crescent	2.6%	9.0%

A structure can be said to be efficient if all members carry an equal stress that is close to the allowable stress. To judge the efficiency of the forms, it is helpful to consider a schematic of the forces in each member (Figure 4). The width of the bars of each member indicate the relative magnitude of the axial force under self-weight. The lower and upper chords of the crescent form carry similar magnitudes of forces uniformly throughout. Since the upper and lower chords have the largest cross-sections, this suggests an efficient structure. The web members carry very little force according to their smaller cross-sectional areas. However, the diagram of the spandrel form shows very little force in the upper chord towards the hinges. Instead, there is much greater force in the diagonals and the verticals here. This suggests that maximum use is not being made of the upper chord by this form and therefore it is not as efficient as the crescent form.

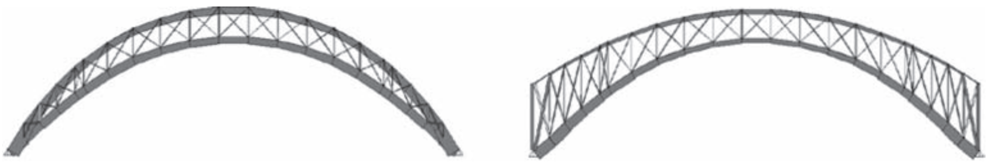


Figure 4. Schematics of the axial forces in each member of the crescent model (left) and the spandrel model (right). Thicker bars indicate higher axial forces.

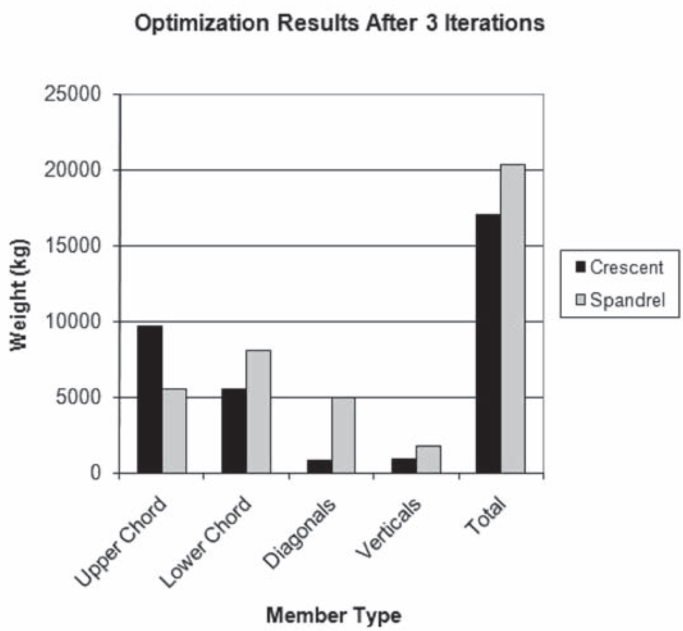


Figure 5. Graph of the self-weight of member types for the crescent and spandrel forms after 3 iterations of the optimization process.

4.3 *Simplified optimization study*

An optimization process presented in this section provides a means of quantitatively comparing the crescent and spandrel forms under self-weight and an arbitrary uniform load. This study began by considering the two finite element models described in the previous section. All members in both models started with the same cross-sectional area arbitrarily chosen as .1 m^2 (155 in^2). We applied an arbitrary point load of 10000 kg (22 kip) at each of the lower chord nodes. This was not an ideal, uniform load since the nodes are not evenly spaced, but it was sufficient for the purpose of this study. This study found the axial forces in each member of each truss. We used this result and the optimal stress (the allowable stress of 6,000,000 kg/m^2 (8.53 ksi)) to calculate new estimates of each member's cross-section. We then re-ran the analysis and preformed this optimization process for three iterations. Successive iterations would eventually lead to each member's axial stress approaching the allowable stress. However, three iterations proved to show qualitative trends. See Figure 5 for a graphical representation of the self-weight of the different element types for each form after 3 iterations. With each iteration, this study found that the crescent form increasingly improved over the spandrel form in terms of the total self-weight of the arch. This can be attributed largely to the sizing of the web members. In the third iteration, the upper and lower chords of the crescent form, when summed together, weighed 1.1 times more than that of the spandrel form. However, the web members (meaning the diagonals and verticals) of the spandrel form weighed 3.8 times more than in the crescent form. Part of the difference in the weight of the web members can be attributed to the two additional vertical members and the additional diagonals. Overall, however, it indicates a fundamental difference in the way that the spandrel form carries load.

With further iterations, this paper predicts that each form would converge to optimized member sizes, but the crescent arch would have a smaller self-weight than the spandrel form. This would result in a fundamental savings in materials. This optimization study also reveals some results related to the dominant members for each form. The crescent forms' upper chord is dominant, while the spandrel form features its lower chord. Also, the spandrel form features very massive web members as compared to both the other features in this form and to the web members of the crescent form.

5 CONCLUSIONS

Within the discipline of efficiency, both the Maria Pia and the Bayonne Bridge exhibit a play of economy and elegance. Two studies comparing the crescent form and the spandrel form reveal that the crescent form is a more efficient form for a two-hinge arch. Despite the advantages of the crescent form, it was not adopted for most long-span two-hinge arch designs of the 20th century. For a review of 20th century arch bridge designs see Thrall (2008). The future of long-span arch trusses is not clear. As engineers push the envelope of spans, they have moved away from this type. For example, the Lupu Birdge (completed in 2003, 550 m (1804 ft)), currently the longest spanning arch, is comprised of steel box sections, rather than a truss (Lin et al 2004). This paper challenges current designers to consider ways to re-invent the arch truss so that this very efficient form is more prominent in 21st century design.

ACKNOWLEDGMENTS

This material is based upon work supported under a National Science Foundation Graduate Research Fellowship. The authors are also grateful for financial support from the National Science Foundation Grant No. 0308549.

REFERENCES

American Society of Civil Engineers. 2009. *Designated Historic Civil Engineering Landmarks Website*. http://www.asce.org/history/landmark/projects.cfm?menu=loc

Ammann, O.H. 1918. The Hell Gate Arch Bridge and Approaches of the New York Connecting Railroad Over the East River in New York City. *Transactions of the American Society of Civil Engineers*. 82: 863, 871–882.

Ammann, O.H. 1928. *Kill van Kull Bridge (Structural Plans)*. New York: The Port of New York Authority.

Ammann, O.H. 1930. *First Progress Report on Kill van Kull Bridge*. New York: The Port of New York Authority. p. 11–12.
Anonymous. 1931. *Address Delivered at Dedication: Bayonne Bridge over the Kill van Kull between Port Richmond, Staten Island, New York and Bayonne, New Jersey.* New York: The Port of New York Authority.
Anonymous, 1981. Press Release. *The Port Authority of New York and New Jersey News*. Nov. 16, 1981: 3.
Bréa, K.L. 2005. *The Ponte Maria Pia: Structural Art Achieved Through Purely Technical Engineering*. Princeton: BSE Thesis, Princeton University. p. 80.
Eiffel, G. 1879. *Notice sur le pont du Douro, à Porto*. Clichy: Imprimerie administrative et des chemins de fer de Paul Dupont. p. 14.
Eiffel, G. 1889. *Mémoire presenté à l'appui du projet définitif du viaduc de Garabit*. Paris: Librarie Polytechnique, Baudry et Cie. Republished: 1988. Spain: Instituto Tecnico de Materials y Construcciones. p. 25.
Janberg, N. 2009. *Structurae Website.* http://en.structurae.de. Source: Library of Congress—Historic American Engineering Record, NJ 9-BAYO, 1–14.
Lin, Y., Zhang, Z., Ma, B., & Zhou, L. 2004. Lupu Arch Bridge, Shanghai. *Structural Engineering International*. 14(1): 24–26.
Menn, C. 1996. The Place of Aesthetics in Bridge Design. *Structural Engineering International*. 6(2): 93.
Seyrig, T. 1878. *Le pont sur le Douro à Porto.* Paris: E. Capiomont and V. Renault.
Thrall, A.P. & Billington, D.P. 2008. Bayonne Bridge: The Work of Othmar Ammann, Master Builder. *ASCE Journal of Bridge Engineering*. 13(6): 635–643.
Thrall, A.P. 2008. *A Comparison of the Work of Gustave Eiffel and Othmar Ammann: The Maria Pia and Bayonne Bridges.* Princeton: MSE Thesis, Princeton University.

Chapter 40

Design and construction of the Third Millennium Bridge over the Ebro River in Zaragoza, Spain

J.J. Arenas de Pablo, G. Capellán Miguel, H. Beade Pereda & J. Martínez Aparicio
Arenas & Associates, Santander, Spain

ABSTRACT: The way from conception to construction of the Third Millennium Bridge, a bowstring arch entirely made of high strength, self-compacting white concrete and located in an urban environment, is described in this paper. The approaching to the city needs, the type and material choosing, the design, the structural solution, the specific studies developed and the bridge construction are explained for showing how this outstanding and challenging structure leads bridge design and engineering one step beyond.

1 INTRODUCTION

The Third Millennium Bridge (Figure 1) is a white concrete bowstring arch, with a 43 m typical deck width and a main span of 216 m, becoming the bridge of this type with the greatest span built to date using this material. The design of a central arch with final open A frames, where the main arch divides itself into two inclined legs linked by a crossbeam, is an evolution of the 168 m span and 30 m wide steel made Barqueta Bridge, designed by the same author and built for the Universal Exposition that took place in Sevilla in 1992 (Arenas & Pantaleón 1992a, b). This bridge can be considered a test model for the design and construction of the much wider and longer post-tensioned high strength concrete made Third Millennium Bridge (Figure 2).

This new challenging bridge is destined to be a new engineering feat due to many factors:

– Its urban function and location in the city of Zaragoza, over the Ebro River (which gives name to the Iberian Peninsula), closing the third ring road of the city, giving the left bank of the city fast access to the recently built high speed train and bus station, and becoming the main road entrance to the International Exposition of 2008, celebrated in Zaragoza between June and September.
– Its dimensions: with 270 m total length, 216 m main span, 36 m height over the deck, and 43 m typical deck width, it becomes the largest concrete bowstring arch bridge built to date.
– The quality of its aesthetical design, becoming an icon for the city and the Expo 2008 event.
– The use of a new material, the high strength, self-compacting white concrete, used to build almost the entire bridge, and with theoretical compression resistance of 75 N/mm^2 for the arch and 60 N/mm^2 for the deck, but reaching real values that exceed 85 N/mm^2 due to early pre-stressing needs during construction.
– The quality of its structural design, leading its slenderness to the limit (Figure 1) and using a great number of external and internal post-tensioning tendons disposed in a highly complex way to reduce the thickness of the different concrete elements to the minimum, enabling the slender shell arch to hold up the bridge own weight.
– The complex erection procedures needed for its construction, including the launching from one bank of a 34 m wide and up to 200,000 kN in weight deck with longitudinal curvature and a curved bottom cross section, and the introduction of a 120,000 kN horizontal load in the crown of the arch using hydraulic jacks for putting its final forces into play.

Dr. J.J. Arenas has designed manifold arch bridges during his career, becoming a reference author in this type of structures due to his treatment of forms and his technological contributions

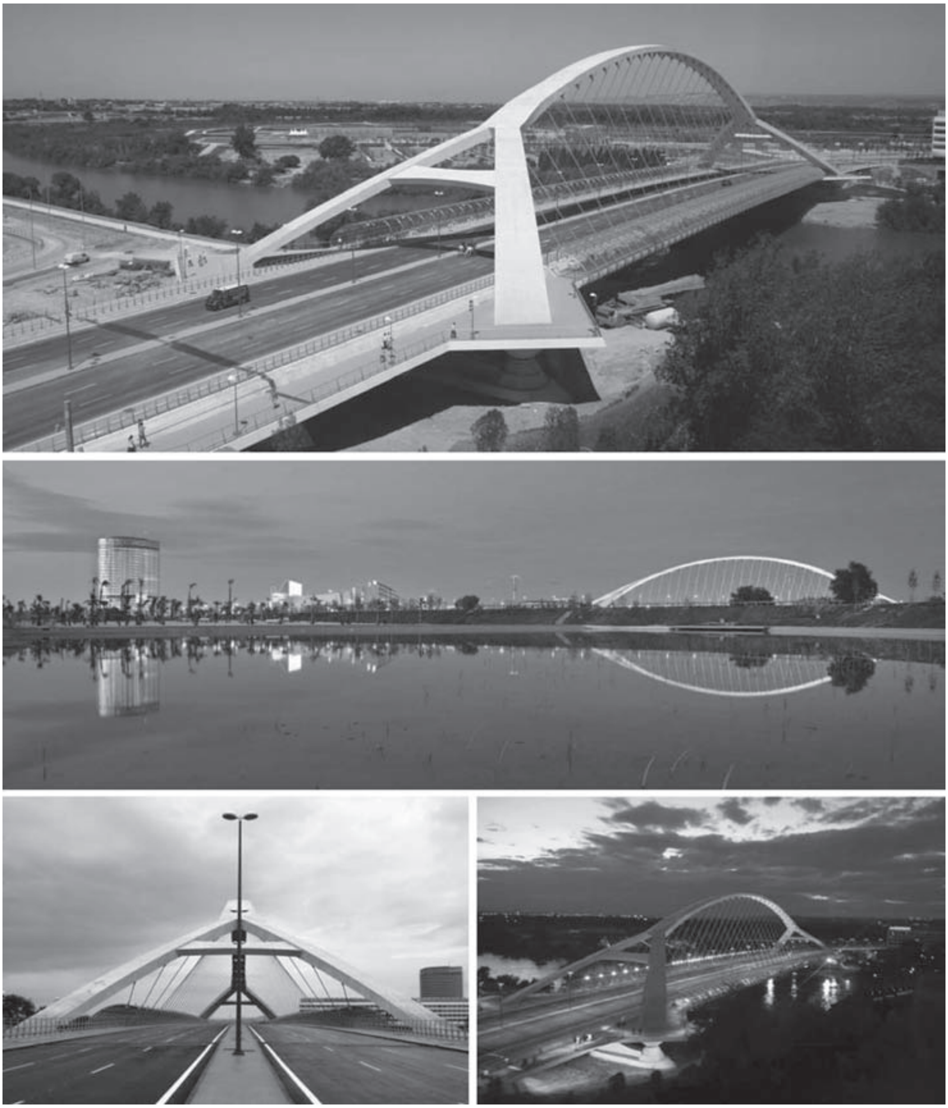


Figure 1. Different views of the Third Millennium Bridge.

Figure 2. Barqueta bridge in Sevilla (left) and Third Millennium Bridge in Zaragoza (right).

Figure 3. From top to bottom and left to right: Barqueta Bridge (Sevilla, Spain), Oblatas Bridge (Pamplona, Spain), La Regenta Bridge (Asturias, Spain), Barrial Bridge (Madrid, Spain), Morlans Bridge (San Sebastián, Spain), Hoznayo-Villaverde Bridges (Cantabria, Spain), 5th Logroño Bridge (La Rioja, Spain), Congressi Bridge (Roma, Italy) and Coega Bridge (Port Elizabeth, South Africa).

(Arenas 1998, 2002, 2004, Arenas & Siviero 2002). The Third Millennium Bridge, with its size and complexity under simple appearance, is the greatest and most ambitious of the arch bridges projected by Arenas & Associates, and a consequence of all the work done and knowledge achieved before for the designing and construction of other arch and non-arch bridges.

Six built arch bridges of the catalogue of Arenas & Associates and three competition winner designs pending to be built are shown in Figure 3.

2 ZARAGOZA, THE EBRO RIVER AND THE THIRD MILLENNIUM BRIDGE

The city of Zaragoza is the fifth of Spain in number of inhabitants (nearly 700,000) and is located halfway between the main cities of Madrid and Barcelona. The Ebro River, which divides de city into two, is the one with highest water flow in the Iberian Peninsula. The right bank of the river hosts the historic city center and the main part of the city, but the left bank has suffered a great population grown in the last twenty years, not accompanied until now with the construction of new bridges in its western side, whose inhabitants needed to necessarily move towards the city center to cross the river.

In recent years, two events changed the life of many of the inhabitants of Zaragoza: the arrival of the high speed train, whose station was built in the western side of the right bank of the city, and its election by the Bureau International des Expositions (BIE) for the organization of the International Exposition of 2008 (Expo2008). The area chosen for the celebration of this event is located in the western side of the left bank in a meander area of the river.

The Third Millennium Bridge becomes a basic urban piece in this context, serving as a great entrance door to receive the Ebro River waters into the city as it solves the important need of a western river crossing, closing the third ring road of the city, giving its left bank fast access to the recently built high speed train (and bus) station and becoming the main road entrance to the Expo2008 site (Russell 2008, Aguiló 2008, Arenas et al. 2008a).

3 THE DESIGN OF THE BRIDGE

3.1 *Global design constraints*

The bridge emplacement has a particular characteristic which makes it noticeably different from those of the other Zaragoza river crossings (Arenas et al. 2008b, c, d). It is located in a meander area, with pronounced curvature and a complex hydraulic response and whose banks inundate with low water flows, at least once a year. Trying not to increase these flood problems of the site and to avoid river bed scour in a so sensitive area, we decided to plan the river jump in an only span, with about 220 m to save.

Thinking on giving an easy access to the bridge, we desired a deck as close as possible to the banks, avoiding large approach embankments which would divide the river sides. At the same time, when over the river and as a requirement of the hydrographical authorities, it is necessary to leave sufficient space for the water flows predicted for a 1 in 500 year flood. These two conditions become constraints on the depth of the deck both from above and below, setting the basis for a design with its main structure disposed over a slender deck (Figure 4).

3.2 *Choosing the bridge type*

Several options for the bridge type were considered, being a cable-stayed bridge the main contender of the chosen bowstring arch (the poor soil conditions at the site turn impossible to build an arch if its extremes are not tied by the deck). Both solutions are compared in Figure 5.

A cable-stayed bridge needs side spans (with an optimum length of about the 40% of the main one) to save the same main distance as a bowstring arch bridge, resulting a larger structure in terms of overall length. It is possible to suspend only the main span anchoring the back stays to retaining blocks, but this solution requires a space that, in our case, is not available on the left bank.

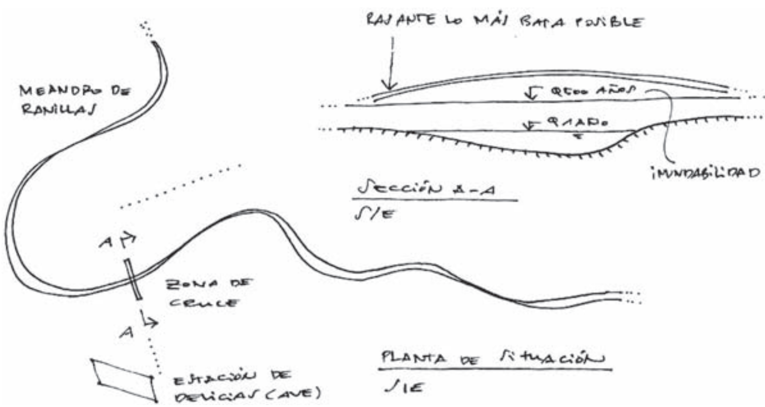


Figure 4. Global constraints for the preliminary design of the crossing.

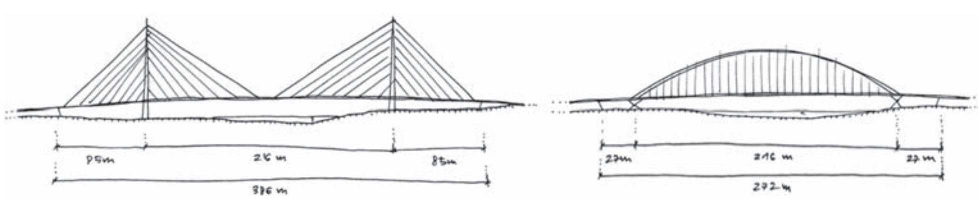


Figure 5. Cable-stayed Bridge (left) versus bowstring arch bridge (right), saving the same main span.

The cable-stayed alternative includes high towers to have good structural response and therefore a slender deck. In our case, we would need towers with a height up to 85 m to save a 216 m clear span with the same deck depth as a bowstring arch with a 36 m maximum height over the deck (16% of the main span). The cable-stayed solution would create an excessive visual impact in the classical landscape of a city whose main landmark, the 17th century Pilar Basilica, has for towers with a maximum height of 80 m.

The chosen bowstring arch bridge gives response to the exposed constraints in a compact way, with single span and a reduced height.

3.3 *Choosing the materials*

A number of technical, aesthetical and economical reasons moved us towards the selection of the white concrete as the main material for the construction of the bridge. Almost the entire bridge was designed in concrete, excepting the side pedestrian areas (formed by a wooden deck over steel ribs and covered by glass over a stainless structure) and, obviously, the hanger cables and their anchorages (Figure 6).

Figure 6. From left to right and from top to bottom, left bank abutement, detail of the transition between pure arch and inclined legs, one of the side glazed balconies and perspective of the three dimensional space inside the bridge.

One of this reasons is the need to improve the dynamic behavior of the structure and avoid vibrations due to traffic and wind (Zaragoza is windy city and its main wind direction is almost perfectly perpendicular to the bridge axis). These actions become important with the distance we need to span and especially when, as in our case, a slender structure is desired. The designed bridge would have a much worse dynamical response if built in steel.

If we continue talking about aesthetics, concrete can create a seamless monolithic structure, a continuous object with shapes connected without cut planes, an object aspiring to compose a great white marble sculpture over the river. Concrete is a noble material with wonderful ageing if a concerned execution and a social respect for its surfaces exist. The white color increases this nobility and its visual quality.

We must also acknowledge that the emotional connection of J.J. Arenas with the dry and earthy landscapes of the region, just 75 km from his birth place and where he spent part of his life, had something to do with the decision of material choosing.

When concrete is chosen as the main material, the self-weight of the bridge becomes de greatest problem for its structure (in fact for the bridge itself as every element seen is structurally needed and its design is an expression of its internal forces). The use of high strength concrete becomes a need because the thickness of every structural element must be as reduced as possible, especially of those which compose the deck, to enable the bridge support itself.

Looking for this reduction of self-weight, the high quantity of complexly arranged reinforcement bars, internal post-tensioning steel ducts and external post-tensioning deviators needed, do not leave free space to use vibrators to accommodate the fresh concrete into the false work. The using of self-compacting concrete for the entire structure guarantees an accurate setting up.

All these conditions required the challenge to develop a new material to build this bridge, a high strength and self-compacting white concrete. Enough tests were done in detailed design phase, to prove the objective was attainable, reaching 92 N/mm^2 of compression resistance and an instant elastic modulus of 41,250 N/mm^2.

3.4 *The conceived bridge*

As we have seen, the designed bridge is a high strength, self-compacting, white concrete bowstring arch with a main span of 216 m and 43 m of typical deck width (Figure 7).

Two secondary side 27 m spans complete a global length of 270 m. These additional spans, designed as a monolithic prolongation of the deck, are not necessary for the bowstring working, but enable to complete the hydraulic section for great floods and contribute on the yet mentioned desired deck slenderness, as it works as a continuous beam with two near supports in its extremes. Triangular slabs, where the deck widens further to 68 m, were designed as the linking element between the inclined legs and the deck (see Figures 7–8). The deck sides are suspended from the arch by 32 pairs of hanger cables, arranged in a 6 m sequence (Arenas et al. 2008e).

The arch saves the central 144 meters with a constant hexagonal solid cross section of 5.40 m width, a minimum depth of 1.20 m at its sides, and a maximum of 1.80 m at the crown. Then, it divides itself into two inclined legs linked by a crossbeam each side. This legs save 36 m (with a length of 43 m) and their section varies from a width of 6.00 m and a depth of 1.10 m at the base to a width of 3.80 m and a depth of 1.74 m. The complex transition node between arch and legs is solved with triangular plane or curved surfaces (Figure 6). The cross beam between the inclined legs, with a variable width between 4.00 m and 2.70 m and a constant 1.00 m depth, is the only post-tensioned element beyond the deck, and carries out the function of reducing the buckling length of the inclined legs and thus enhance the structural stability.

The deck carries six traffic lanes (three in each direction) and four bicycle lanes (two in each direction, on each side of the traffic lanes). Pedestrian access is provided on each side, outside the cable planes an in a 0.60 m lower level than the traffic one. Pedestrians have a wind shielding running along the main span, to improve comfort for walkers in windy conditions. The shielding design consists of a stainless steel structure covered by glass pieces, creating a continuous glazed balcony (bottom left picture in Figure 6).

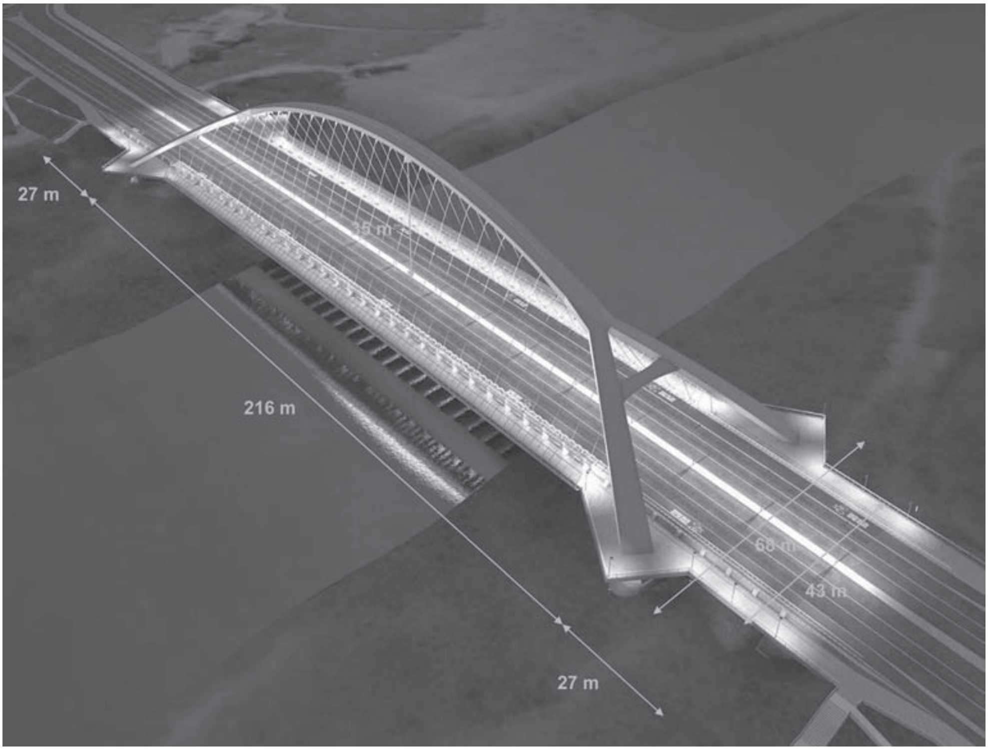


Figure 7. Main dimensions of the bridge.

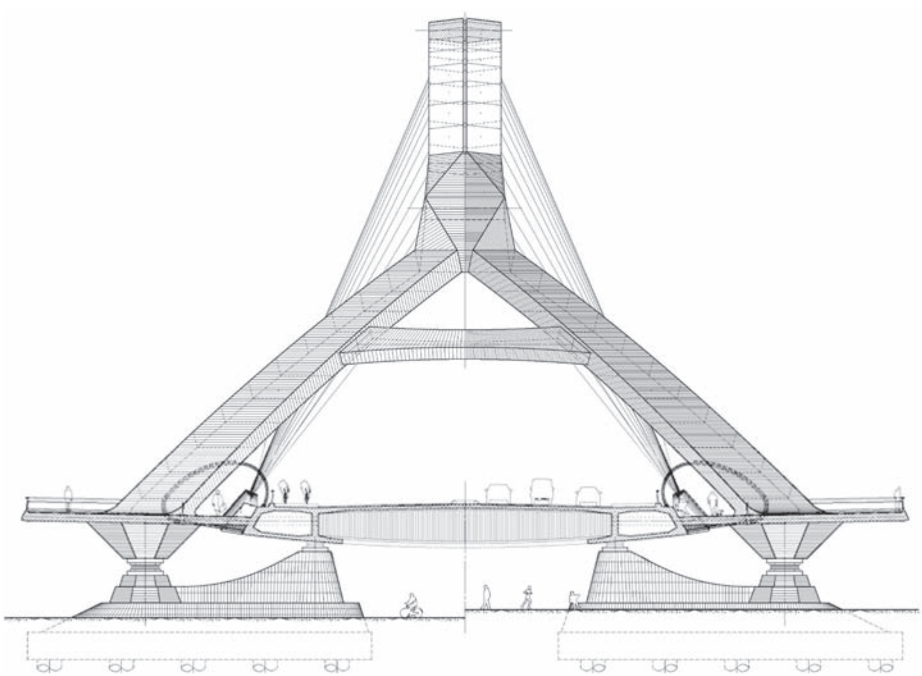


Figure 8. Bridge cross section from side spans, showing its main elements.

The deck has a main post-tensioned concrete body with a width of 33 m and a variable depth from 2.00 m to 3.20 m with a curved lower face (Figure 8), carrying the traffic and bicycle lanes. This shape response to the transverse flexion moments derived from its extraordinary width. A more common and less aesthetically cared design would have divided this width into two, building two parallel bridges. The deck cross section incorporates two hollow boxes (exceptionally solid in the support sections), one each side, transversely linked with asymmetric double T section diaphragms with continuity inside the boxes. The top flange of these diaphragms is part of the global deck top slab and the bottom flange has variable width. They are spaced every 6 m and create wide, clear areas between them and the side boxes. The typical thickness of the different elements composing the deck is really reduced as the self weight, as yet mentioned, is the main load of the bridge: 0.50 m for hollow boxes webs and the diaphragm area inside them, 0.30 m for hollow boxes top and bottom slabs and diaphragm bottom flanges, and 0.24 m for diaphragms webs and top slab between side boxes.

As the stiffness of the bridge deck is much higher than the stiffness of the shell arch, the hanger cables pass the loads from the first to the second in a quite uniform distribution, even though they are applied in a variable way at the deck level, allowing the arch work as a funicular compression structure (Arenas 2001). It never has tensioned fibers, as the compression load is always located near the center of the section (with a maximum eccentricity of 0.20 m). In Figure 9 the global structural behavior of the bridge is shown.

The arch is always under a compression force Z1 (with a maximum value of 230,000 kN and average compression stress in service state of 29 N/mm^2), generating a tension force Z3 in the deck due to the bowstring working (maximum value of 230,000 kN). As concrete has low tension resistance, the deck needs pre-stressing to confront this force. A group of 14 centered external post-tensioning tendons are arranged inside each side hollow box for this purpose (8 of 31 strands of ø0,6" and 6 of 37 strands of ø0,6", introducing a total load of 91,650 kN each side), to be added to the internal post-tensioning tendons arranged in top and bottom hollow box

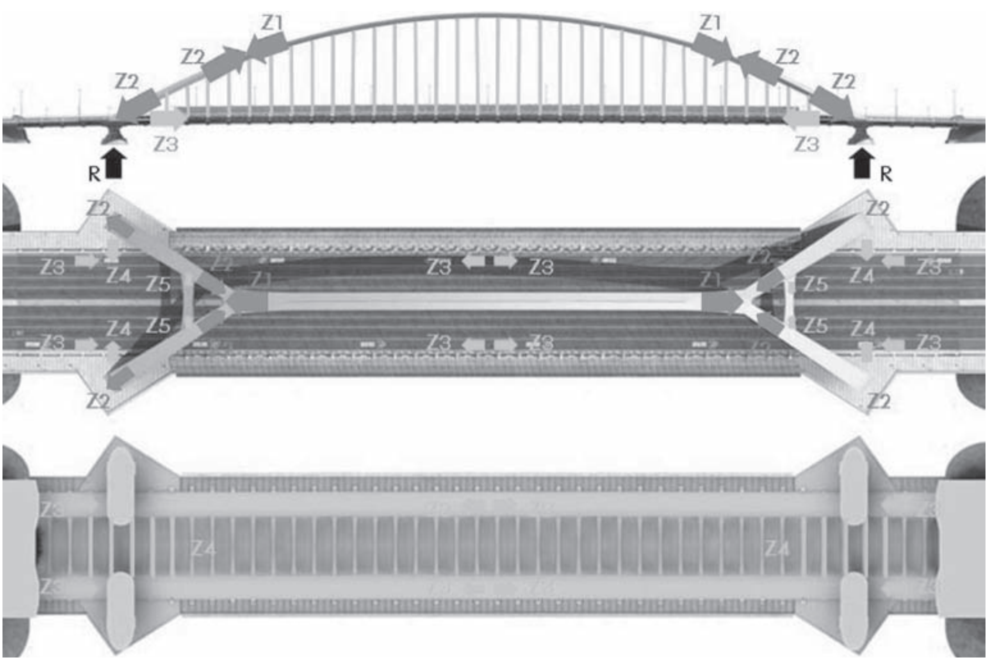


Figure 9. Force scheme of the bridge.

slabs (16 and 12 tendons of 19 strands of ø0,6" in top and bottom slab respectively, introducing a load of 103,740 kN). This internal post-tensioning has a basic role in the deck erection stage, during its launching, when it works as a continuous moving beam. After carrying out this function, it completes the deck tension capacity when the bridge works as a bowstring.

When the arch divides itself into the inclined legs, Z1 changes into a Z2 compression in each leg (maximum value of 140,000 kN). The component of Z2 parallel to the axis of the bridge causes the Z3 tension in the deck. This transition is produced in the 0.80 m deep triangular slabs that give connection between legs and deck, which is crossed by them. This complex elements (both for designing and construction) have a main post-tensioning in the direction of the plan projection of the legs (44 tendons of 19 strands of ø0,6", introducing a total load of 163,020 kN each slab) and a perpendicular secondary one (10 tendons of 12 strands of ø0,6", introducing a total load of 19,800 kN each slab). The component of Z2 perpendicular to the bridge axis introduces a transverse tension in the deck (maximum value of about 70,000 kN), Z4 in Figure 9, taken by the diaphragms located in this area, which have a straight post-tensioning, up to two top tendons of 31 strands of ø0,6" and two bottom tendons of 24 strands of ø0,6", in comparison with the common curved flexion post-tensioning of the typical diaphragms, with two tendons of 9 strands of ø0,6" going from top to bottom flange and 19 strands of ø0,6" along the bottom flange.

As a result of this complex post-tensioning arrangement, the deck is crossed in some sections by nine levels of tendons as displayed in Figure 10, where can also be seen the reduced free space in the slender concrete elements due to this reason, complicating the reinforcement and requiring a self-compacting concrete to guarantee an accurate setting up.

The Bridon supplied galvanized locked coil strand hanger cables, of 100 mm diameter and with a minimum breaking load of 10,100 kN, are anchored to the arch at their top end with a passive fork socket and steel plates introduced in the arch and linked to it with headed stud connectors and the own steel-concrete friction under compression. Their bottom end active anchorages, located at the lower surface of the deck after crossing it through especially disposed steel tubes, are solved with a cylindrical socket including a spherical washer.

The bridge has four main bearings under the inclined legs, supporting up to 82,500 kN of vertical reaction. This enormous value demanded the provider MK4 the fabrication of specifically designed pot bearings. They were tested in the Mageba Swiss laboratory up to their maximum load to eliminate adjustment descents and to determine the load-deformation behaviour of

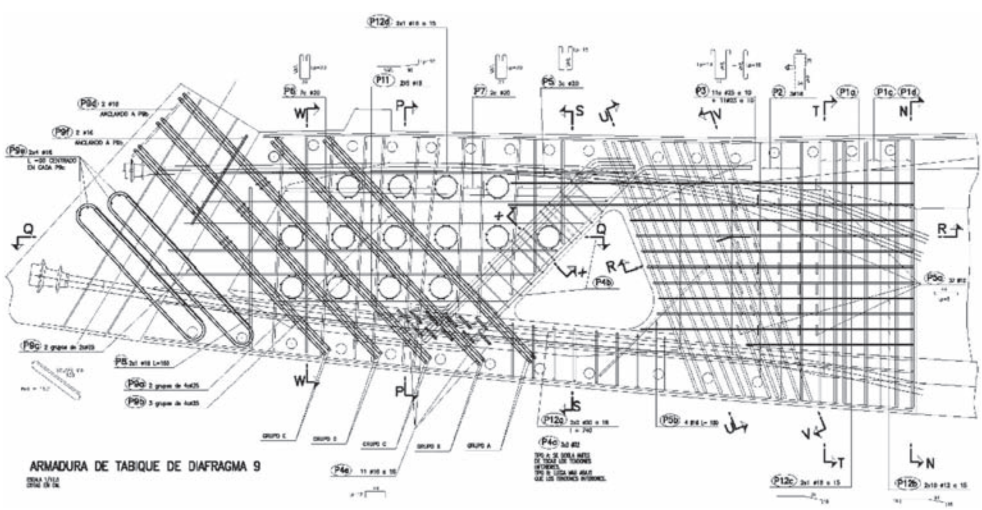


Figure 10. Reinforcement scheme of the area of diaphragms inside the side hollow boxes of the deck.

Figure 11. View from inside (left) and outside (right) the side glazed balconies.

such unusual bearings. As the distance between the two inclined legs of each end of the arch is 48 m, two additional pot bearings, designed for a maximum load of 28,500 kN were needed to support the deck in the same section as the previous ones (Figure 8). Inverted octagonal pyramid trunks work as force funnels between the wide base sections of the inclined legs and the relatively reduced section main pot bearings. Then, the load under both main and secondary bearings opens again through the body of the piers to be distributed in the ten 2 m diameter and up to 50 m deep foundation piles. The form resulting of these tension distributions creates an aesthetically expressive and structurally rational support substructure (Figure 8).

The main structure of the side pedestrian glass covered paths (Figures 6, 11) is made up of steel ribs (with a 15 mm diameter circular hollow section bottom flange) anchored each 3 m to the concrete deck. Seven longitudinal 15 cm × 15 cm glue laminated timber beams spaced 0.75 m are disposed over them, and transverse 15 cm × 3.5 cm Tali wood cross beams serve as a high quality pavement. This pavement is covered by curved tempered and laminated 10 + 10 mm glass pieces supported by curved stainless steel circular hollow section tubes. These tubes continue the ones of the bottom flange of the ribs. As a consequence of the careful selection of materials, all the elements seen by the pedestrians at short distances are high quality ones, not only for their aesthetics but also for their structural properties: Tali wood, stainless steel, curved tempered laminated glass and white concrete.

These glazed balconies are not only useful for protecting the walkers and cyclists from the wind function but also have a structural function since their shape, as tested in wind tunnel tests, improve the dynamic behavior of the deck under wind loads.

4 STUDIES DONE IN DETAILED DESIGNING AND CONSTRUCTION PHASES

The size and complexity of the Third Millennium Bridge has forced us to a huge calculation and detail definition effort from the beginning of the detailed designing (Arenas et al. 2008f), not interrupted in any phase of the construction stage, to achieve the objective of making such a remarkable structure become a reality (Figure 12).

Detailed hydraulic and geotechnical studies were also undertaken due to the importance of the Ebro River water flow, the complex response of the meander area and the low capacity of the soil in the bridge location (Figure 13).

Getting the right mix during construction for the high strength, self-compacting white concrete, a material specifically developed for this bridge, was an ongoing challenge. This concrete was very sensitive to changes in the atmospheric conditions, being constant tests and adjustments in the blend needed to ensure it was consistent. Clearly differenced seasonal mixtures were created. A complete concrete production facility and a testing laboratory were specifically built on site for guaranteeing a permanent control of the blend properties. Real scale models including

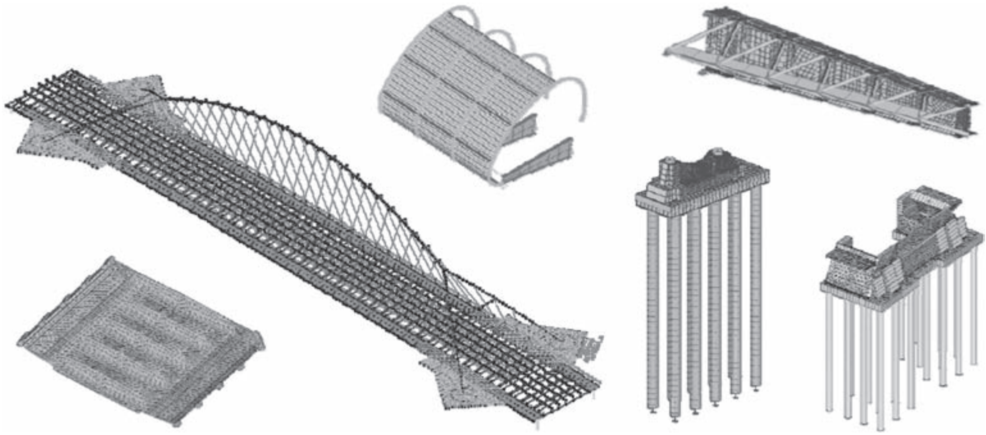


Figure 12. Different finite element models used for the bridge analysis.

Figure 13. Additional geotechnical drilling campaign during construction (left) and hydraulic model to characterize the meander response during construction and after the bridge finishing (center and right).

Figure 14. Site concrete production facility (left), site testing (center) and real scale model (right).

reinforcement bars and post-tensioning ducts were done to verify the accurate setting up of the designed blend in the most complex elements (Figure 14) (Martínez & Segura 2008).

Unusual bridge elements or those of special responsibility as the main pot bearings or the locked coil strand hangers were tested to guarantee their accurate response. The four main pot bearings were tested up to their maximum load (Figure 15) and one cable was led to its ultimate load (Figure 16).

We were compelled to accomplish the analysis of the effects of wind for the bridge through a complete wind tunnel test, due to the wind intensity in the area where the bridge is located and the slender and unusual design of the structure.

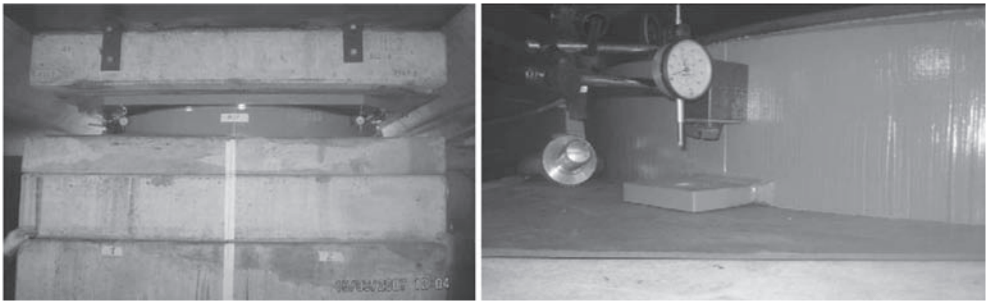


Figure 15. Main pot bearings during their testing in Switzerland.

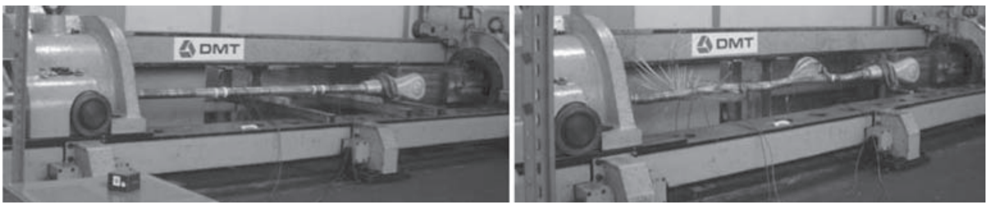


Figure 16. Locked coil hanging cable tested in Germany to its ultimate load, before and after breaking.

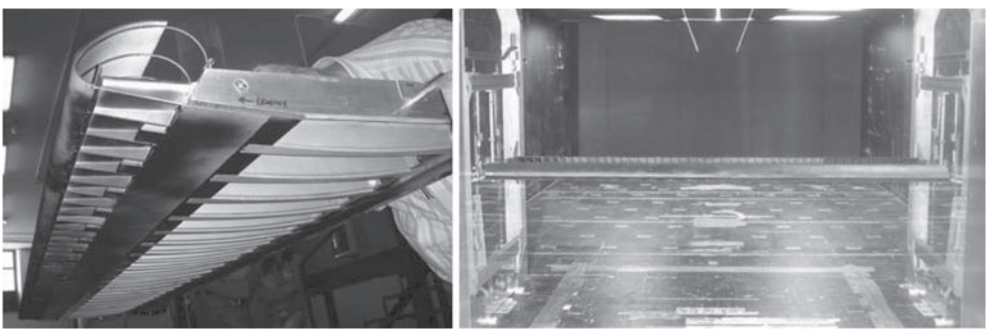


Figure 17. 1:70 deck model for aerodynamic wind tunnel test (left) and the same model being tested.

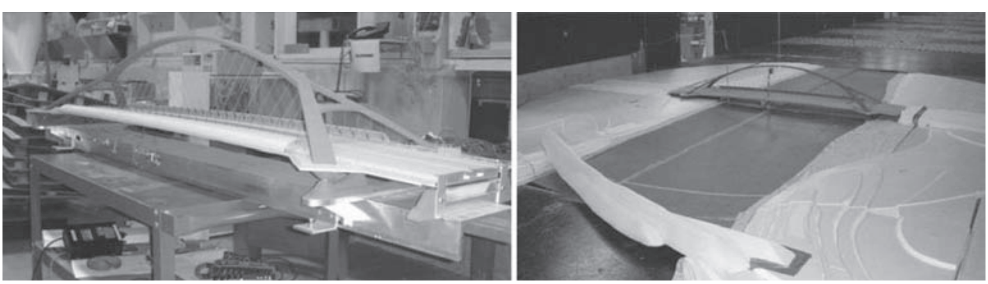


Figure 18. Complete 1:125 scale model for aeroelastic analysis in wind tunnel test (left) and the same model being tested in the Boundary Level Wind Tunnel II of the University of Western Ontario (right).

The response of the bridge under wind loads was studied with a 1:70 section model of the typical deck cross-section (Figure 17) and a 1:125 full bridge aeroelastic model (Figure 18). The section model was tested both in the Boundary Layer Wind Tunnel I of the CEAMA (Andalusian Environmental Center) located at the University of Granada and in the Boundary Layer Wind Tunnel II at the University of Western Ontario (BLWT2-UWO). The full bridge aeroelastic model was tested in the BLWT2-UWO (Terrés-Nicoli et al. 2008).

The section model was tested in smooth flow conditions to investigate the potential for vortex-shedding induced oscillation and aerodynamic instability (either torsional or coupled flutter) and also in turbulent flow conditions in order to develop wind loads associated with turbulent buffeting. Dynamic tests were performed with several values of modal properties in an attempt to assess the symmetric and anti-symmetric modal contributions to the response of the bridge. The test of the section model also included estimates of the aerodynamic forces which are induced by the motion of the bridge deck.

The aeroelastic model was tested in the completed bridge configuration under two upstream roughness conditions (defined as open country and suburban turbulence conditions) to analyze the response characteristics of the full structure to three dimensional turbulent wind representative of the project site, over a full range of wind speeds.

The results were completely satisfactory as no instabilities due to wind effects were found, confirming the good aerodynamic behavior of the bridge, in great part due to the use of concrete as the main construction material and to the shape of the side covered pedestrian paths. This analysis also made possible to determine the real pressure distributions over the glass covering of this balconies, enabling to reduce in 40% the weight of carbon and stainless steel support structure respect to the initial design with code loads.

5 BRIDGE CONSTRUCTION

The construction of the Third Millennium Bridge, carried out by contractor Dragados and Arenas & Associates as construction managers, needed of highly complex erection procedures due to its size, weight and shape, in addition to the need of crossing an important river.

The construction sequence consists in three main activities: deck erection using the launching method over previously built final substructure and additional temporary piers, arch construction over scaffolding arranged over the previously launched deck and introduction of a horizontal load in the crown section of the arch for putting the final loads into play in the whole structure (Ortega et al. 2008a, b).

The launching of a concrete 34 m wide deck, with a total weight up to 200,000 kN, with longitudinal curvature and with a curved bottom cross section, was a real challenge (Figure 19). Once the deck was completely launched and resting over definitive and temporary piers, with 24 m

Figure 19. Launchig of the deck over temporary piers.

Figure 20. Constuction of the arch over a scaffolding supported by the previously launched deck.

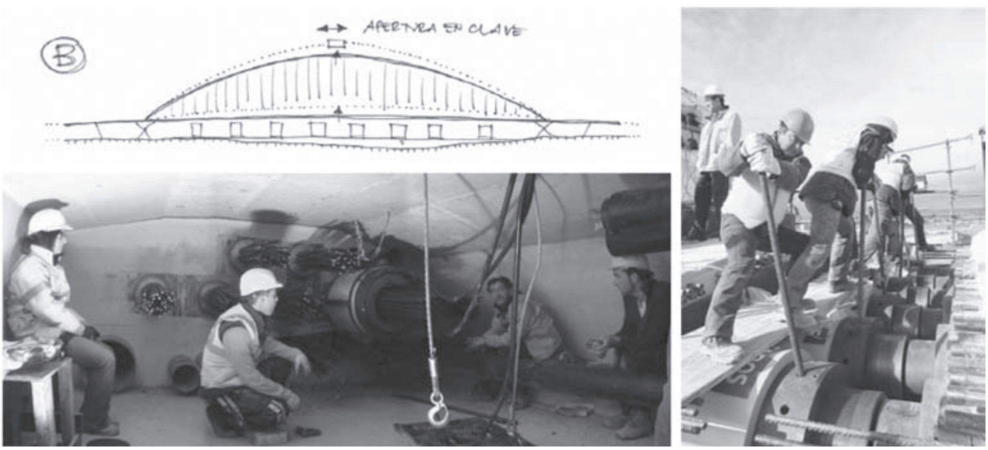


Figure 21. Putting the forces into play in the structure after the arch is constructed. Top left: Sketch of the crown jacking procedure. Bottom left: Introduction of load in the external pre-stressing tendons of the deck to permit it to receive tension efforts when the bridge starts to work as a bowstring during the opening of the crown section of the arch using hydraulic jacks. Right: Adjusting the safety screws of the hydraulic jacks during this operation for avoiding structural damages if a jack failure happens.

spans, scaffolding was built over its top slab, and the arch, inclined legs and cross beam, were constructed (Figure 20).

After these elements were ready and linked with the hanging cables, the deck was partially pre-compressed (to become capable to receive tension forces) through the pre-tensioning of some of the external tendons located inside its side hollow boxes and six huge hydraulic jacks provided by Enerpac were arranged in a free 1.5 m space left in the top of the arch. A load of 120,000 kN was introduced in its crown in a delicate operation without precedent due to its magnitude. This jacking apart of the crown of the arch, carried out with a synchronous hydraulic system, caused an elevation of the arch, an increasing of the tension in the hanging cables, and, after helping with jack cable tensioning, an elevation of the deck (Figure 21). After this world record crown jacking operation and a final cable tension adjustment, the bridge increased its span from 24 m to 216 m, starting to work under its definitive bowstring configuration.

After the jacking operation was concluded, the six jacks were left inside the arch body during the casting of the last section. The slenderness of the arch and its consequently reduced cross section, made impossible the arrangement of an auxiliary structure able to support the introduced load for recovering the hydraulic jacks.

6 CONCLUSIONS

The Third Millennium Bridge intends to be an example of what we call structural architecture, a field of design where every seen element is a response for the structural needs. We tried to create an aesthetically pleasing bridge as an expression of its internal forces.

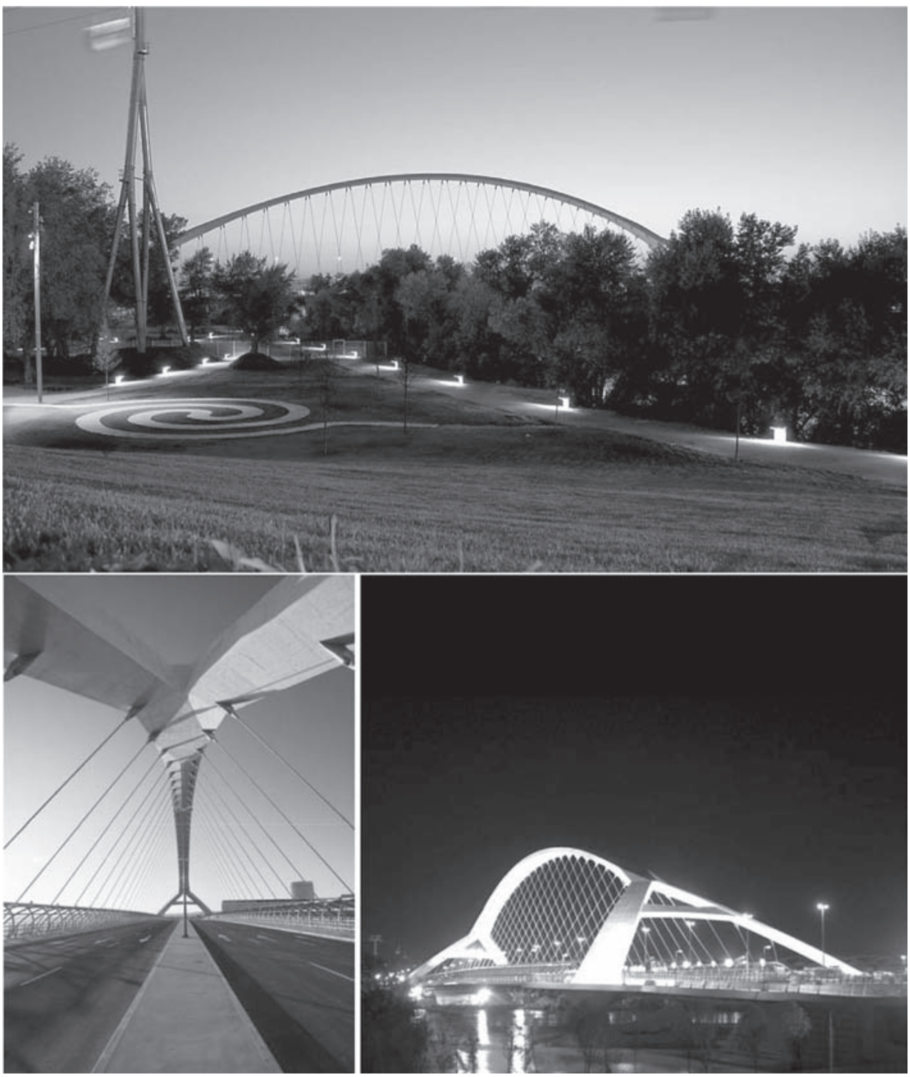


Figure 22. Views from inside and outside the Third Millennium Bridge.

This time size and slenderness were taken to their limits (Figure 22), requiring exhaustive design, calculation and site control effort to create this unique and elegant bridge that becomes innovative in:

- Bridge type and dimensions.
- Material development, with the manufacture of a new high strength, self-compacting white concrete.
- Bridge erection methods, with the launching of a 200,000 kN of weight and double curvature shaped deck and the introduction of a 120,000 kN load in an arch crown jacking operation for making the bridge, when supported by scaffolding and temporary piers, work as a bowstring.

The honest collaboration and effort of many people, working from the beginning to the end as a united team, has made possible the huge challenge of designing and constructing such a remarkable bridge.

REFERENCES

Aguiló, M. 2008. *Puentes para una exposición. Zaragoza 2008*. Madrid: Abada Editores.

Arenas, J.J. 1998. Puente de Las Oblatas en Pamplona. *Hormigón y Acero* 210: 41–53.

Arenas, J.J. 2001. Arch Bridges: shape and stiffness. In C. Abdunur (ed.), *Arch'01: Third international arch bridges conference. Paris, 19–21 September 2001.* Paris: Presses de l'école nationale des Ponts et Chausses.

Arenas, J.J. 2002. *Caminos en el aire: Los Puentes.* Madrid: Colegio de Ingenieros de Caminos, Canales y Puertos.

Arenas, J.J. 2004. Quinto puente sobre el Río Ebro y accesos: Prolongación de la Avenida de San Millán. Logroño. *Proyectar Rioja* 27: 183–185.

Arenas, J.J. & Pantaleón M.J. 1992a. El Puente de La Barqueta sobre el viejo cauce del Río Guadalquivir, en Sevilla. *Revista de Obras Públicas* 3311: 47–63.

Arenas, J.J. & Pantaleón M.J. 1992b. Barqueta Bridge, Sevilla, Spain. *Structural Engineering International* 2(4): 251–252.

Arenas, J.J. & Siviero, E. 2002. El nuevo "Ponte dei Congressi" en Roma. *Revista de Obras Públicas* 3419: 7–14.

Arenas, J.J., Beade, H. & Ortega, A. 2008a. El Puente del Tercer Milenio: un hito simbólico de la ciudad. *Cauce 2000: Revista de la Ingeniería Civil* 141: 18–25.

Arenas, J.J., Capellán, G., Beade, H., Martínez, J. & Ortega, A. 2008b. El Puente del Tercer Milenio sobre el Río Ebro en Zaragoza. *Revista de Obras Públicas* 3490: 75–98.

Arenas, J.J., Capellán, G., Beade, H., Martínez, J. & Ortega, A. 2008c. El Puente del Tercer Milenio sobre el Río Ebro en Zaragoza. *Rutas*: *Revista de la Asociación Técnica de Carreteras 127: 61–64*.

Arenas, J.J., Capellán, G., Beade, H. & Martínez, J. 2008d. El Puente del Tercer Milenio. Retos en el diseño de puentes desde la perspectiva de la ingeniería creativa (I). Génesis de formas. *IV Congreso de ACHE. Valencia, 24–27 November 2008*. Madrid: ACHE. Asociación Científico-Técnica de Hormigón Estructural.

Arenas, J.J., Capellán, G., Beade, H. & Martínez, J. 2008e. El Puente del Tercer Milenio. Retos en el diseño de puentes desde la perspectiva de la ingeniería creativa (II). Desarrollo. *IV Congreso de ACHE. Valencia, 24–27 November 2008.* Madrid: ACHE. Asociación Científico-Técnica de Hormigón Estructural.

Arenas, J.J., Capellán, G., Beade, H. & Martínez, J. 2008f. El Puente del Tercer Milenio. Retos en el diseño de puentes desde la perspectiva de la ingeniería creativa (III). Comportamiento y Cálculo. *IV Congreso de ACHE. Valencia, 24–27 November 2008*. Madrid: ACHE. Asociación Científico-Técnica de Hormigón Estructural.

Martínez, J.F. & Segura, P. 2008. Hormigones blancos autocompactantes de alta resistencia en el Puente del Tercer Milenio—Zaragoza 2008. *IV Congreso de ACHE. Valencia, 24–27 November 2008.* Madrid: ACHE. Asociación Científico-Técnica de Hormigón Estructural.

Ortega, A., Menéndez-Pidal, G., López, S., Tarquis, F. & Arenas, J.J. 2008a. La construcción del Puente del Tercer Milenio. Zaragoza 2008. *IV Congreso de ACHE. Valencia, 24–27 November 2008*. Madrid: ACHE. Asociación Científico-Técnica de Hormigón Estructural.

Ortega, A., Menéndez-Pidal, G., López, S., Tarquis, F. & Arenas, J.J. 2008b. Apertura en clave del arco. Puente del Tercer Milenio. Zaragoza 2008. *IV Congreso de ACHE. Valencia, 24–27 November 2008*. Madrid: ACHE. Asociación Científico-Técnica de Hormigón Estructural.

Russell, H. 2008. Spanish Soul. *Bridge Design and Engineering* 50: 28–30.

Terrés-Nicoli, J.M., Ortega, A., King, J.P.C., Menéndez-Pidal, G., Arenas, J.J. & Losada, M.A. 2008. Efectos del viento en el Puente del Tercer Milenio. ExpoZaragoza 2008. *IV Congreso de ACHE. Valencia, 24–27 November 2008*. Madrid: ACHE. Asociación Científico-Técnica de Hormigón Estructural.

Chapter 41

Walkway over the Hudson—Fast track design and construction

P. Melewski & J. Brizzell
Bergmann Associates, Albany, New York, USA

H.J. Marcelle
The Fort Miller Company, Schuylerville, New York, USA

J. DiStefano
Harrison & Burrowes, Glenmont, New York, USA

ABSTRACT: The Poughkeepsie—Highland Railroad Bridge was originally opened to railroad traffic in 1888. At the time of its opening, and at 1.25 miles in length, the Poughkeepsie—Highland railroad bridge was touted as the longest bridge in the world. Following a fire in 1974, the bridge was closed to all traffic. Walkway Over the Hudson, a private non-profit organization, envisioned a re-birth of this historic structure as the world's longest pedestrian/bike bridge. This concept gained traction in 2007, as the idea of opening the world's longest pedestrian bridge was linked to the Fall 2009 Quadricentennial of Henry Hudson's exploration of the Hudson Valley. New York State Office of Parks Recreation and Historic Preservation will operate the bridge as New York's newest State Park.

To meet this target event, the project had to progress on a very aggresive accelerated schedule. Project processes and determinations had to comply with a myriad of public/private funding partners. And, concerns of a host of third party permit grantees and stakeholders had to be addressed. The private sector 'owners' of the project were focused on the budget and the schedule.

The project was advanced using combinations of the following 'fast track' contracting/procurement processes:

- Traditional Sequential Construction Phases run simultaneously.
- Public—Private Partnership Funding.
- Pre-qualification of Contracting Teams.
- Modified Design—Build Procurement.
- Best Value contracting determination.
- Separate procurement for materials purchases and on-site work.

At the time of the 2009 New York City Bridge Conference, on-site construction work is approximately 95% complete. The presentation will include observations by the bridge deck panel manufacturer (The Fort Miller Company) and the general construction contractor (Harrison & Burrowes).

Attendees will obtain knowledge of the construction methods and contract procurement processes utilized on this unique $30M project. These innovative concepts may be of use to conference attendees in future project applications.

1 BACKGROUND

1.1 *History*

The Poughkeepsie-Highland Railroad Bridge is a 19th Century engineering marvel listed on the National Historic Register and currently under consideration by the American Society of Civil Engineers (ASCE) for National Landmark status. The first cornerstone was laid in 1873, and

Figure 1. Etching of original Poughkeepsie-Highland Railroad Bridge (C. 1900).

when it was completed in 1888, five years after the Brooklyn Bridge, it was touted as the longest bridge in the world (same time frame as Tay and Forth bridges in Scotland). As the first bridge spanning the Hudson River between Albany and New York City, it had an enormous impact on the transportation of freight in the Northeast, and played a key role in shaping today's transportation network (Figure 1).

Railroad traffic over the bridge increased dramatically in the early 20th century, and by World War II train volumes exceeded 3500 cars/day. This rapid growth resulted in the need for immense improvements to the original bridge to accommodate longer and heavier trains. The original main span structure consisted of only the north and south trusses, but in 1906, a center truss was woven into the structure to increase capacity. A second strengthening project was undertaken in 1918 when the main span deck system was replaced.

Usage began a steady decline in the late 1950's after the creation of the interstate highway system and the opening of a competing railroad bridge just south of Albany. The turning point in the history of the bridge occurred at the east approach when, in 1974, a spark from a passing train ignited the timber ties. Fire retardants that had been applied to the ties were ineffective and the fire suppression system piping failed due to the lack of maintenance, resulting in the complete loss of the timber deck on the east approach trusses (approximately 900 feet), along with significant damage to the floorbeams and stringers. Operations on the bridge were terminated, and although repair plans were prepared, they were not implemented and rail service on the bridge came to an end.

After 30+ years of discussion, studies, and failed endeavors by various agencies and private groups on how to make the best use of the structure, Walkway over the Hudson, a 501(c) (3) not-for-profit membership organization, obtained full ownership of the bridge in 1995. In the fall of 2006, Walkway over the Hudson hired the Bergmann Team (Bergmann Associates; McLaren Engineering Group; Ulazewicz, Melewski, and Greenwood; and Howard/Stein-Hudson) from a field of nineteen (19) teams to certify the structural integrity of the dormant bridge and to produce and implement a comprehensive plan for its use as a public park and walkway.

1.2 *Bridge description*

The Poughkeepsie-Highland Railroad Bridge spans between the City of Poughkeepsie, Dutchess County, and the Town of Lloyd, Ulster County, New York. At each end, the bridge connects to abandoned railroad right-of-way that is currently being converted to a 30 mile trail system that will in turn link up to an extensive Hudson Valley Greenway trail network (Figure 2).

The bridge consists of three distinct segments: a west approach viaduct, the main spans over the Hudson River, and an east approach viaduct (Figure 3).

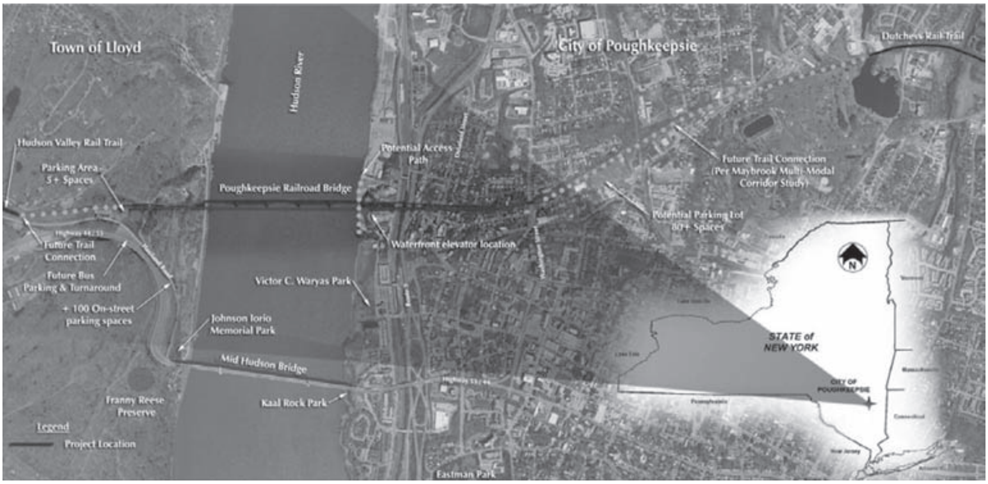


Figure 2. Project location map.

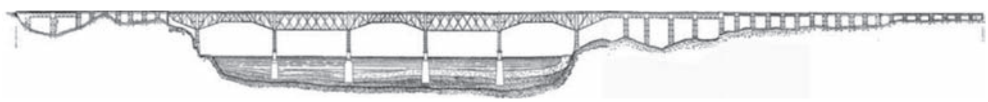


Figure 3. Bridge profile.

The West Approach is 1,034 feet long on a tangent alignment and 1.25% downgrade towards the river, and consists of two truss spans and numerous girder spans supported on steel towers. The north, center, and south structure lines are spaced 11 feet apart. The West Approach spans over one local roadway and through rolling terrain in the Town of Lloyd.

The Main Span is approximately 3,094 feet long on a tangent alignment and flat grade, and consists of seven truss spans supported on steel towers on concrete piers at each shore and four steel towers on concrete and masonry piers on concrete filled timber cribbing in the Hudson River. The Main Span crosses over a local roadway and CSX railroad on the west shore, the Hudson River, and the Central Hudson utility station on the east shore. The north, center, and south trusses are spaced 15 feet apart. The bridge deck is over 200 feet above the Hudson River.

The East Approach is 2,640 feet long on a tangent followed by circular curve alignment and 1.25% downgrade away from the river, and consists of five truss spans and numerous girder spans supported on steel towers. The north, center, and south structure lines are spaced 11 feet apart. The East Approach spans several local roadways, one state highway, MTA's Metro North railroad, and an established urban residential neighborhood in the City of Poughkeepsie.

2 FAST TRACK DESIGN AND CONSTRUCTION PROCESS

2.1 *Public/private collaboration*

The primary stakeholders, Walkway Over the Hudson, and New York State Parks established the opening of the project for early October, 2009 to coincide with New York State's celebration of Henry Hudson's exploration of the Hudson Valley in 1609. This aggressive goal was etched in stone during Governor Spitzer's State-of-the-State address in January, 2008. In short, we had two years from the first day that the dormant bridge had been inspected in 40 years (September 2007) to completion of construction (September 2009).

In order to achieve this very aggressive goal, several functions that are usually done in chronological/linear fashion such as inspection, design, environmental review, construction methodology and funding had to be conducted simultaneously (Figure 4). Early and continual collaboration with stakeholders was required. For example, while our inspectors were out on the steel, brainstorming sessions were conducted with regional bridge contractors, prefabricated panel manufacturers, and local stakeholders to identify any fatal flaws in the construction alternatives (Figure 5).

Having the completion date for the project coincide with the Henry Hudson Quadricentennial, and having the Walkway Over the Hudson as the focal point of the Hudson Valley celebration, certainly facilitated our efforts to accelerate the design process. This also energized the cooperation of numerous stakeholders and regulatory agencies that had to sign off on various aspects of the design or construction. Permittees and regulators included: the NYS Department of State; the NYS Department of Transportation; NYS Department of Environmental Conservation; NYS Department of Labor; U.S. Coast Guard; U.S. Army Corps of Engineers; Central Hudson

Figure 4. Fast track design and construction process.

Figure 5. One of many collaborative brainstorming sessions.

Gas & Electric which has a major substation beneath the bridge; MTA/Metro-North, AMTRAK, and CSX railroad companies; local municipalities and two counties.

As stated earlier, the aggressive schedule would not have been possible without public and private collaboration. The spirit of cooperation became contagious as the design and construction advanced, to the point that no one stakeholder wanted to be the one "holding up" the project. Stakeholders who had a significant role in funding include: Governor Patterson's Office; NYS Office of Parks and Recreation; the Dyson Foundation; U.S. Senators Charles Schumer & Hilary Clinton; Congressman Maurice Hinchey; Scenic Hudson; State Senator Stephen Saland; and the Dutchess County Industrial Development Agency. Total cost of the project, including design and construction inspection is approximately $35,000,000. The cost is split approximately 50/50 between public and private sources. Federal monies, provided through various grants, are being utilized on later phases of the work such that the involved federal review processes did not delay the start of construction activities.

In addition, the NYS Bridge Authority (which owns the nearby Franklin Delano Roosevelt Mid-Hudson Bridge), and the National Parks Service offered free technical advice, and the use of their facilities. The support and assistance of the City of Poughkeepsie, the Town of Lloyd, and Ulster and Dutchess counties were also vital in securing the necessary reviews, permits, and agreements.

2.2 *Compressing the construction schedule*

Significant traditionally linear steps in the final design and construction phase were run in parallel in order to progress the schedule. This took careful planning and coordination to be sure nothing was missed, that the functions could operate simultaneously and that the accelerated processes were in compliance with NYS funding requirements. While design was still in its infancy in the Fall of 2007, it was determined that distinct, overlapping construction contracts were necessary in 2008 in order to sufficiently compress the schedule. The Demolition Contract (removal of existing ties, rail, etc), and the General Construction Contract would be let within weeks of each other. In addition, a separate pre-purchase agreement for the 1,000 feet of west approach span decking was made with the panel manufacturer to get a jump start on panel production; this assured that the general contractor had a sufficient supply of panels on-site at the time demolition was completed. The General Contractor (Harrison & Burrowes) moved onto the site days before the demolition contractor (ERSI) left the bridge.

The following techniques were utilized for the general contract:

- Mandatory Bidder Site Visit.
- Pre-Qualification of Contractor Bidding Teams (General Contractor and Fabricator).
- "Best Value" (Cost and Quality) Bidder Team Selection.
- "Modified Design/Build" with Bidder Team Selection bids based on 90% plans. The designer and contractor team work together to achieve 100% design/final bid proposal plans using the Bidder Team's means and methods.
- Work collaboratively in a design/build fashion throughout construction.

3 PROPOSED RESTORATION

The intent of the proposed restoration is to stabilize and preserve this historic structure, while converting its use from a railroad crossing to a linear park. The renovated structure will be incorporated into the New York State parks system in conjunction with regional open space and a 30 mile trail system that extends from either end of the bridge. The renovated bridge is expected to be a destination for tourism, inducing some 267,000 visits annually (see Figure 6). Finally, the renovated bridge needs to be able to support vehicles to be used for maintenance and inspection, as well as serving as an emergency alternate river crossing should the adjacent Mid-Hudson Bridge become closed due to an incident (Figures 7 & 8).

Visitation & Direct Spending in Dutchess & Ulster Counties		
Type of Attendee	Visits	New Direct Spending
Local Users (hiking, biking, jogging)	157,699	$ -
Visitors (outside Dutchess & Ulster)	110,000	$14,617,969
Total Local Users & Visitors	**267,699**	**$14,617,969**

Total New Economic & Fiscal Impact	
Local Jobs	258
Local Visitor Spending	$21 million
Local Tax Revenue	$727,411
State & Local Tax Revenue	$1.3 million

Figure 6. Economic benefits table.

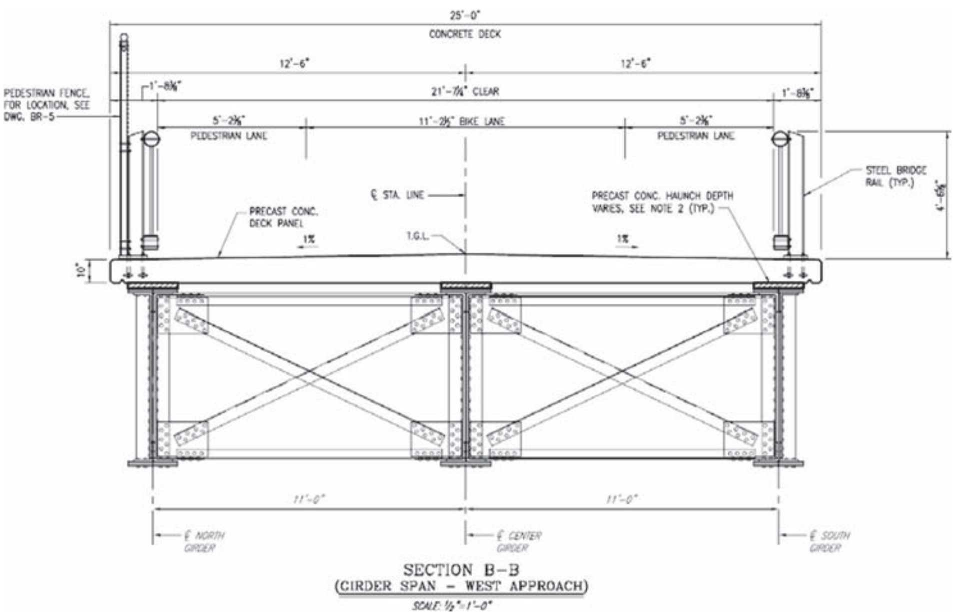


Figure 7. Proposed approach span deck section.

Primary components of the restoration include:

- Removal of remaining railroad track and utilities,
- Steel repairs (both immediate and short term repairs, (1–5 years, are identified),
- Fabrication and installation of new pre-cast deck panels on the approaches, main span and overlook areas,
- Installation of three 35' wide river viewing platforms,

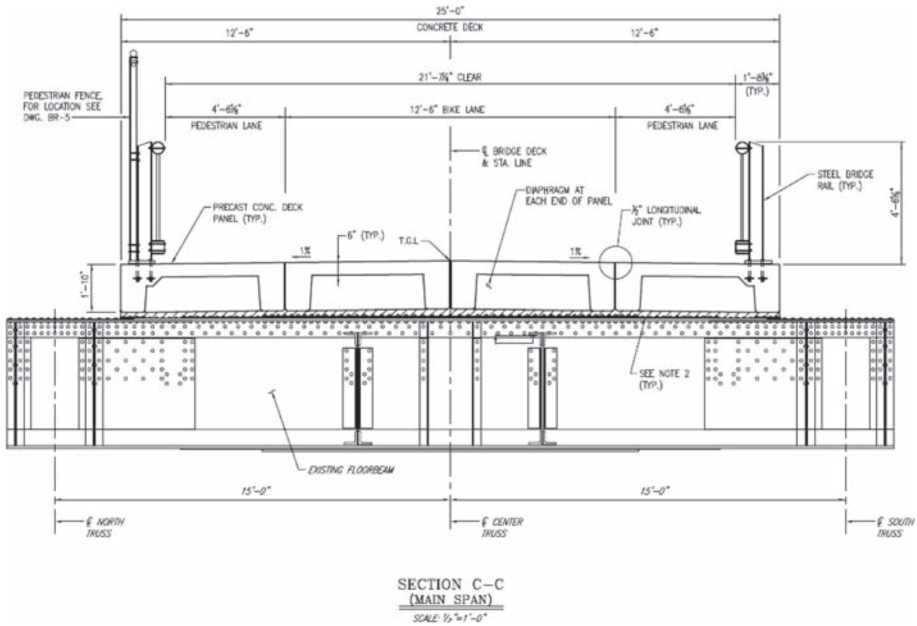


Figure 8. Proposed main span deck section.

- Fabrication and installation of new bridge railing,
- Installation of "dark sky" lighting, electrical outlets and security systems,
- Grading and paving approach access and parking areas, and
- Construction of a 20 story riverside elevator.

Each of these areas posed unique challenges, involving the overall project objectives of safety, cost containment and completion for opening by September, 2009. Each of these challenges were met and resolved through the use of innovation and cooperation.

3.1 *"Fast Track" contracting actions and results*

Early in the project, it was obvious that conventional—linear methods of project development and contracting would not allow completion of a useable walkway by the target opening in September, 2009. Actions were taken to compress all phases so that there were concurrent activities underway. Contracts for certain portions of the work were split apart, scheduled and prioritized to facilitate completion of the walkway within time and budget constraints (Figure 9).

3.1.1 *Demolition contract*

The first activity involved removal of the remaining railroad tracks, timbers and utilities on the structure. A bid contract for this work was advertised concurrent with advertising for pre-qualification for the General Construction. Bidders were required to submit their cost proposals 18 days after the ad was issued. This time period included a mandatory pre-bid meeting at the project site. Many of the potential bidders had been involved with earlier concept development of the project, and were well able to meet this short time frame. The successful bidder, Environmental Remediation Services Inc. (ERSI) offered a bid proposal in line with the estimate.

ERSI's proposal included a very competitive bid for alternate work (removal of abandoned electric transmission lines), which was incorporated into the contract agreement.

The demolition contract included incentive/disincentive provisions intended to promote timely completion of the demolition work so that the general contractor would have unrestricted access to

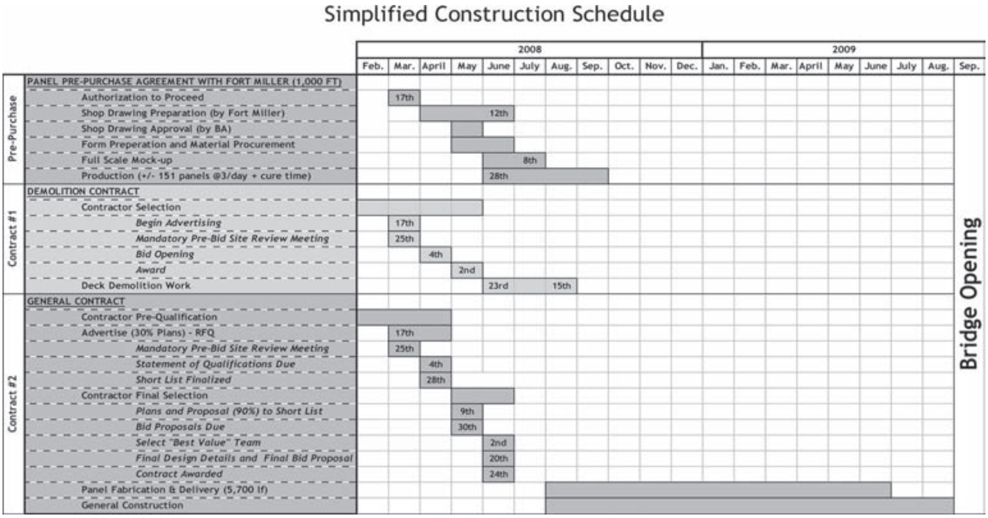


Figure 9. Simplified schedule of the three major contracts (lighting/electrical contract not shown).

Figure 10. Demolition of rail ties.

the work site. ERSI was able to meet the target date for removal of all railroad features. By mutual agreement with the owners, and the then selected General Construction contractor (Harrison & Burrowes), removal of the abandoned electric transmission lines was deferred until the new deck is placed. Concurrent with this change in the contract, all incentive and disincentive provisions were eliminated from the contract (Figure 10).

3.1.2 *Panel pre-purchase* (*West approach spans*)

Lead time for deck panel shop drawings and fabrication was greater than could be accommodated in the General Construction contract. To assure that panels were available for placement, a separate no-bid materials purchase agreement was negotiated for the panels on the west approach spans—approximately 1,000 feet. The selected pre-cast supplier, Fort Miller Company, had been involved in the concept development phases of the project, and was the highest rated of 4 pre-cast firms that were reviewed as part of the General Construction pre-qualification process. Materials

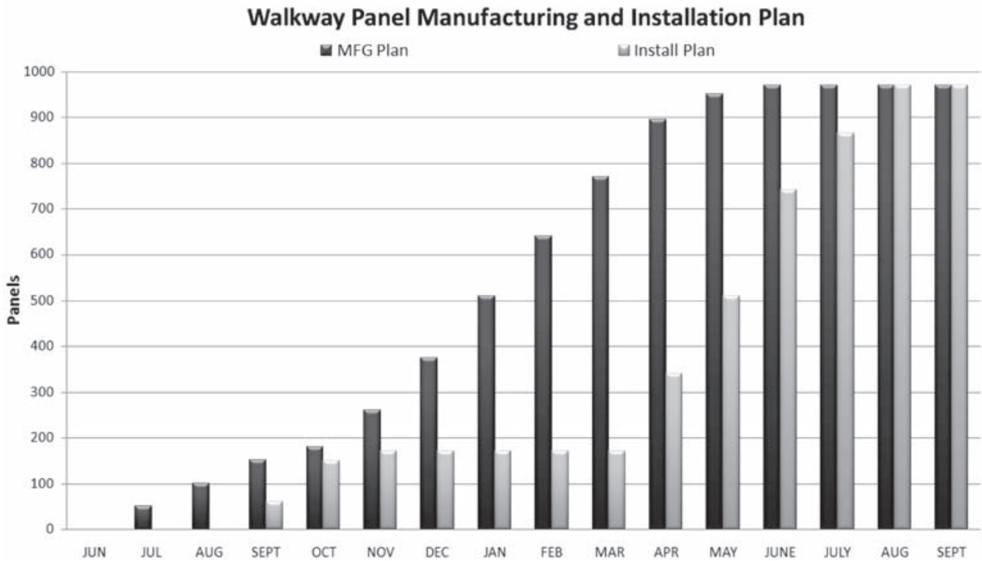


Figure 11. Prefabricated concrete panel production and installation schedule.

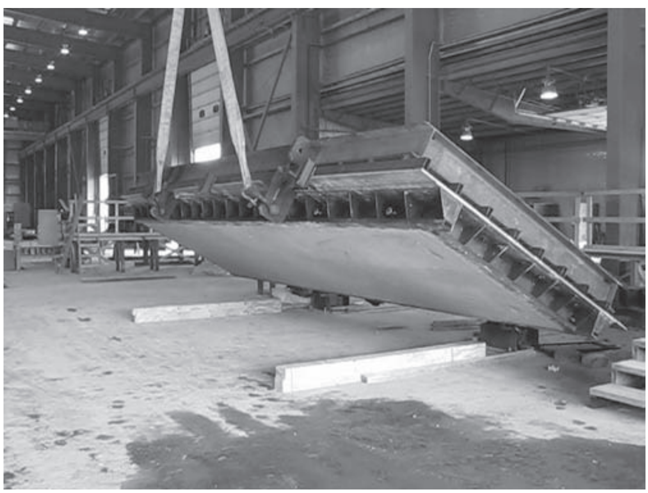


Figure 12. 25" by 6'–9' panel being rotated at the Fort Miller facility.

costs for this pre-purchase were in line with estimates. Deck panels were detailed, fabricated and delivered to the project site for installation in accord with the over-all schedule for installation commencing the first week of September, 2008 (Figures 11 & 12).

3.1.3 *General construction contract*

General Construction activities are intended to complete all work necessary to open the bridge in September, 2009. These include: initial steel repairs on the entire structure; modification of sub-structures (primarily abutments); fabrication of deck panels for the Main Spans and the East Approach Spans; installation of deck panels on the entire structure; and fabrication and installation of the bridge railing.

To achieve assurance that the work would be awarded to a contracting team that will provide a quality product and will complete on time, the project utilized pre-qualification of bidder teams. A detailed Request for Qualifications (RFQ) included both pass/fail and quality based factors that were used to rank the proposing teams. Four contractors, utilizing a total of 3 pre-cast fabricators, responded to the RFQ. Following initial review, proposing teams were contacted for additional information in areas where their initial proposal did not fully address the object of a qualification factor. With the supplemental information, a four (4) person team each separately rated each of the proposers' submission in each of the quality factors. These were reduced to an over-all rating for each proposing team through consensus among the team members. The ratings were then used to generate a quality adjustment factor that would be applied to each teams bid proposal.

All four proposer teams were invited to submit bid proposals based on 90% complete plans. Initially, bid proposals were due 2 weeks after the 90% plans were provided to the bidding teams. Requests for more preparation time resulted in an extension of the bid submission date by one week. As the letting date approached, a number of factors—including schedule, risk assessments, and skyrocketing fuel and material costs—resulted in three of the four teams electing to not submit bid proposals. The single submitting team, Kirchhoff Construction Management (KCM) with Harrison & Burrowes Bridge Constructors (H & B) & the Fort Miller Company (FMC) had been the highest rated of the four proposing teams. However, the bid proposal was significantly above the estimate due to the factors mentioned.

Following the bid submissions, a series of meetings with the proposing team and design refinements were completed. Much of the focus of this final design effort was intended to reduce contractors' risk, required project staffing, and customize design to match contractors' equipment. The most significant design change resulting from this effort modified the panel attachments from under deck operations to top of deck. In addition, KCM withdrew as the Construction Manager, Incentive/disincentive clauses were removed, the work was split into two phases, all parties agreed to work in a cooperative/partnering manner, and a decision was made to directly purchase the precast panels from Fort Miller (similar to the West Approach). After revised plans were provided to the contracting team, a final bid proposal was submitted, reflecting a cost reduction of approximately 19%.

The general contractor began steel repair work in mid-August and began installing deck panels in early September, 2008 (Figure 13). By the end of 2008, all deck panels and bridge railing was

Figure 13. Panel installation on Main Span (looking East).

Figure 14. Steel crew (right), followed by panel installation crane (left).

installed on the west approach spans; deck placement also extended approximately 360' on the main span and steel repairs had progressed to the second main span pier (Figure 14).

3.1.4 *Bridge approaches and parking*

Most of the properties used for construction access to the bridge, and intended for future use as access and parking, are owned by third parties. Work required numerous temporary easements and rights of entry. Obtaining final approval to create permanent facilities on these properties has required that the contracting for that work be delayed. At the time of this writing, the design of these facilities is complete, and the general contractor has provided a price to include this work as a change order to the existing contract.

3.1.5 *Lighting, electrical & security*

These features were deferred while the ownership team pursued alternate feature details and funding. These issues are now resolved. A separate project was fast tracked through the federal funding processes in early 2009, and a contract will be let in the spring of 2009. Completion of this work, in time for the bridge opening, will require close coordination with the general contractor.

3.1.6 *20 story waterside elevator at east shoreline*

Identified as a desirable multimodal feature connecting the bridge deck to the Poughkeepsie Railroad Station and city waterfront, final design work will begin in 2009 when separate funding becomes available.

4 CONTRACTOR'S PERSPECTIVE

4.1 *Jeff DiStefano of Harrison and Burrowes bridge constructors (General contractor)*

In May of 2008, Harrison & Burrowes was selected to build a pedestrian walkway on the old railroad bridge that spans the Hudson River from Highland, NY to Poughkeepsie, NY. This project is, more than likely, the most unique project we have ever been involved with. It is unique in a variety of ways. Probably the most unique aspect of this project is the fact that, when completed, it will be the longest pedestrian bridge in the world. But as I continue you see more clearly what I mean by unique.

In the summer of 2007, I was invited by our friends with the Fort Miller Group to attend a brainstorming session at the NYS Bridge Authority Headquarters in Highland, NY. It was at this meeting that I met Mike Duffy and some of the Walkway Committee people. These people had a vision, and as far-fetched as I thought it was, I was very impressed by the way they listened and learned. They also had some very interesting ideas and concepts.

It was also at this meeting that I met a group of engineers from Bergmann Associates. This group of guys had taken a very complex project and done some very preliminary plans in a very short period of time. From this meeting and throughout the fall of 2007 I had the opportunity to continue brainstorming with Bergmann guys and the dialog was always very open and very productive. Guys like Mike Cooper and Peter Melewski were always very ready to listen to what I tried to offer for ideas.

As we fast forward to May of 2008 and bid day, the group of Harrison & Burrowes and Fort Miller were the apparent low bidders. But! … there was a problem! A major problem! That major problem was we were way over budget.

But this is where the word unique was redefined. Bids were due at 11:00 am that day and by 2:00 pm our team and Bergmann and the Owner convened the first of a series of meetings to now try and arrive at a design that would work and be within the budget that the Owner could live with. On that bid day it was exciting to hear the great ideas that were put on the table for discussion. Over the next several weeks a design was arrived at that worked for the "WHOLE" team. I say "WHOLE" team because we were now a complete team—Owner, Designer, Fabricator, and Contractor. As construction began the design process continued and everyone continued to pitch in as one team.

We are now approximately 25% complete (Winter 2008/2009) and it is an absolute testament to work Partnership. Being a part of this very unique opportunity truly speaks to the concept of partnering (Figures 15 &16).

4.2 *The Fort Miller Company* (*Pre-cast panel fabrication*)

The Fort Miller Co., Inc. was selected as the manufacturer of the precast concrete elements for the project and established a contract directly with The Walkway Over the Hudson. The main objective for the direct purchase by the owner was to control costs and to expedite the fabrication process. The contract called for the use of high quality precast concrete components manufactured with tight tolerances. The panel details were developed by Bergmann Associates with input from Fort Miller and Harrison & Burrowes that would allow the panels to be placed in the field at an aggressive pace required by the project schedule. The use of precast members assure both strength and functionality.

Figure 15. Existing steel survey (looking South).

Figure 16. Structural steel repair crew (looking East).

Using innovative casting methods with flexibility built into the forming system, we are able to make changes quickly and adapt to address field conditions. Multiple form setups for both the Main Span panels and the East Approach panels are being used in order to meet the project schedule. The West Approach was completed before the end of the 2008 construction season with a portion of the Main Span installed prior to the winter shut-down. Precast panels for both the East Approach and the Main Span were produced throughout the winter months of 2009, with panels stock piled and ready to go for the 2009 construction season. Fabrication of all of the precast elements for the project is scheduled to be complete by late May to early June 2009.

The project has been a "work in progress" with a design/build flare. The unimpeded communication between Bergmann Associates, Harrison & Burrowes and The Fort Miller Co., Inc. is the major contributing factor in expediting the project as any adverse field condition is quickly overcome with a telephone call. The cooperation between the team members has been key to the success of this project.

5 PRE-CONSTRUCTION CONDITIONS AND ANALYSIS

5.1 *Bridge inspection*

The inspection followed the New York State Department of Transportation guidelines for an In Depth Inspection, with either hands-on or visual inspection of all elements of the superstructure and all land-based substructures above the ground line. A diving inspection by McLaren Engineering Group was also conducted for the four piers located within the Hudson River. The inspection team consisted of two Team Leaders and up to seven Assistant Team Leaders. In addition to meeting the qualifications set forth by New York State for bridge inspectors, each inspector was also certified by the Society of Professional Rope Access Technicians (SPRAT), which is a national standard for industrial rope access. The inspection methods were limited to hands on and visual inspection, sounding of concrete or masonry substructures, and non-destructive ultrasonic thickness gauging of various structural steel members. No destructive testing was performed. Access to the superstructure and substructure was achieved by employing a variety of industrial rope access climbing methods in accordance with the guidelines established by SPRAT. Ropeworks Industrial Group, Inc. and Skala Group, Inc. provided technical climbing assistance, on site safety oversight, and emergency rescue capabilities; facilitated installation of safety and climbing ropes; and coordinated inspection access with local emergency service providers, utilities, and adjacent property owners (Figure 17).

5.1.1 *Approach viaducts*

The steel superstructure and substructures of the West and East Approach viaducts are in fair condition (10% to 20% typical section loss) with isolated areas of greater than typical section loss, primarily where timber ties were in contact with the steel superstructure. Although the section loss in these areas is relatively high, the affected components comprise only a small percentage of the overall member section. For example, the girder spans are typically in fair condition (10% to 20% section loss) with the exception of the top flange cover plates which are in poor condition (up to 50% section loss) with severe rivet head loss (up to 100% loss of rivet heads). Similarly, the truss spans are typically in fair condition (10% section loss at bottom chord and diagonals), except that section loss is greater at the top chord (10% loss in the webs, 20% loss in the bottom flange angles, up to 50% loss in the top flange angles). There are also numerous top and bottom lateral bracing members and gusset plates on the girder spans that have 50 to 100% localized section loss (Figure 18).

In addition, in 1974 the East Approach suffered fire damage to the deck framing system from Pier 7 to the middle of the span 8/9, and within this area there are several buckled stringers as well as a few buckled floorbeams. These particular stringers and floorbeams appear to be no longer capable of functioning as originally intended; however, the remainder of the deck framing system

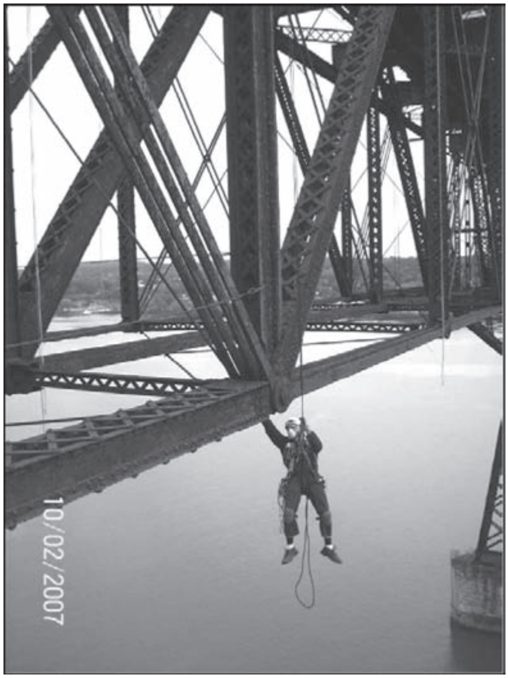


Figure 17. Inspectors employ redundant rope access to examine a lower chord pinned connection.

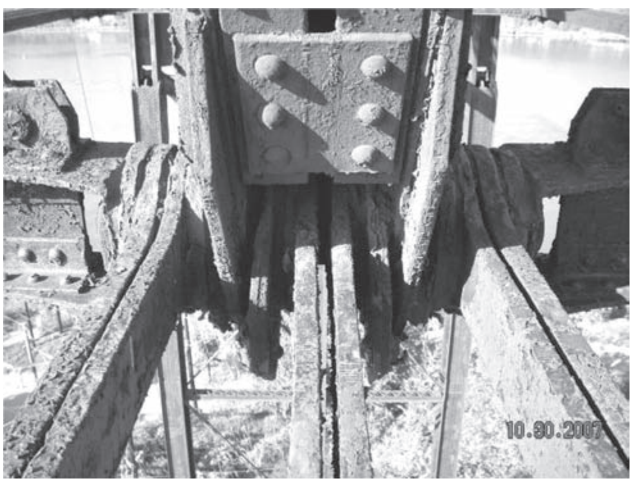


Figure 18. Congestion and pack rust between numerous structural elements at truss panel points made it difficult for inspectors to assess the extent of section loss.

is otherwise in fair condition (10% to 20% section loss). There was no noticeable fire damage to the truss superstructure.

5.1.2 *Main span*

The deck framing system (stringers and floorbeams) is typically in fair condition (10% to 20% overall section loss), with the exception of the top flange cover plate at the center of the floor-beams which shows additional deterioration (25% to 75% local section loss).

The truss members are typically in fair condition (10% to 20% overall section loss), with the exception of the following cases of greater than typical section loss: the top cover plate of the top chord built-up members (up to 50% local section loss), and eyebars at the Center Truss suspended span hangers (up to 30% local section loss). There is also significant localized deterioration in the top and bottom chord lateral bracing, especially at connections and splices.

The bearings are in fair condition, with the exception of Piers 2 and 5 where the expansion bearings show no signs of recent movement and are presumed to be frozen in an expanded position.

The steel tower substructures are in fair condition (10% to 15% overall section loss in primary members), except the secondary bracing members which show significant deterioration (up to 100% localized section loss) at connections to the tower columns. This condition is caused or accompanied by pack rust that has buckled and/or cracked the legs of the bracing angles and led to increased localized section loss in the pier tower column cover plates (up to 30% localized section loss).

5.2 *Analysis and load rating*

Varying degrees of complexity were required to perform the structural analyses and load rating. For the east and west approach spans, simple two-dimensional analysis methods were employed. Engineers took advantage of the extensive repetition of structural configuration and analyzed groups of similar spans together. This approach greatly minimized the amount of effort required to load rate the approach spans. Engineers also made conservative assumptions with regard to section loss rather than examining numerous levels of deterioration; for example, in order to address the deterioration of the girder span top flange cover plates, these elements were excluded from the calculation of structural capacity in all girder spans.

For the Main Span, a more detailed analysis was conducted. Engineers chose to model the span over the Hudson River with a three-dimensional truss model that incorporated both the superstructure and the pier towers. Again taking advantage of symmetry, only half the structure was modeled, with the dead loads from the mid-span suspended span applied as constant dead loads at the end of the mid-span cantilever span. Knowing that the bearings at Piers 2 and 5 were frozen, modeling the steel pier towers together with the superstructure allowed the model to take advantage of the flexibility of the pier towers in order to dissipate the loads that would have otherwise been attracted by the frozen bearings.

The controlling Capacity/Demand ratio for the load cases described above was 1.4 occurring at an approach span truss. This corresponds to approximately 30% section loss. Based on these results, engineers chose to develop repair details for any primary member exhibiting greater than 20 percent section loss.

Figure 19. Existing bridge deck in 2007 (looking East).

Figure 20. New deck and railing in early 2009 (at west abutment looking East).

Figure 21. Filming of "Project Xtreme" episode.

Figure 22. Rendering of the new deck.

6 CONCLUSIONS

After three decades, the historic Poughkeepsie Railroad Bridge is awakening from its long slumber (Figure 19) to resume its role as the "great connector". As with the original opening in 1888, the bridge's rebirth in 2009 is creating considerable excitement, and will be an economic benefit to the Hudson Valley (Figures 20–22). To learn more about Walkway Over the Hudson's efforts, read additional project information, and see the latest construction photos, please visit www.walkway.org.

REFERENCES

Mabee, C., "Bridging the Hudson", Purple Mountain Press, 2001.

Melewski, P. et al., 4th New York City Bridge Conference "Walkway Over the Hudson (Historic Bridge to Northeast Recreational Destination)". "Innovations in Bridge Engineering Technology", Taylor & Francis, 2007.

Chapter 42

Verrugas Viaduct and its reconstruction, Peru, South America

K. Gandhi
Gandhi Engineering, Inc., New York, New York, USA

ABSTRACT: Henry Meiggs of San Francisco, a fugitive from the U.S. Government, signed a contract with the Peruvian Government in 1869 to build a 136 mile railroad connecting Callao (main port of Peru) and Lima (the Capital about 7.5 miles away) with Oroya (a small town close to the mining district of Cerro de Pasco) in six years for U.S. $25,875,000. Mr. Meiggs was the biggest employer of U.S. engineers next to the U.S. Army at that time. The Verrugas Viaduct was 51.8 miles from Lima and 5,850' above sea level in the Andes mountains. The wrought iron viaduct was designed, fabricated and shipped by the Baltimore Bridge Company via land and sea to Callao from where it was transported to the site on rail and mules. The viaduct consisted of four Fink truss spans, three of 100' in length and one 125' long. The three piers were 146', 252', and 179' in height; and were all 50' wide. The total length of the viaduct was 575'. The fabricator had planned to frame the spans in the bed of the gorge and lift them bodily into place. However, Leffert L. Buck, thew Resident Engineer, devised an ingenious erection scheme, and the viaduct was opened to traffic on January 8, 1873. In March 1889, heavy floods and rock slides in the gorge pushed the central pier causing collapse of the bridge. Mr. Buck was retained for the replacement bridge and he designed a cantilever bridge with two piers, two side spans of 140 ft. each, and a central opening of 235 ft. The existing abutments were used with the same span of 575 ft. Construction started on July 1, 1890, and the bridge was completed on January 1, 1891. This paper describes details of both bridges, difficulties encountered during their constructions, and people connected with them.

1 INTRODUCTION

In December of 1869, Henry Meiggs, an American entrepreneur and railroad contractor, signed a U.S. $25,875,000 contract with the Government of Peru in South America to design and build a 136 mile railroad between Callao, Lima and Oroya through the Andes Mountains, generally known as Oroya railroad (Figure 1). The contract was to be completed in six years. Callao was the port city, Lima was the Capital of Peru, and Oroya was a small town in the mining district of Cerro de Pasco.

Table 1 (New York Times, 1873b) conveys some idea about the magnitude and difficulty of the work where the grades were limited to a maximum of 4 percent, and the minimum radius of curves to 352 ft. (Rand and Owen).

The Verrugas Bridge or Viaduct was oriented east-west and situated 51.8 miles from Callao. The mean elevation above the sea level was 5,836 ft. The gorge which it crossed was for a very small stream which discharged into Rimac River, about 800 ft. north of the bridge. The width of the gorge was 525 ft. and the depth at the center was 252 ft.

Meiggs selected Baltimore Bridge Company (BBC) of U.S. to design and fabricate the bridge. Because of the difficulties in designing and erecting a bridge in a remote location, he requested assistance from Walton White (W.W.) Evans of New York who was a consulting engineer to the Peruvian Government. Charles H. Latrobe was Assistant Engineer of BBC in charge of design.

Evans was familiar with the wrought iron viaducts being built in the U.S., and was sure that such a viaduct would work for Verrugas. He prepared several schemes, and asked for plans and cost estimates for these schemes from the BBC. To reduce the span lengths of trusses, he designed 50 ft. long trestle piers. The final scheme that was accepted by both Evans and BBC for fabrication

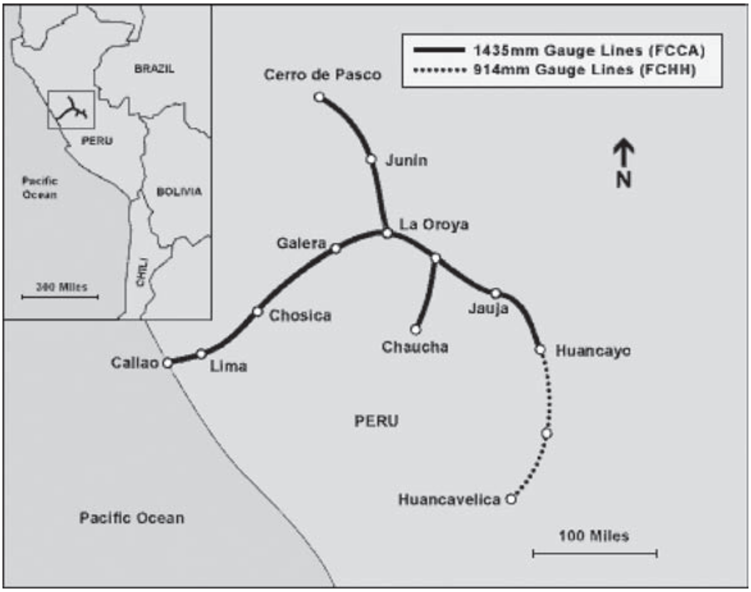


Figure 1. Map showing route from Callao to La Oroya.

Table 1. Distances and elevations along the railroad.

From Callao to	Miles	Elevation, feet.
1. Lima	7.50	448
2. Quiroz	11.75	808
3. Santa Clara	18.50	1,312
4. La Chosica	33.50	2,800
5. Cocachara	44.75	4,588
6. San Bartolome	46.75	4,905
7. Verrugas Viaduct	51.75	5,840
8. Surco	55.75	6,655
9. Matucana	62.25	7,788
10. San Mateo	77.50	10,530
11. Summit Tunnel	104.50	15,645
12. Yauli	119.00	13,420
13. La Oroya	136.00	12,178

had four Fink trusses (3 of 100 ft. span and one of 125 ft. span) and 3 piers (each 50' long) making a total bridge length of 575 ft. The heights of three piers were 179 ft., 252 ft., and 146 ft. starting from west to east (Figure 2, Railroad Gazette, 1873).

2 ORIGIN OF IRON RAILWAY VIADUCTS IN U.S. (GREINER, 1891)

It is interesting to note that Benjamin H. Latrobe, father of Charles H. Latrobe, was the first person in the U.S. to promulgate the use of iron viaducts for railroad bridges. He was associated with the Baltimore & Ohio Railroad; and two of his assistants Wendel Bollman and Albert Fink developed distinct iron trusses that bear their names.

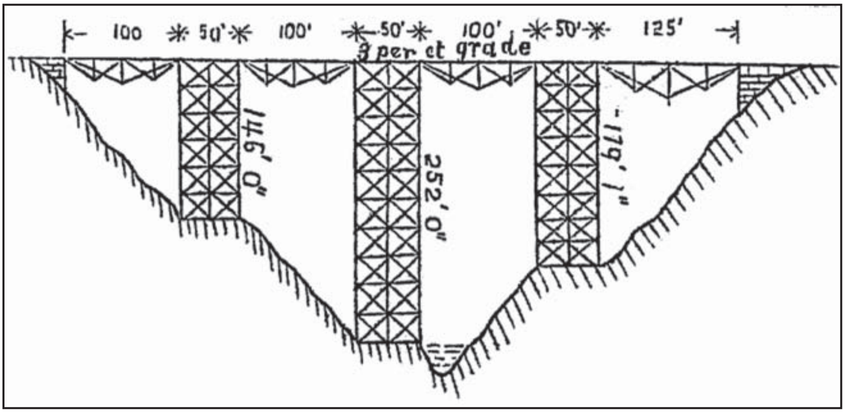


Figure 2. Verrugas Viaduct elevation.

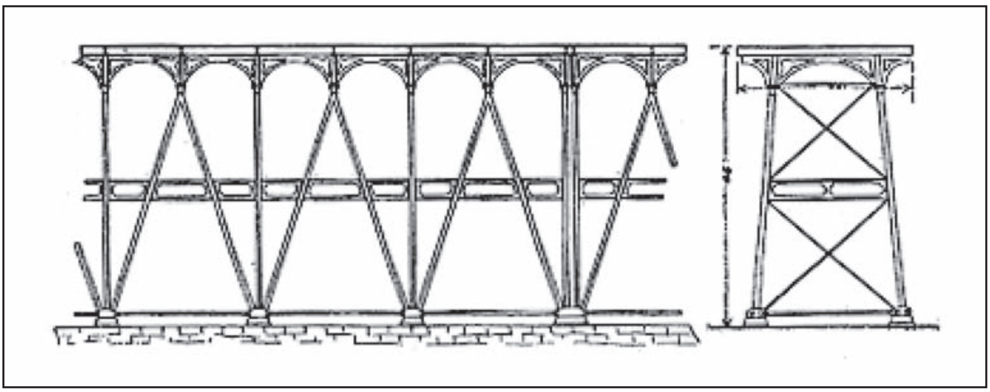


Figure 3. The first iron Viaduct in the U.S. (1853).

It was in 1851 that Bollman designed Carey Street trestle over B&O RR tracks in Baltimore that had iron diagonals connecting successive bents. Albert Fink designed the first cast iron viaducts for the B&O RR in 1852 known as the Buckeye and Tray Run viaducts, and they were built in early 1853 (Figure 3, Greiner, 1891).

3 VERRUGAS VIADUCT

3.1 *Site*

The earth in the vicinity of the bridge was sort of concrete. The westside was the hardest, and was composed of water-washed granite, boulders, cobblestones, and closely packed pebbles. It required blasting for its removal. The eastside was not quite so hard, but had a texture resembling of "blue stone" of which curbs were made in New York City around 1870s. It was very compact, and provided a good foundation in a dry climate at Verrugas. The sides of the gorge were very rugged.

People working in this area were getting sick with a mysterious disease called "Verrugas" which covered the body of its victims with painful blood boils. It was found that this disease was active between the height of 3,000 ft. and 6,600 ft. above sea level. Foreigners to this locality were more susceptible to catching the disease if they stayed for more than 24 hours; and if it was not attended to in a timely manner, it was causing deaths of some patients. Meiggs had provided extensive quarters and accommodation for his staff and workers at great expense so that men got immediate medical attention at the first sign of the disease, but there was no sure cure (Bogue, 1876/7).

Skilled labor was difficult to obtain at this location as no similar bridge and tunnel projects were built in Peru up to that time. The labor turnover was great due to death or injuries. Most of the unskilled workers were Chinese, Chileans, and Peruvian Indians. Between 1860 and 1872, more than 80,000 Chinese workers were shipped from Macau to Peru in 192 vessels (New York Times, 1873a). About 30 percent of these workers died in shipwrecks, or due to sickness or suicides. The best labor that could be made available or used was that of runaway sailors who were good at climbing and rigging, but not skilled as mechanics or ironworkers for erecting bridges.

3.2 *Bridge dimensions*

The bridge had three wrought iron piers, four wrought iron spans, and two stone abutments which were 42 ft. wide. The description of four spans is already given before.

Each pier was composed of three transverse bents with four columns in each bent. Thus, there were 12 columns in each pier. Each column was provided with a cast iron shoe supported on a granite pedestal. The height and area of each pedestal were calculated based on the load and the foundation material. The piers were built up in sections of 25 ft. height, and connected by cast iron joint boxes to which the columns were bolted.

The piers at the grade measured 15 ft. × 50 ft. All 12 column legs battered transversely one half in and one half out, as they descended. The outward batter was 1 in 12, but the inward batter was variable to bring the converging legs together at their feet. Each group of four columns formed an inverted M transversely. Longitudinally, the piers were vertical, holding their size of 50 ft. from top to base. The pier legs were covered by heavy cast iron shoes, planed to a true surface, and were anchored to the rock or base blocks. All bearing surfaces were planed and truly dressed (Engineering, 1872a).

At each joint there was a casting with a tenon on each end. This casting formed the joint connecting the pieces of the column, and intersections of the column with the longitudinal and transverse struts, and the tie rods by which the pier was braced. All horizontal struts consisted of double channels. All columns were proprietary "Phoenix" columns.

The bridge was on a 3 percent grade rising from west to east. However, with the use of shim plates, all wall plates, bridge seats, and roller beds were made level.

3.3 *Erection of the bridge*

The Baltimore Bridge Company had planned to erect the 125 ft. span on the western end using wooden falseworks. It intended to bodily raise the two center spans of 100 ft. each through a height of 252 ft.

This meant that BBC had to erect the 252 ft. high pier first by means of derricks, which would rest on the top of a 25 ft. section, and raise the pieces to their proper places, the derricks being shifted to the top of each section when completed, to serve similarly on the section above. This method would have involved the lowering of each piece to the bottom of the valley, which would then be raised on the pier. Since all the bridge components were delivered by railroad cars at the top of the bridge, this procedure did not make much sense.

Figure 4 shows side elevation of erection work in progress along with cables, temporary span, and temporary bents, at the two abutments, etc. Figure 5 is a transverse view of the top section of one of the piers, showing end view of temporary span (Buck, 1876/7a).

BBC had sent two foremen, Mr. W.H. Tipton and Thomas Flanagan from the U.S., who were employed by Meiggs to manage the erection, with Tipton being in control of the erection. BBC plan of erection was given up, and a new plan developed by the Resident Engineer L. Lefferts Buck was adopted in consultation with Tipton. Buck presented a paper to the American Society of Civil Engineers (ASCE) in December 1875 describing the erection of the Verrugas Bridge (Buck, 1876/7a). The procedure described below is compiled from four different sources (Latrobe, 1873; Cleemann, 1873; Buck (1876/7a and 1876/7b)).

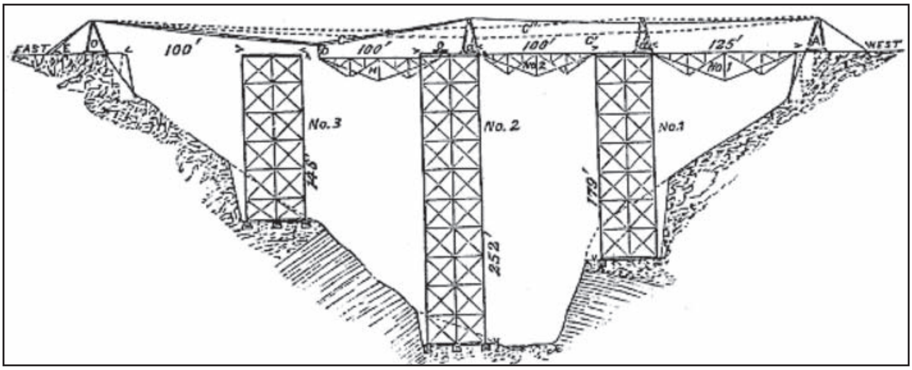


Figure 4. Erection of Piers 2 and 3 and Spans 2, 3, and 4.

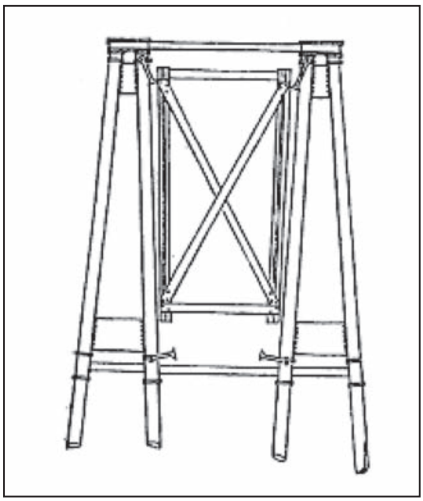


Figure 5. Transverse view of pier.

Two 30 ft. high wooden bents or towers A and D were erected on the road bed beyond the abutments; and between them were suspended two sets of wire cables sufficiently high for the lowest point of the curve to clear the top of the bridge, one set vertically over each truss line. The ends of the cables were secured into the ground. Each cable was supplied with a traveler 'b'.

3.3.1 *Erection of piers*

Two columns with struts and diagonal rods connecting them were put together on the graded roadbed at the western end where the railroad cars delivered the pier components. They were attached to travelers on the cables by means of tackles, so that they could be raised or lowered. Once the columns assemblies were launched into the air, a rope attached to the traveler served to regulate their position so as to be directly over the point of the pier where they were to be lowered by means of the tackle. Connections were made with the other columns so that a 25 ft. high section of pier with 12 Phoenix columns was completed. The same procedure was repeated for each pier so that it reached its design height. The maximum design pressure on the base plates of pier columns was 121 pounds per square inch (Engineering, 1872).

To facilitate unloading and error-free erection, each pier was painted with a different color. Pier 1 was painted red, Pier 2 black, and Pier 3 green. The piers were assembled with screws and nuts to avoid riveting due to the lack of skilled labor. After the initial learning curve, the piers were constructed in 42 days.

3.3.2 *Column stresses*

There were two types of "Phoenix" columns, six-segment and four-segment. The six-segment column had a cross-sectional area of 20 sq.in. and when fully loaded, had a stress of 4,612 pounds per square inch (psi). The four segment column had a cross-sectional area of 13 sq.in. and a design stress of 3,127 psi under full load (Latrobe, 1873).

3.3.3 *Erection of truss spans*

This was the most difficult and tricky operation in erection of the bridge. In the bottom of the gorge where the 252 ft. high middle pier was planned, there was a large quantity of loose material thrown down in making deep cuts in mountainside to the made the roadbed at each end of the bridge. He was concerned about the difficulty of supporting any falsework on this material for the iron truss spans on either side of this middle pier. This prompted him to turn his attention to development of a scheme using a temporary span to erect the three identical 100 ft. span Fink Trusses.

On completion of all three piers, and the 125 ft. span, the cables were strengthened by additional wires to prepare them to support heavier loads. Towers or bents, a′ and a″, were placed on Piers 1 and 2 to reduce the span of the cables as shown in Figure 3. c′ and c″ are the respective positions of the cables. Two temporary or auxiliary Fink trusses made from wood and iron were built to support the permanent iron trusses while they were being put together. H represents the two temporary Fink trusses. They were braced together at such a distance apart so that when the permanent iron trusses were put outside of them, the latter would be in their final position.

The temporary trusses were then attached to the wires at the middle points and at ends, and swung over their intended positions, lowered, and connected to the piers. Thus, they became the falsework for the permanent iron spans.

When the iron trusses had been put together, so as to support themselves, the upper bracing of the middle pier was taken out, and the temporary truss pushed through (suspended from the cables) to serve in a similar manner for the next span, the diagonal bracing of the permanent span being put in as rapidly as the progress of the wooden one left the space clear.

It took only two days to move the wooden span from one opening to the next, and only 16 hours to complete the iron one afterwards.

3.4 *Erection time*

Pier No. 1, 179 ft. in height was erected in 18 days. Pier No. 2, 252 ft. in height in 12 days, and Pier No. 3, 146 ft. in height in 12 days. In the meantime, Span No. 1, 125 ft. in length was erected on scaffolding in 5 days, and two strong wooden Fink trusses to act as temporary trusses were fabricated which were then going to be used for the erection of three 100 ft. spans.

Span No. 2 was raised, and swung into position in 22 hours. The wooden tower was moved from Pier 1 to Pier 2, and Span No. 3 was swung into place in 16-1/2 hours. Span No. 4 between Pier No. 3 and the east abutment was swung into place in 18 hours.

It took few days to lay the floor and the permanent tracks. The total time consumed in the erection of Verrugas Viaduct, including all preparations, was 3-1/2 months. The actual time used in raising operations was 55 working days.

No one was seriously injured. One piece of column broke loose from the strap, fell down, and it was badly damaged. It was taken to Lima for repairs, and reused in such a way as not to have any effect on its structural strength or properties.

3.5 *Weight of the bridge (Table 2)*

The bridge members were made of wrought iron and the joints in compression were made of cast iron. The connections were made using screws and nuts to avoid riveting.

Table 2. Weight of individual bridge components.

Member	Height or length in feet	Weight in pounds		Total weight in pounds	Weight per linear feet
		Wrought iron	Cast iron		
Pier 1	145	250,724	37,112	287,836	1,985
Pier 2	252	442,494	52,261	495,755	1,962
Pier 3	178	295,397	39,338	334,735	1,880
Span 1	125	62,195	7,037	69,232	N.A.
Span 2	100	42,776	7,055	50,831	N.A.
Span 3	100	42,776	7,055	50,831	N.A.
Span 4	100	42,776	7,055	50,831	N.A.
Total		1,182,138	156,913		

Total weight of the bridge = 1,339,051 lbs or 670 U.S. tons (Latrobe, 1873).

3.6 *Cost of the bridge*

The cost was estimated at U.S. $360,000 including fabrication, shipping, sending of 2 foremen to Peru, and labor and other costs (Engineering News, 1889).

3.7 *Opening of the bridge*

A rendering of the bridge with a train on it is shown in Figure 6 (Engineering, 1872). On January 8, 1873, a train pulled by a locomotive named “Matucana” was sent from Lima with a number of excursionists and company officials and it crossed the bridge safely and without any incident. Many officials, not sure of the strength of the viaduct to support a locomotive with railroad cars, decided not to participate. The entire entourage crossed the viaduct again, and made the return trip to Lima safely. President Pardo of Peru accompanied by members of the Peruvian Congress and representatives of foreign nations living in Lima visited the Verrugas Viaduct, and they were treated with great hospitality by John G. Meiggs, brother of Henry Meiggs (Engineering, 1873).

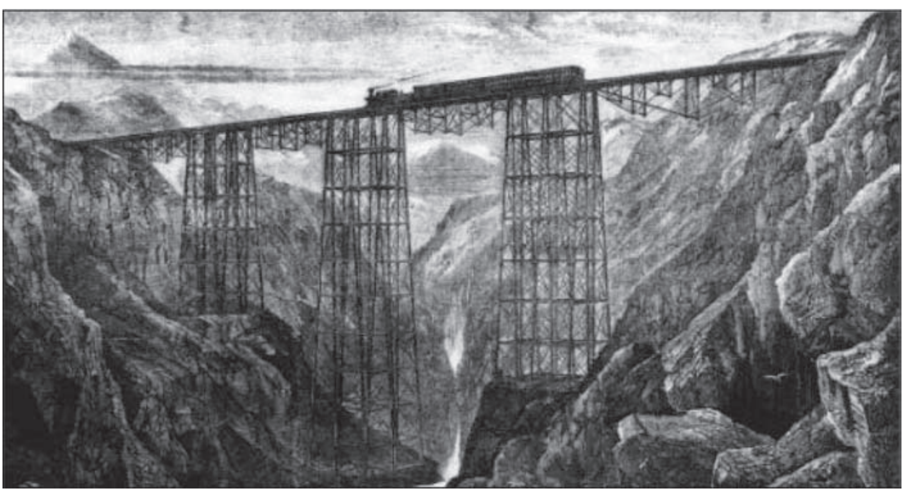


Figure 6. Rendering of completed Verrugas Viaduct.

4 COMPARISON OF VERRUGAS VIADUCT WITH OTHER VIADUCTS

In 1875, Ernest Pontzen, an Austrian Engineer wrote a paper comparing the Verrugas Viaduct with other prominent viaducts of Europe. At that time, the European bridges used riveting to connect truss members. American practice of that period used pins instead of rivets, and then used bolts and screws to make the connection. Field riveting in a remote location such as the Andes Mountains without riveting equipment and trained labor force was almost impossible or extremely costly.

He noted that the workmen that Mr. Buck employed to erect the Verrugas Viaduct were mostly deserted sailors and not blacksmiths or machinists. They were good at climbing and hoisting. Even though the work was at a considerable height on unsteady scaffolding, they were very efficient in putting the bridge together under proper guidance.

Mr. Pontzen cited several examples to demonstrate superiority of pin-connected bridge over riveted and latticed bridges (Pontzen, 1876).

Mr. F. de Garay, a Mexican engineer, read a paper at a meeting of the Societe des Ingenieurs Civils at Paris in 1889, and compared European versus American bridges using Verrugas Viaduct as an example. He said that three 100 ft. spans of Verrugas Viaduct were erected in 16 hours each by a gang of 50 men at a height of 250 ft. above the ground (Engineering, 1891b).

On the same Lima-Oroya Railroad four other bridges of 100 ft. spans were constructed. Two were erected by Englishmen, and the erection lasted over two months. When test load was put on them, one bridge fell to the bottom of the ravine, the second one gave way during the trial and rested on the scaffolding below which was kept as a precaution. The third bridge was a riveted girder bridge of French construction. It took one-month of erection time and passed the load test without any problem. The fourth bridge of Fink Truss System was an American bridge. It took five days of erection time and passed the load test.

The European bridges each weighed 125 tons for a 100 ft. span, the American weighed only 67 tons for the same span.

5 DESTRUCTION OF THE VERRUGAS VIADUCT

The destruction of the Verrugas Viaduct took place on March 23, 1889. The summer season was an especially rainy one in the mountains along the line of the Oroya Railway. As a result of torrential rains on that day, the narrow ravines became raging streams, and tore away loose materials

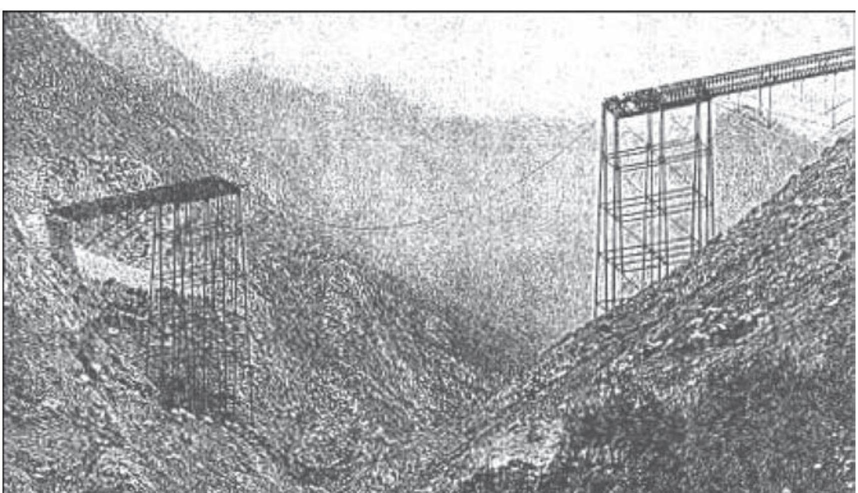


Figure 7. Unsupported rail spanning between Piers 1 and 3 after the accident.

such as trees, stones, boulders, earth, etc. and came rushing down into the main valley of the Rimac River sweeping before them everything they encountered.

This mass estimated at 8,000 tons pushed out the central pier in its path (Engineering and Building Record, 1889). The wall that was built to protect the central pier offered no resistance. The two 100 ft. adjoining spans that were resting on the central pier also collapsed. The central pier and the two spans were found one-fourth mile downstream after the floods subsided. Figure 7 shows the unsupported rail in the form of a catenary spanning between piers 1 and 3 (Engineering, 1891a). The masonry work for the piers was not damaged when the center pier was washed away.

6 BANKRUPTCY OF PERU AND FORMATION OF SYNDICATE OF DEBTORS

In July, 1875, Mr. Meiggs wrote a letter to President Pardo of Peru summarizing the status of his 7 contracts to build railways in Peru stating that because the Peruvian government had not succeeded in raising funds for the railroad construction, it was impossible for him to continue in such a ruinous and disastrous position (Engineering, 1875).

Since 1876, Peru had failed to pay any interest on its external debt to bondholders in England. The war with its neighbor to the south, Chile, between 1879 and 1883, bankrupted the country. Peru also lost about 775 miles of railroads already built by cession of the territory containing them to Chile (Engineering News, 1887b). The railroads were built by the Peruvian government with money obtained by the sale of bonds in England.

A syndicate of bondholders was formed to assume the national debt of Peru in return for huge concessions. The general principle of this arrangement was that Peru was unable to meet its financial obligations, and had agreed to hand over to its foreign creditors certain properties to administer for their own benefit (Engineering News, 1887c). The syndicate included Michael P. Grace of W.R. Grace & Co. of New York and two other Englishmen, Sir H.W. Tyler and G.A. Ollard. Michael Grace's brother William R. Grace was a Mayor of New York from 1881–1882, and a partner in the firm of W.R. Grace & Co. Peru assigned to the bondholders for a term of 66 years some 769 miles of railways, telegraph and telephone facilities so that these bondholders and their representatives may extend and maintain them (Engineering News, 1887a).

The foreign debt of Peru when the bondholders' contract was approved on October 25, 1889 was $160 million. The internal debt consisted of $100 million with interest. The bondholders' contract had nothing to do with the internal debt (New York Times, 1890).

7 RECONSTRUCTION OF THE VERRUGAS VIADUCT

At the time of the collapse of the Verrugas Viaduct, the Oroya Railway was being managed by the Syndicate headed by the Grace brothers—William R. and Michael P. Grace. The latter obtained approval from the bondholders' consulting engineers in England, Messrs. Levesey and Son, to select Leffert L. Buck to design the new replacement bridge because of his reputation and familiarity with the site (Engineering, 1891a).

Buck designed a cantilever bridge with two piers, the end spans acting as anchor spans and the center span as a suspended span. He also used the existing piers to support the new work. In Figure 8 the full lines show the elevation of the new Verrugas Viaduct, and the dotted lines the position of piers of the old viaduct (Engineering News, 1891). The new bridge was symmetrical about the centerline.

7.1 *Bridge, dimensions and details*

The Center span was 235 ft. between piers, the two side spans were 140 ft. each, and the length over piers 30 ft., making a total length of 575 ft. between end pins. The cantilevers were 30 ft. deep at the piers, and 15 ft. at the ends. The bearings for the end pins were anchored to the masonry abutments by four 1-3/4 inch diameter 12 ft. long bolts with anchor plates at the ends. The piers

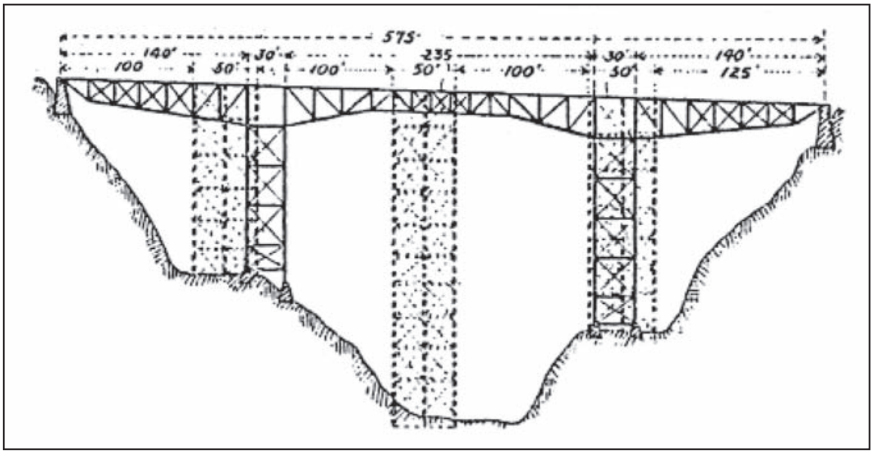


Figure 8. Comparison of piers of new Verrugas Viaduct (solid lines) and old viaduct (dotted line).

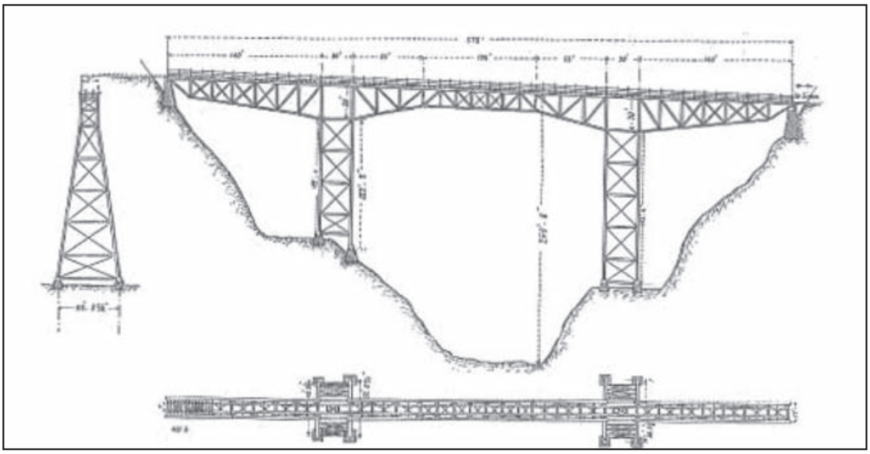


Figure 9. Plan, elevation and transverse view of new Verrugas Viaduct.

consisted of four columns braced together, and were 17 ft. × 30 ft. at the top. The upper piers had two legs of unequal heights (109'-6" and 122'0"), and the lower pier was 142'–6" high. The height from the lowest point of the gorge to the top of the rail was 250 ft. The width was 17 ft. center to center of girders. There was a single track over the bridge, and a handrail was fitted on each side to prevent anyone from accidentally falling down.

Because the railroad was on a 3% grade, the westside (Lima and the coast) was at an elevation of 5826.16 ft. and the Aroyo side (or the eastside) had an elevation of 5,845.41 ft. above sea level.

Figure 9 shows the plan, elevation, and transverse view of the new Verrugas Viaduct (Engineering, 1891b). The details of shore arm with abutment, and of tower with lower arm of cantilever are presented in Figure 10 (Engineering News, 1891).

7.2 *Award of fabrication and erection contracts*

Michael P. Grace invited bids from fabricators in the U.S. based on the drawings prepared by Buck. The fabrication contract was awarded in April 1890 to the New Jersey Steel and Iron Company,

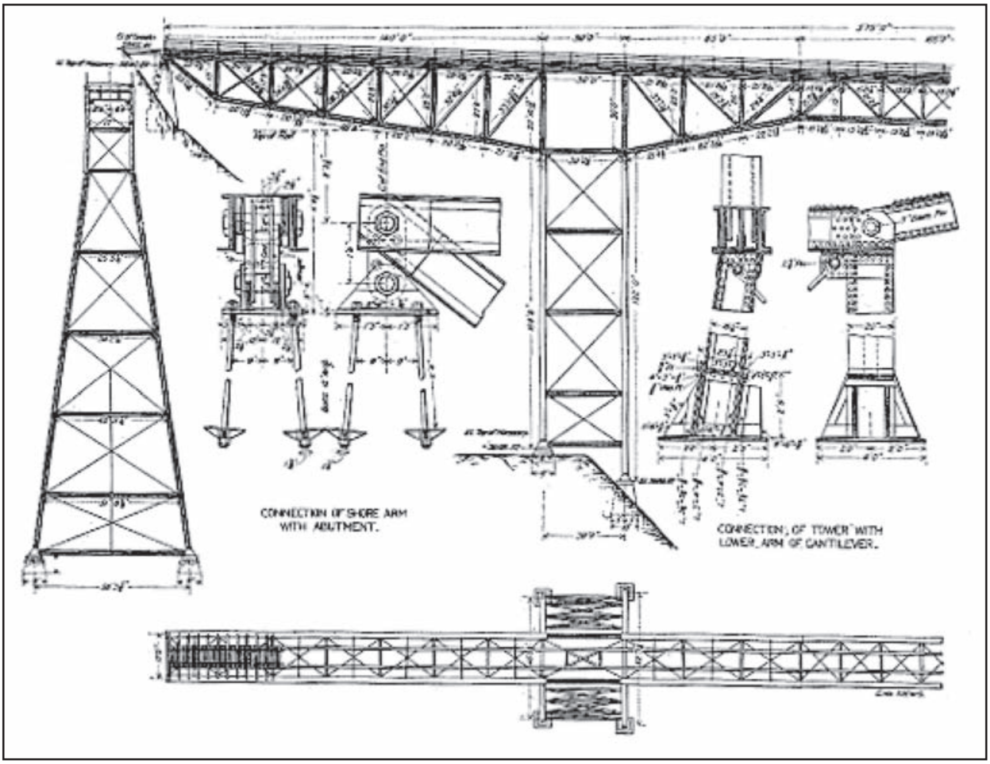


Figure 10. Details of new Verrugas Viaduct.

Trenton, NJ. A separate contract for shipment of the bridge components, and erection of the bridge was awarded to Cooper, Hewitt & Co. of Trenton, NJ (Engineering News, 1890).

7.3 *Bridge erection (Engineering, 1891)*

The first shipment of bridge iron members and plant arrived at the site on July 1, 1890. This new bridge was riveted as opposed to the original bridge which was pin connected and bolted. Cooper, Hewitt & Co. sent from the U.S. riveting and erection crews to assemble and erect the bridge. Additional help was provided by runaway sailors of all nations who were picked up at the port city of Callao, and sent to the Verrugas Viaduct site by train. There was a high turnover of these sailors.

L.L. Buck had sent two of his men to supervise the construction; Albert S. Riffle was Engineer-in-Charge of construction, and H.H. Dougherty, the foreman. Mr. P.A. Frazer acted as the Superintendent of Oroya Railroad, and he represented the owner, Michael P. Grace, at the job site.

Engineering work related to the preparation of new foundations, and erection of new towers, and anchor and suspended spans was supervised by Riffle and Dougherty. There was no cure found for Verrugas sickness or fever in 1890. Riffle stayed on the ground during the period the construction work was going on. Both Riffle and Dougherty took the risk of getting infected by the Verrugas sickness.

Seventeen of the 20 staff members were stricken with the Verrugas sickness, and 9 of these 17 men died. During the six month construction period there were 100 to 200 workers on payroll, and many of them either left or fell victim to the sickness.

The following procedure for bridge erection was adopted.

1. The scaffolding was built between the old piers and the abutments, and the new towers were constructed around the old piers. The pedestals of masonry on which the towers rested were built of granite laid in Portland cement (Figure 11).
2. The old Fink trusses with deck spans were removed, lowered, and disposed of.
3. The shore arms (anchor spans) of the cantilever bridge were then built and anchored to the abutments.
4. Using a traveler on each side, the cantilever arms of the middle span were built towards the center of the bridge (Figure 12), and the final connection made (Figure 13).

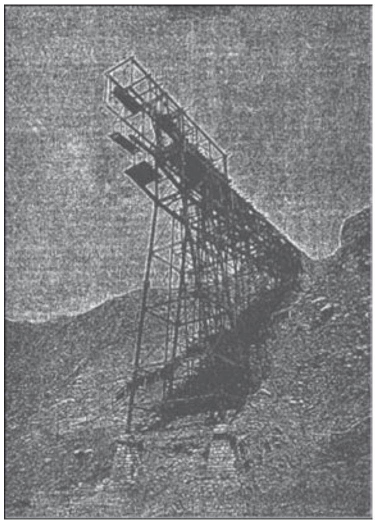


Figure 11. View showing newly built west pier and use of traveler to build out cantilever span.

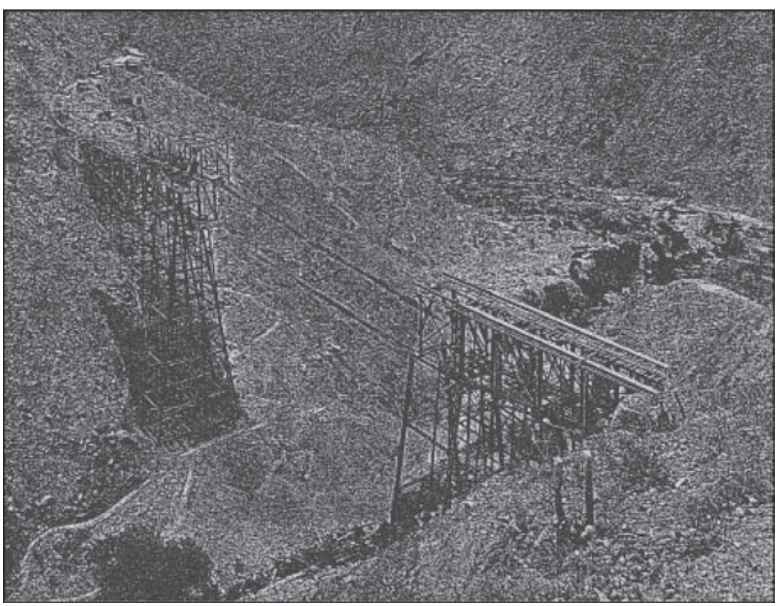


Figure 12. Aerial view of on-going new construction of cantilever span on Lima side (left).

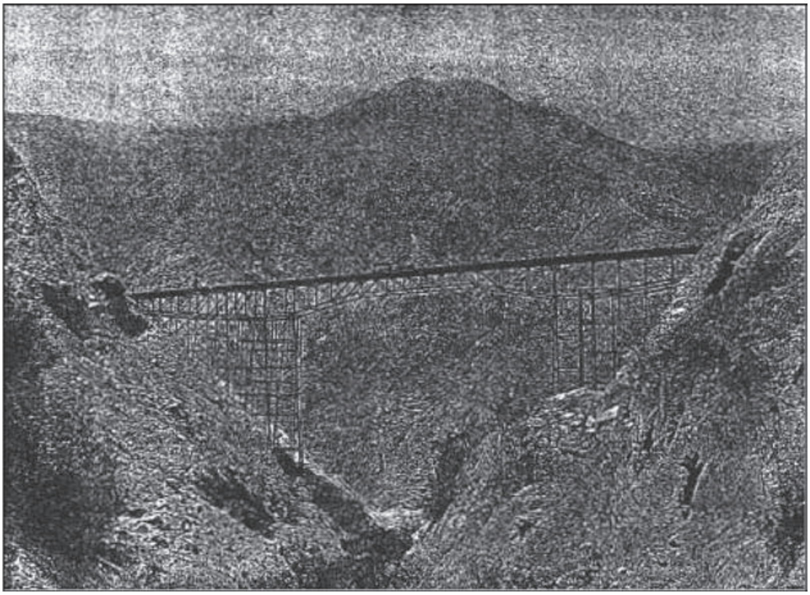


Figure 13. View of the new viaduct, completed January 1, 1891.

7.4 *Erection time (Engineering, 1891)*

The erection time is summarized below:

1.	Erection of false work or scaffolding	16 days
2.	Erection of iron work for east pier	16 days
3.	Erection of iron work for west pier	16 days
4.	Transferring of iron work of half bridge across the gorge	17 days
5.	Demolition of the west span of the old bridge	5 days
6.	Erection new west span	11 days
7.	Demolition of the east span of old bridge	3 days
8.	Erection of new east span	10 days
9.	Erection of west half cantilever and center	13 days
10.	Erection of east half of cantilever and center span	7 days
		114 days

7.5 *Weight of the bridge (Engineering, 1891)*

The total weight of the bridge was about 700 U.S. tons.

7.6 *Construction cost (Engineering News, 1889)*

The construction cost of the bridge was about U.S. $2 million.

7.7 *Load test and bridge opening (Engineering, 1891)*

The test train had 3 engines and 10 American cars of rails and other construction materials, weighing about 350 tons. This train ran over the bridge at 20 miles an hour, and the deflection of the center span was measured at 1-¼ inches. This was less than L/2000 (or 1.41 inches). The bridge was opened to traffic on January 1, 1991.

8 PEOPLE CONNECTED WITH VERRUGAS VIADUCT

8.1 *Henry Meiggs*

Henry Meiggs was born in Catskill, Green County, New York State in 1811. At age 20, he went to Boston and made some money by speculation in the lumber business. He came to New York in 1835, made a lot of money in the same business. However, in the financial panic of 1837, he lost his fortune. He recovered a year later, but then again became insolvent in 1842. When gold was discovered in California, he and his brother John loaded a ship with lumber, travelled around the Cape, and arrived in San Francisco in July of 1849. He sold his lumber for a profit of $50,000.

He prospered in the lumber business, but the financial crisis of 1854 ruined him financially. He committed forgeries of over $900,000, and realizing that he will be caught eventually, he sailed with his family to Chile in South America. In 1861, he took a contract to build 33 miles of railroad on Valparaiso and Santiago Railroad, with a rise of 4,800 ft. and completed the work in three years (instead of four) for $12,000,000 and earned a profit of $1,320,000. He stayed in Santiago, Chile until 1867, and then relocated to Lima, Peru.

In 1868, Henry Meiggs undertook the study for the construction of the Callao, Lima, Oroya railroad with the help of a Polish engineer Ernest Malinowski who proposed the route to be taken

Table 3. Total cost of railroad projects.

Name of railroad	Length	Completion date	Cost
Mollendo-Arequipa	107 miles	1870	$11,250,000
Callao-Oroya	136 miles	1876	$25,875,000
Ilo-Moquegua	63 miles	1872	$6,781,250
Arequipo-Puno	222 miles	1873	$30,000,000
Pacasmayo-Cajamarca	83 miles	1873	$6,656,250
Chimbote-Huaraz	172 miles	1876	$22,500,000
Puno-Cuzco	230 miles	1874	$23,437,500
	1,013 miles		$126,000,000

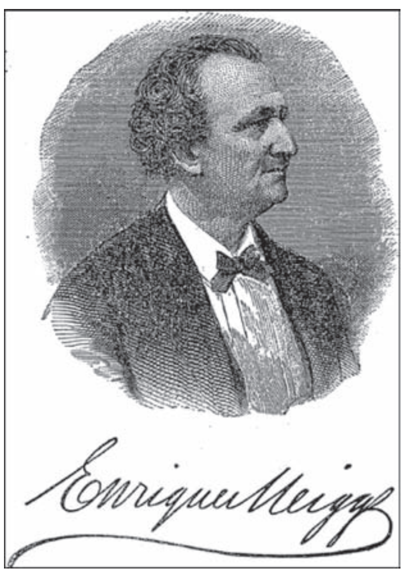


Figure 14. Henry Meiggs (Engineering and Mining Journal, 1878).

through the Andes Mountains for a distance of about 136 miles. On this route was located the Verrugas Viaduct. On December 18, 1869, the Peruvian Government signed the contract with Henry Meiggs for U.S. $25,875,000, to build one of the most difficult and challenging engineering works.

Meiggs had $126 million in contracts with the Peruvian government to build 1,013 miles of railroad on seven lines as follows (Engineering, 1874):

At the peak of Andean railroad construction, it is said that Meiggs had 14,000 workers, and he was probably the second biggest employer of the U.S. engineers after the U.S. Army, and the largest foreign purchaser of American railroad materials (Railroad Gazette, 1877). In 1875, the government ran out of money, and stopped paying interest on its bonds. This virtually stopped all of Meiggs' operations.

In his desperation to generate cash, he committed forgeries which initially helped him, but in the end he was caught. He suffered paralytic strokes, and it is believed that they caused his death. He died in Lima on September 30, 1877 (New York Times, 1877a).

8.2 *William H. Cilley*

He was Meiggs' most loyal and trusted employee, and second in command to build the Aroyo railroad. He was born in Northfield, New Hampshire on May 26, 1839. At a young age, he joined one of the railroads in New Hampshire and rose quickly from a fireman, engineer, roadmaster, to chief of traffic. Then, he joined Boston Gas Works and rose to the position of Manager. Attracted by opportunities in the west, he moved to California where he met Henry Meiggs for the first time, and learned mining operations of California. He returned to Boston with a modest fortune, but lost it due to poor investments.

In 1861 Meiggs signed a contract with the Government of Chile to build a railroad from Valparaiso to Santiago in four years. A short link of Valparaiso to Quillota was completed in about a year. In 1862, Meiggs wrote a letter requesting Cilley to join him. His contract called for the completion of Quillota to Santiago road in 3 years with a bonus of $10,000 per month for early completion, and like penalty for delay. Meiggs and Cilley completed the project in two years.

Based on Meiggs' success in building mountain railroads in Chile, Col. Balta, President of Peru, invited him to undertake construction of Arequipa and Orya railroads—the two most difficult and daring engineering works ever undertaken in the history of railroad construction. Meiggs again called Cilley, who was working on a small railroad in the U.S., to come to Peru to join him, and run the operation with full control.

In his role as Superintendent of construction, Cilley hired some of the best, brightest, and enthusiastic American engineers who upon their return to the U.S. were instrumental in building railroads and major bridges, and opening up the western U.S. for development.

Some of the engineers that Cilley hired were Roswell E. Briggs, who later became Chief Engineer of Denver and Rio Grande Railroad; Virgil G. Bogue who was consultant for the foundation of the Brooklyn tower of the Williamsburg Bridge, and later became the Chief Engineer of Union Pacific; Leffert L. Buck, who was a classmate of both Brigss and Bogue at RPI, and later reconstructed Roebling's Railway Suspension Bridge over Niagara Gorge and became Chief Bridge Engineer of New York City and designed the Williamsburg Bridge; Othniel F. Nichols, who was also an RPI graduate, later became Chief Bridge Engineer of New York City from 1904–1906 and consulting engineer thereafter until his death in 1908, and was responsible for the design and construction of the Manhattan and Queensboro Bridges.

Cilley was a pallbearer at Henry Meiggs' funeral, and one of the executors of his will (New York Times, 1877b). He was for 27 years prominently connected with building of railroads in South America, and generated goodwill and respect for the U.S. business and technical acumen. He died in October 1889, and was buried at his home in Leominster, Massachusetts (Railroad Gazette, 1889b).

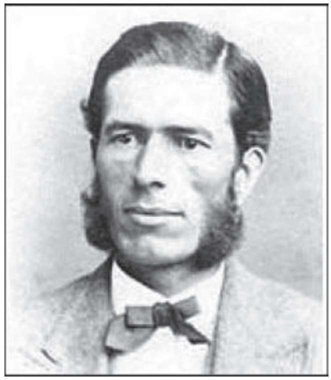


Figure 15. Leffert L. Buck (Circa 1875). (Photograph Collection, Institute Archives & Special Collections, RPI, Troy, NY 12180.)

8.3 *Leffert L. Buck (Nason and Young, p. 386)*

Leffert Lefferts Buck was the resident engineer, and one of the first to use principle of suspension bridge in erection of the Verrugas Viaduct where it was difficult or impractical to use false work (New York Times, 1909). He was born in Canton, New York in 1837. Before completing high school, he served an apprenticeship to the machinist's trade. In 1859 he entered St. Lawrence University but left two years later at the outbreak of the civil war to enlist as a private in the 60th New York State Volunteers unit, and actively fought in the war. For his bravery, he was promoted to the rank of captain. Following his release from the army, he joined Rensselaer Polytechnic Institute in Troy, New York, as a sophomore, and graduated in the class of 1868 (Engineering News, 1909).

He gained some experience running the first Adirondacks railway, and went to Peru, South America where he built a difficult tunnel section of the Lima & Oroya Railroad and constructed the Verrugas Viaduct on that line.

He returned to the U.S. in 1873 and joined the mechanical department of the Illinois Central Railroad. In 1888, he prepared plans for sinking pumping wells of Erie Dry Dock at Brooklyn Navy Yard. He was selected to rehabilitate the Roebling's first railway suspension bridge over the Niagara Gorge (Gandhi, 2006).

In 1891, he was appointed by the President of Peru, a Commissioner to represent Peru in the engineering congress at Washington, D.C., convened to study plans for an international railroad to connect North and South America (Engineering Record, 1891).

In 1895, he was appointed Chief Engineer of the Williamsburg Bridge. The bridge, when built in 1903, was exceeded in size only by the Firth of Forth Cantilever bridge in Scotland. He died on July 17, 1909 at his home in Hastings, NY due to apoplexy.

8.4 *Anthony Walton White (W.W.) Evans (Nason and Young, pp. 213–214)*

He was responsible for the design of the Verrugas Bridge as a Consulting Engineer to the Peruvian Government. He was born in New Brunswick, New Jersey in 1817. He dropped "Anthony" from his name early in life. After attending local schools and Rutgers College he entered Rensselaer Polytechnic Institute in Troy, New York in 1834, and graduated with a degree of Civil Engineering in 1836. He joined the Erie Canal project as an Assistant Engineer upon his graduation. In 1845 he became Assistant to Allan Campbell to building an extension of the New York and Harlem Railroad to Albany. In 1850, he resigned to join Campbell in building the Copiapo Railroad in Chile, South America, which he completed in 1853. While in Chile, he built the first pier on the coast of South America.

Figure 16. Walton White Evans (Circa 1875). (Photograph Collection, Institute Archives & Special Collections, RPI, Troy, NY 12180.)

In 1853, he moved to Peru as Chief Engineer of Arica and Tacua Railroad, and completed the construction in 1856. He returned to the U.S. in 1856, got married, and went back to Chile with his wife in the fall of 1856 to build 50 miles of Southern Railroad which was completed in 1860. He then spent two years in visiting, studying, and examining public works, and opened an office as a consulting engineer in New York.

In 1862, he was appointed engineer of harbor defenses of the port of New York, and stayed in that position until the close of the civil war. In 1866, he went to England, Germany, and Russia to see and report on the railways and qualities of iron used.

For the Lima and Oroya Railroad, he was Inspecting or Consulting Engineer to the Government of Peru, and assisted Meiggs and Baltimore Bridge Company in the design and fabrication of the Verrugas Viaduct. In the 1860s and 1870s, European engines and rolling stock were considered superior to their American counterparts. He prodded foreign governments to buy American engines and rolling stock, and compare their performance using controlled tests. Using these tests results, Evans proved that the European products were overrated and did not perform as claimed (Evans, 1870 and 1877). He acted as agent for a number of foreign railways to purchase equipment and recruit staff; and in that capacity he promoted American products and engineers. Up to 1878, he was supplying, designing, and exporting a large quantity of railway plant and iron bridges to Peru, Australia, New Zealand, and Mexico. He died at his home in New Rochelle, New York on November 28, 1886.

8.5 *Charles H. Latrobe*

He was the design engineer for the Baltimore Bridge Company, the fabricator of the Verrugas Viaduct. He worked very closely with Evans in developing several truss and pier systems and preparing their cost estimates. He was born in Baltimore in 1833. His father Benjamin H. Latrobe was the first person to survey the Allegheny Mountains to establish a railroad route; and a member of the committee that reviewed John Roebling's scheme to built the Brooklyn Bridge and approved it, which made it possible for Roebling to obtain funding for the bridge. His grandfather rebuilt the Capitol in Washington, D.C. after its destruction by the British.

Charles Latrobe designed beautiful bridges spanning Jones Falls and for grade-elimination in Baltimore. The Baltimore Bridge Co. also fabricated all parts, assembled them into a viaduct structure, and disassembled for shipping to Peru. When Latrobe read about the destruction of the Verrugas Viaduct, he sent a letter to the Engineering and Building Record, and one of his comments was, "What the Revolutionists tried in vain to blow up, the water, in a rainless region, has carried away…" (Latrobe, 1889). He died in Baltimore on September 19, 1902 (New York Times, 1902).

8.6 *Thomas M. Cleeman (Nason and Young, p. 360)*

He was born in Philadelphia in 1843. He graduated from Rensselaer Polytechnic Institute in Troy, NY with a Civil Engineering degree in 1865. After graduating, he joined Pennsylvania Railroad as Assistant Engineer. After working for a few years, he became Assistant Engineer for the Allegheny Valley Railroad in Pennsylvania. Then he moved to Peru, South America. During the construction of the Verrugas Viaduct, he was the Division Engineer of the Callao, Lima and Oroya Railroad, and responsible for the layout of the route which was then built by thousands of workers employed by Meiggs. In 1873 he returned to Philadelphia. He was the author of the "Railroad Engineers' Practice" which went through several editions. He also taught the Railroad Engineering and Route Survey course at RPI in 1892. He died at Guayaquil, Equador, S.A. in 1893.

8.7 *Ernest Malinowski*

He was born in Poland in 1818 and came to Peru in 1852 to work on public works projects. He was instrumental in establishing the feasibility of the Lima-Oroya route. In 1868, when Henry Meiggs undertook the study for the construction of the Oroya Railroad, he sought assistance from Malinowski to lay out the proposed 136 mile route through the Andean Mountains. A commission of engineers, that included Federico Blume, Felip Arancivia, and Walter F. Morris, approved the project. In December of 1869, Meiggs signed a contract with the Peruvian government to build the Oroya Railroad for $27,600,000 in six years starting on January 1, 1870 and delivering the completed railroad on or before January 1, 1876. Malinowski died in Lima in 1899.

9 CONCLUSIONS

1. There was a lot of goodwill for American engineers and know-how in South America in the late 19th century.
2. When the railroad building program started in Peru in the 1860s, England and France were the major suppliers of locomotives, rolling stock, rails, bridges, and other supplies. However, with documented superior performance of comparable American products, all these items were purchased in the U.S. and shipped to South America, thus helping and developing the American economy. Most of the credit goes to engineers like W.W. Evans and American manufacturers and suppliers for maintaining very high standards.
3. It is amazing how the Verrugas bridge was designed and fabricated so that no part weighed more than 300 pounds. It was fabricated at Phoenixville, Penn; all parts were assembled on the ground in Baltimore, their positions numbered and the parts of the three piers color-coded; and shipped from New York around the Horn. The shipment arrived at the port of Callao in Peru, and it was put on a train for delivery near the bridge site. The bridge was built precisely as designed, thousands of miles away with mostly untrained labor.
4. These South American railway projects in mountainous countries were instrumental in training American engineers to take more responsible positions in building America after they returned home. For example, Virgil G. Bogue became Chief Engineer of Union Pacific, Leffert L. Buck and Othniel F. Nichols became Chief Bridge Engineers of New York City. Buck designed the reconstruction of Roebling's Niagara Railway Suspension Bridge, and the new Williamsburg Bridge in New York City. Nichols was in charge of Manhattan and Queensboro Bridges in New York from 1904 to1906 as Chief Bridge Engineer. Roswell E. Briggs became the Chief Engineer of the Denver & Rio Grande Railroad.

ACKNOWLEDGEMENTS

The author thanks Brenda Hill and Issa Purdue of his office for their help in performing research and preparation of this paper. He also acknowledges assistance received from Amy Rupert, Christopher Bassett and Irving Stephens of Rensselaer Polytechnic Institute, Folsom Library.

REFERENCES

Bogue, Virgil G., *"On the Erection of the Verrugas Viaduct"*, Trans. ASCE, V. 5, 1876/7, pp. 240–242.
Buck, L. Lefferts, 1876/7a, *"Erection of the Verrugas Bridge",* Trans. ASCE, V. 5, 1876/7-a, pp. 103–106.
Buck, L. Lefferts, 1876/7b, *"Closing Remarks",* Trans. ASCE, V. 5, 1876/7-b, pp. 242–243.
Cleeman, Thomas M., *"Peruvian Railways",* Engineering, V. 17, February 13, 1874, p. 126.
Engineering, 1872, *"The Verrugas Viaduct",* V.13, April 12, 1872, p. 247 and renderings.
Engineering, 1873, *"Peruvian Railways"*, V. 15, Jan. 17, 1873, p. 39.
Engineering, 1874, *"Peruvian Railways and Public Works"*, V. 17, Feb. 20, 1874, pp. 145–146.
Engineering, 1875, *"Peruvian Railways"*, V. 20, Aug. 6, 1875, p. 122.
Engineering, 1891a, *"The New Verrugas Viaduct"*, V. 51, April 17, 1891, pp. 460–461 and photos.
Engineering, 1891b, *"European v. American bridges"*, V. 51, April 24, 1891, p. 495.
Engineering and Building Record, 1889, *"The destruction of the Verrugas Viaduct"*, V. 20, July 13, 1889, pp. 88–90.
Engineering and Mining Journal, 1878, *"The Cerro De Pasco silver mines and the Callao, Lima, and Oroya Railway of Peru"*, Dec. 21, 1878, pp. 435–438.
Engineering News, 1887a, *"Peru's debt to be removed"*, V. 18, July 30, 1887, p. 81.
Engineering News, 1887b, *"Peruvian Railroads"*, V. 18, August 20, 1887, p. 134.
Engineering News, 1887c, *"The Grace Contract in Peru"*, V. 18, December 24, 1887, p. 451.
Engineering News, 1889, *"The Verrugas Viaduct"*, V. 22, July 27, 1889, pp. 78–79.
Engineering News, 1890, *"The Verrugas Viaduct"*, V. 24, December 27, 1890, p. 581.
Engineering News, 1891, *"The Verrugas Viaduct"*, V. 25, May 9, 1891, p. 441.
Engineering News, 1909, *"Leffert Lefferts Buck",* V. 62, July 22, 1909, p. 107.
Engineering Record, 1891, *"L.L. Buck"*, V. 23, Jan. 17, 1891, p. 104.
Evans, W.W., 1870, *"American locomotives and rolling stock"*, Engineering, V. 9, February 4, 1870, pp. 84–85.
Evans, W.W., 1877, *"American and Fairlie Engines"*, Railroad Gazette, V. 9, November 30, 1877, p. 528.
Gandhi, Kirti, *"Roebling's Railway Suspension Bridge over Niagara Gorge"*, Fifth International Cable-Supported Bridge Operators' Conference in New York City, August 28–29, 2006.
Greiner, J.E., *"The American Railway Viaduct—Its origin and evolution*", Engineering News, V. 25, June 6, 1891, p. 536.
Latrobe, C.H., 1873, *"The Agua de Varrugas Viaduct",* Railroad Gazette, V. 5, April 5, 1873, pp. 135–136.
Latrobe, C.H., 1889, *"The destruction of the Verrugas Viaduct"*, The Engineering and Building Record V. 19, April 6, 1889, p. 245.
Nason, H.B. and Young, W.H. (1887), *"Biographical records of the officers and graduates of the Rensselaer Polytechnic Institute, 1824–1886".* Benjamin Franklin Greene, D.H. Jones and Co.: Troy, NY (1885). RPI Archives and Special Collections, Troy, NY.
New York Times, 1873a, *"Peru"*, May 20, 1873.
New York Times, 1873b, *"A Mountain Railroad",* December 6, 1873.
New York Times, 1877a, *"Obituary—Henry Meiggs"*, October 12, 1877.
New York Times, 1877b, *"The Will of Henry Meiggs"*, November 4, 1877.
New York Times, 1890, *"Peru and her interests"*, June 15, 1890.
New York Times, 1902, *"Death List of a Day—Charles H. Latrobe"*, September 20, 1902.
New York Times, 1909, *"Leffert L. Buck, Bridgemaker, Dead"*, July 18, 1909.
Pontzen, Ernest, *"The Verrugas Viaduct compared with several other viaducts",* Railroad Gazette, V. 8, May 12, 1876, pp. 201–203. (Translated from German from the Journal of the Austrian Society of Engineers and Architects, No. 14, 1875.)
Railroad Gazette, 1873, *"Railway Engineering in the Cordilleras"*, V. 5, July 26, 1873, pp. 298–299.
Railroad Gazette, 1877, *"Henry Meiggs",* V. 9, April 27, 1877, p. 192.
Railroad Gazette, 1889, *"William H. Cilley"*, V. 21, October 18, 1889, pp. 679–680.
Rand, Charles S., and Owen, Albert Kimley, "*The Railroads of Peru"*, published by Printing Establishment, No. 66, Calle de Junin, Lima, 1873, 71 pages, p. 43.

Chapter 43

Design and erection of four signature urban bridges recently built in Spain

J.J. Arenas de Pablo, G. Capellán Miguel, E. Merino Rasillo & M. Sacristán Montesinos
Arenas & Associates, Santander, Spain

ABSTRACT: Four signature urban bridges designed by Arenas & Associates have been built in the last years in the Spanish cities of Madrid, Logroño, Santander and Zaragoza. Every design has special architectural and urban values and all of them were developed searching for new structural and aesthetical solutions to solve each different kind of crossing. The four designs, striking and elegant, constitute iconic gates to the cities where they are located. Our experience as authors of the conceptual and detailed design and construction managers of the four bridges is summarized in this article on these challenging structures.

1 INTRODUCTION

Las Rozas Bridge, in the outskirts of Madrid (with a span of 102 m), and Science and Technology Park of Cantabria Bridge, in Santander (with a span of 72 m), are both asymmetrical cable stayed bridges with a central plan of main stays and a deck over 20 m wide and reduced depth to allow vertical clearance on their crossing over busy highways.

Despite of these common characteristics they have important global appearance and detail differences and they are both meant to become iconic gates of entrance to the City of Las Rozas and to the new Technology Park because of its quality conceptual design.

La Cava Footbridge in Logroño crosses the main entrance road to the city with a main span of 61 m. The structural scheme is a spatial truss with a curved closed cross section and variable width and height. The main span is glazed for protecting users from both traffic noise and disturbances and weather conditions.

Delicias Footbridge in Zaragoza is a light self-anchored curved hanging structure with a main span of 90 m over an important traffic joint next to city's high speed train station. The steel mast is inclined towards the deck and anchored with two cable backstays to a buried counterweight.

Both La Cava and Delicias footbridges constitute iconic gates to the cities where they are located.

Our experience as authors of the conceptual and detailed design and construction managers of these four bridges is summarized in this article on these challenging structures. Engineering ethics and aesthetics compels us to limit unjustified costs to create affordable quality urban environments.

2 LAS ROZAS BRIDGE

2.1 *Location and design*

This contest awarded design, will become the main access to the city of Las Rozas, overpassing the A-6 highway 20 km north of Madrid. Las Rozas is a 68,000 people city next to Madrid, important both as a dormitory town and as a business and industrial area associated to the highway. The crossing is skewed 20º to the traffic, which together with possible future enlargement of the road, results in a 102 m span. Asymmetric cable stayed bridge scheme is chosen because of deck depth restrictions, and the will for the structure to become a symbol for the city, meant to outstand in a semi industrial environment with tall commercial signs and office buildings. Also the curved profile in elevation and steep and asymmetric slope (up to 8%) in access ramps made us choose this typology, when other typologies like bowstring arches did not suit well these limiting factors. The visible mast structure is placed opposite to Las Rozas, pointing the city entrance (Figure 1).

As a result to this initial approach, the steel mast of the cables stayed bridge is set 39 m high, and with a double central plan of 9 cable stays it supports the deck composite section, 22 m wide. Its design includes two triangular cells with rigid steel back stays anchored to a buried counter-weight, and a triangular key element as the anchorage for cable stays in the top of the mast (Arenas et al. 2008).

2.2 *Geometry of the structure*

The bridge transversal section includes two lanes of traffic on each sense, and a wide central walk-way (Figure 2). The deck central suspension is made throug two vertical plans of stays separated 72 cm. The central walkway allows the users to be protected from the disturbances of traffic of the highway, and to walk next to the stays on a wooden deck 5 m wide. The structural cross section is a composite section with 150 cm height steel central girder, and a 22 cm concrete slab supported on transversal ribs with a curved profile.

Nine groups of two stays suspend the deck every 9 m (Figure 3). Each multi-strand stay formed by 31 units of 0.6" wires, with three layers of protection against corrosion. Double plan of stays

Figure 1. View of Las Rozas Bridge from the A-6 highway.

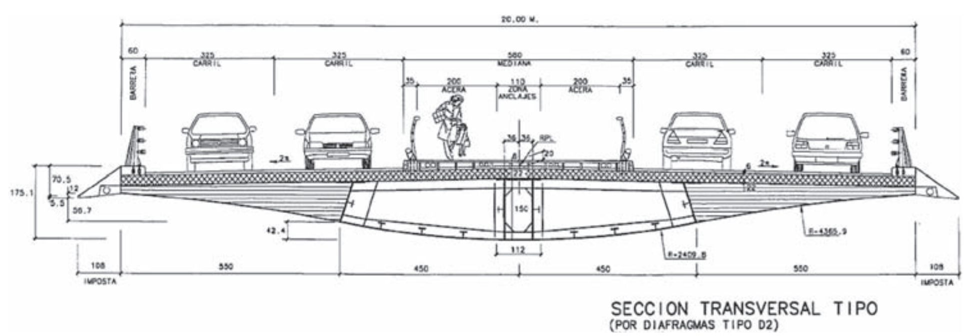


Figure 2. Transversal section of the bridge.

Figure 3. Night view of Las Rozas bridge.

allow limiting maximum size of the elements to these dimensions, with interesting constructive advantages. Stays are hinged at both ends with terminal elements designed to enable jacking and tensioning of the stays. All stays are anchored on two triangular steel sheets of 60 mm at the top of the mast, as the key element of the design and structure.

The mast develops downwards from here including the inclined struts and counter stays in a pyramidal configuration. Rigid steels counter stays help to stiffen the bridge behavior and improve its deformational and dynamic response under vibration. Each inclined lateral plan of the mast formed by one strut and counter stay is closed by a horizontal element to form a triangular cell, to reduce horizontal reaction in the front support and back anchorage, balancing them within the closed rigid cell. Main structure of the mast is steel, but almost half the height of the struts is concrete filled to reduce sheet thickness and to improve its performance. Both counter stays are anchored to the buried counterweight in this abutment through pre-stressed bars to face up to 60,000 kN on each anchorage. Counter weight is materialized through earth filled concrete cells in this abutment forming an orthogonal grid.

Final skewed abutment receives lateral ramps, and has to solve through its design this geometrical condition, at the same time it solves gently the transition to deck section. The deck is simply supported on both abutments through elastomeric bearings, also in horizontal direction in the counterweight abutment in order to receive the horizontal component of the stays' force, compressing the deck against the abutment.

Lighting of the finished bridge serves two objectives, functional light for traffic and walkways, and artistic lighting of the mast and stays. Lamp posts are avoided to eliminate visual interference with the structure, so all functional lighting is made from lateral barriers and balustrades.

2.3 *Method of erection*

Main activities of erection procedure are the positioning of steel structure of the deck and mast using cranes, and positioning and tensioning of the stays.

Foundations are direct footings buried 2 m on existing ground level, with 12 × 12 m inclined footing on each of the mast struts. Counter weight abutment includes pre-stress bars families in all three directions in the anchorage of counter stays.

Steel structure of the composite deck is erected in portions using cranes and positioned on temporary supports every 21 m average, respecting the existing highway and lateral service roads (Figure 4).

Concrete slab is poured over prefabricated plates of 7 cm, without any scaffolding.

Figure 4. View of the steel deck during construction, and end abutment.

Figure 5. Erection of the mast using cranes with one temporary support.

Figure 6. Tensioning of the stays using isotension system.

Each half mast, including one triangular cell and one of the top anchorages is lifted using three cranes (Figure 5). Each one of these pieces weighed over 2500 kN and was rotated and positioned using just one temporary support in only one operation.

Tensioning of the stays takes place after its positioning with a very low initial force. Tensioning is done with two unitary jacks applied simultaneously on both stays in one pair, using isotension system that ensures having the same stress in all units of the cable all through the process (Figure 6). Because of time restraints, tensioning takes place in only one sequence. Construction stage analysis is reproduced backwards from final state, to find tensioning forces of each stay to ensure achieving this final permanent state of charges and geometry. Tensioning took place successfully with no need of a second sequence, as it showed the instrumentation extensometers in steel structure and load cells in stays, which allowed following real time what happened during the process.

3 SCIENCE AND TECHNOLOGY PARK OF CANTABRIA BRIDGE (SANTANDER)

3.1 *Location and design*

Located in the North coast of Spain, Santander, with about 180,000 inhabitants, is the capital of Cantabria, and an important port area. The new Science and Technology Park of Cantabria (PCTCAN) is being built on the city entrance next to the highway S-20, and will be working by 2009 meaning a major thrust for research and industrial development in the region. The new bridge is an overpass allowing communication between both sides of the highway and constitutes an important access point for the Technology Park and the surrounding areas. Also the new bridge is meant to become an icon to identify the PCTCAN and to signal the city entrance.

The highway width and prevision of future enlargements asks for a main span over 60 m. The asymmetric cable stayed typology suits well the site conditions, allowing to reduce deck depth to the minimum, at the same time the inclined mast serves as the desired sign pointing the entrance of the PCTCAN. The resulting design is a cable stayed bridge with 72 m span, and counter stays anchored to a buried counter weight (Figure 7). The mast is about 30 m high from deck level and is placed opposite to the PCTCAN, materializing a technologic and symbolic gate to Santander.

3.2 *Geometry of the structure*

The main span of 72 m is suspended from the mast through 9 stays placed in a central plan every 6 m. In 66 m of the main span, the deck is a composite section with steel structure 150 cm deep with a concrete slab of 22 cm over steel ribs every 3 m. The typical section is 22 m wide with four lanes of traffic of 3.5 m, two sidewalks 2.4 m wide, and central reservation of 2.4 m (Figure 8).

Figure 7. View of the finished Science and Technology Park Bridge (Santander).

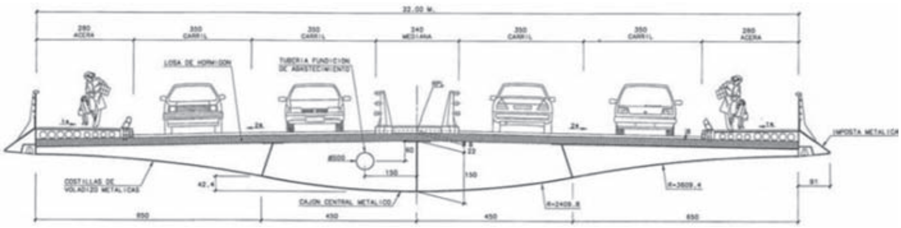


Figure 8. Typical cross section of PCTCAN Bridge (Santander).

Figure 9. Perspective of the main span and inclined mast.

The deck is fixed through a 6 m concrete cantilever to the counter weight abutment with a triangular shape in plan, with a curved vertex in the mast support opened towards the counter stays anchorage to embrace the buried counterweight.

Two lateral plans of 6 counter stays develop downward from the mast head, and balance the horizontal force of the 9 front stays. Each multi-strand stay formed by a maximum of 37 units of 0.6” wires, with three layers of protection against corrosion. Tensioning of the front stays is made from inside the mast head with classic stay adjustable anchorage, while the counter stays are tensioned from underground chambers inside the counterweight abutment, which allow future inspections or maintenance. Fixed anchorages are hinged with a reduced sized design.

Composite section of the deck is fixed to the concrete counter weight abutment using prestressed bars hidden inside the main girder. This way the abutment receives not only the compression force of the deck from the horizontal component of the stays’ force, but also the bending force transmitted towards the mast support in which the deck is fixed. The deck is simply supported on the opposite end abutment on elastomeric bearings.

The inclined mast is the main element of the structure design (Figure 9). Its inclination towards the front gives the structure a sense a dynamism, somehow leaning to the front only retained by the opened plans of counter stays. The steel mast has and arrow shape, whose lateral opened plans are the same plans of the counter stays, simplifying force transference. These side steel sheets have a curved profile to receive the counter stays giving rise to the mast pointed head. Front stays pierce the central body of the masts to anchor inside of it. The mast shaft is formed by two elements, central body increasing to the back, and lateral decreasing wings to the front. Different colors in the steel structure underline the slender profile of the mast fixed in the center of the deck.

For 30 m length in the concrete counter weight abutment over 6 m concrete side cantilevers continue the composite side cantilevers of the main span. Concrete ribs reproduce the rhythm and forms of the steel ribs with curved profile supporting the 22 cm slab.

Counter weight abutment has a wedge plan. This triangular shape is curved and inclined in the front as continuation of the mast shaft. This curved element conforms the bow of head of the abutment and contributes to the optimistic zest that the design configuration tries to reflect, as part of the spirit of the Science and Technology Park as an innovation and progress thrust.

Figure 10. Erection of the steel inclined mast, and night view of the finished structure.

3.3 *Method of erection*

The method of erection for the main span consists in positioning of the steel portions of the deck on temporary supports, and welding of these portions together. The 22 cm concrete slab is poured on the deck while resting on these supports using prefabricated 7 cm elements, with no need for any scaffolding or disturbance to the highway traffic.

The mast is erected after completing the deck and counter weight abutment. The mast is lifted in two pieces separately, shaft and head (Figure 10), and welded in place. It is also helped by a small temporary support until the counter stays are installed and tensioned to initial force. All stays are positioned and tensioned to initial force in successive order. Tensioning sequence is studied to avoid extra charging of temporary supports. Stays and counter stays are tensioned alternatively in only one sequence in order to reach final state of forces and geometry. After tensioning the deck is raised 9 cm in mid span in order to face permanent loads of pavement, sidewalks and balustrades. During analysis it is found how counter stays influence strongly each other and front stays as they are tensioned, therefore high accuracy is necessary in all the process to avoid deviations which would implicate necessarily many later adjustments. Tensioning is finished successfully following the procedure developed from stage analysis model.

Lighting again has a strong importance because of the representing function of the structure as image of the PCTCAN and gate to the city of Santander (Fig. 10). Placing of light points is studied carefully to have uniform light in mast and stays with no disturbance to the traffic, or light pollution because of excessive lighting.

4 LA CAVA FOOTBRIDGE IN LOGROÑO

4.1 *Location and design*

Located in the North part of Spain, Logroño, with about 160,000 inhabitants, is the small capital of the La Rioja region, well known because of its high quality and important production of red wine. The city lies along the south bank of the Ebro River, flowing from the West to the Mediterranean, giving rise to an urban model made up of straight streets arranged as an orthogonal net.

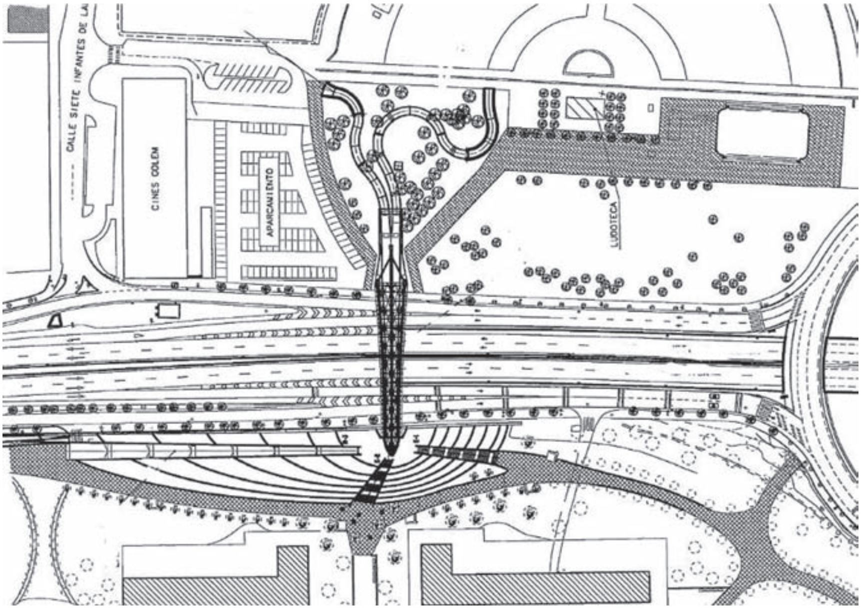


Figure 11. General plan of the footbridge and access ramps.

The South border of the city has been marked for many years by a peripheral highway that has limited its urban development, until the City Council decided to jump over the highway and develop the South areas, with the unavoidable requirement of erecting several pedestrian bridges. Logroño being now a red spot for symbolic, modern architecture, the pedestrian bridge of la Cava is the result of a contest launched by the Logroño City Council. The span to be covered by the structure was about 60 meter and the minimum vertical clearance over the highway level being 530 cm. Both ends of the pedestrian bridge were green park areas with enough space for developing the access ramps (Figure 11).

The winner proposal of the contest was the engineering firm Arenas & Associates. The structure was conceived like a steel three dimensional truss, spanning 61 meter over the highway, with a cross section of curved members and triangular shape, with variable height and width. The overall impression transmitted by the truss is that of a one span flexure resisting girder. Materialized with steel profiles and tubes, the new structure recalls us the gothic art shapes. Since the pedestrians walk inside the three dimensional truss, their relation with the structure allows them to enjoy the internal architectural space. The fact of covering the lateral curved façades of the structure with glass creates a pedestrian space fully protected from the rain, the wind and, very important detail, from the noise and disturbances of traffic.

The pedestrian bridge spans over the urban highway. The right zone is a green park area where two structure ramps of different slopes are located. At the left one where new residential quarters are under construction a new artificial hill allows several ramps to develop.

4.2 *Geometry of the structure*

4.2.1 *Main span of the footbridge*

The steel three-dimensional truss that solves the 61 m main span shows clearly the variable size of its different cross sections (Figure 12). Simply supported at the left abutment, it is fixed in the right (North) support, giving rise here to a negative bending moment materialized by a couple of forces, where straight, external members are ready to receive them and to translate them to the foundations (Arenas & Capellán 2006, 2007, Velando 2008).

Because of the ground organization, we conceived the truss girder as a beam fixed in the North abutment and simply supported at the South one with the width and height of the truss decreasing gently from the North to the South abutments. It is interesting to note that varying the depth of a beam is a usual decision in bridge design with clear structural and aesthetic consequences. But in the case of the Logroño pedestrian bridge, such height variation is accompanied and underlined by the decreasing width of the deck when one walks from the city to the new quarters (Figure 13). The result is that crossing the bridge constitutes a true dramatic experience, because the cross section of minimum size is coincident with the opening of the South abutment, this last frame acting as a threshold of the crossing, when the walker clearly feels that, throughout the bridge, he has been translated to a different and more open urban ambience.

As bridge designers, we are usually constrained to constant width designs. The width must be maintained in traffic bridges but in pedestrian bridges a constant width is not at all compulsory. On the contrary, one of the cultural values of an urban footbridge is the contribution to a particular ambience where the pedestrian is invited to go through the bridge. We think that a strong variation

Figure 12. Elevation view of the finished footbridge.

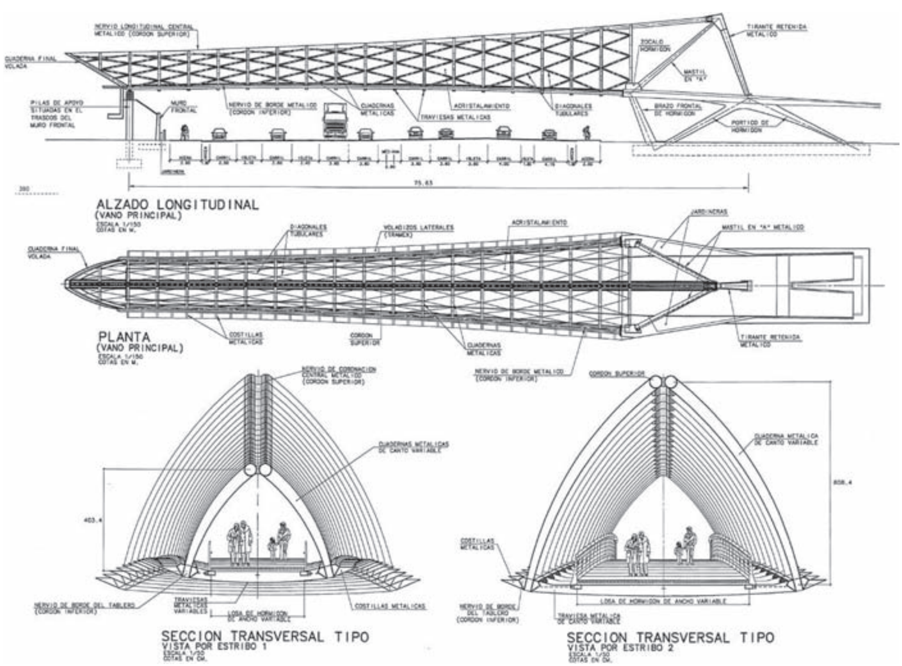


Figure 13. Drawing of the main span elevation, plan and sections.

of the width and the height of the truss, like the one offered in the Logroño design, creates the wish and appeal people to walk along it (Figure 14).

Glazing of the lateral façades of the steel spatial truss is important to protect the users and to create the internal space. The shapes of the structure become subtle under the glass. The transverse gothic frames and the longitudinal members at the crown and bottom define a clear construction. The reticulated thin tubes in the lateral façades transform the basic truss into a complete structure.

4.2.2 *Access ramps and artificial hill*

In the North abutment there are two curved ramps that allow people to go up and down for accessing the footbridge, offering slopes of 6 percent and 9 percent. They are placed in a green park and we have taken care of their insertion among the existing trees. Because the footbridge is intended for the use of a population located along a sizable park, the two ramps allow us to a better distribution of the citizens walking to the bridge (Figure 15).

Figure 14. Glazed lateral façades of the main span.

Figure 15. Access ramps and north abutment.

Figure 16. Night view of the access artificial hill in the south abutment.

The design of the South abutment is central to the urban quality of this project. The proposal included a new artificial earth hill of elliptical shape on this end allowing to develop several ramps and stairs with greater or lesser slope. The hill composes by itself a green area and a noise protector for the inhabitants of the South quarters (Figure 16).

4.3 *Method of erection*

The construction techniques used during the erection of the new footbridge are mainly the following:

- Construction of the concrete access ramps using temporary falsework.
- Construction of the steel structure of the main span on the ground.
- Erection of the main span using cranes.
- Pouring of the concrete slab, using steel sheets composite deck system.

The light weight steel structure of the main span is chosen from the very start to be erected in one piece using cranes. In this way the construction of its skeleton is carried out without any disturbance to the traffic. Other than the glazing and the final touches of protective paint, only the pouring of the concrete slab is carried out over the existing road, but even this is carried out without any falsework, with the composite deck steel sheet system.

5 DELICIAS FOOTBRIDGE IN ZARAGOZA

5.1 *Location and design*

Located in the North Eastern part of Spain, Zaragoza, with about 650,000 inhabitants, is the fifth city in Spain and capital of the Aragón region. The city is to house the 2008 International exhibition during the summer. It lies along the south bank of the Ebro River, flowing from the West to the Mediterranean. Zaragoza needed for 2008 a new footbridge over an important traffic joint next to the high speed train station. The footbridge will serve to cross from Delicias station to Almozara district towards the Expo, and continue by foot or using the cableway installed for the occasion.

This delicate location due to the proximity to the station and its architectural value, and not far from the Third Millennium Bridge (Arenas 2002, Arenas et al. 2008a, b, c, Aguiló 2008) and the International EXPO 2008, asked for a footbridge up to this design standards but not competing with the train station and the rest of the elements (Casas 2008, Aguiló 2008).

The access ramp is over 100 m long to reach an elevation of 6 m over the road, with a limited slope of the 6%. A curved ramp gives access both to the main span over the traffic joint and a viewpoint towards the river. For the main structure a self-anchored curved hanging footbridge is designed with a main span of 90 m. The steel mast is inclined towards the footbridge and anchored to the back to a buried counterweight. The lightness of this structure makes it completely transparent, at the same time the curved deck and cables create interesting surfaces and volumes due to the three-dimensional character of the cable net. Its transparency and limited height, smaller than the station arches, and the dialogue of both the station and footbridge in its color and shapes, ensures a good integration in its urban environment (Figure 17).

5.2 *Geometry of the structure*

5.2.1 *Main span of the footbridge*

The main span of 90 m has a 6 meters wide deck curved in plan and suspended trough hangers from four main cables. The structural scheme is that of a self anchored hanging bridge, with the deck taking in compression the horizontal component of the cable tension force (Figure 18).

The curved plan of the deck makes the form finding of the main cables an iterative work in order to consider the transversal component of the hangers force, and the second order analysis of cable stiffness under tension.

It is the main structure which because of its span and central position allows and demands for a more visible structure. In any case, we prioritize to create and element integrated as far as in scale and height in relation to its surroundings from its structural design, with high transparency and lightness to diminish its visual impact, but of a high technological and design content so the

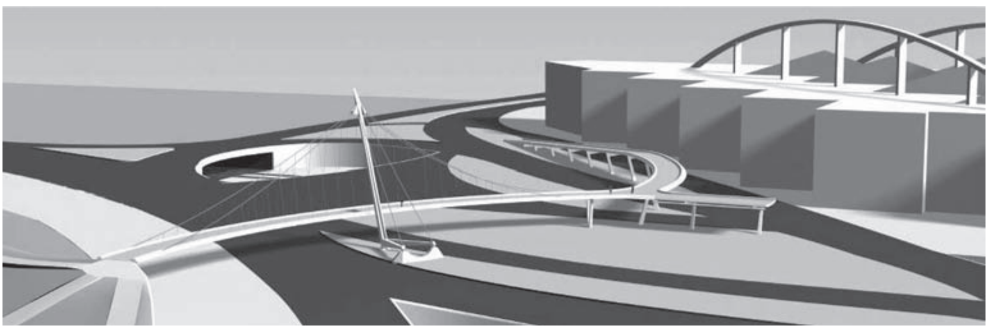


Figure 17. Conceptual image of the footbridge in relation to the train station of Delicias, Zaragoza.

Figure 18. Perspective of the main span and inclined mast.

crossing will become the desired attractive frame within the panorama of Zaragoza city at the door of the international Expo 2008.

As a consequence, the inclined mast adopts simple shapes at the same time it tries to maximize its expressiveness. Its inclination frees the deck of direct contact to the mast, making the 90 m long curved deck a slender line with no visual obstacles. Because of this inclination the mast needs counter stays to a buried counterweight for balance. Instead of transmitting inclined forces of tension and compression to the foundation, we decided to have the balanced horizontal component taken by struts elements, making them visual to everyone instead of burying this element, and allowing us to transmit only vertical forces to the foundation on new earth fills.

All the elements of the mast are duplicated: shafts, horizontal struts and counter stays, to collaborate to transversal stability of the structure under unbalanced loads on the deck (Figure 19). The result increases the interest of this element, making its configuration much more three dimensional, and allowing the mast shaft to adopt an "A" shape with a desired pointed vertex (Figure 20).

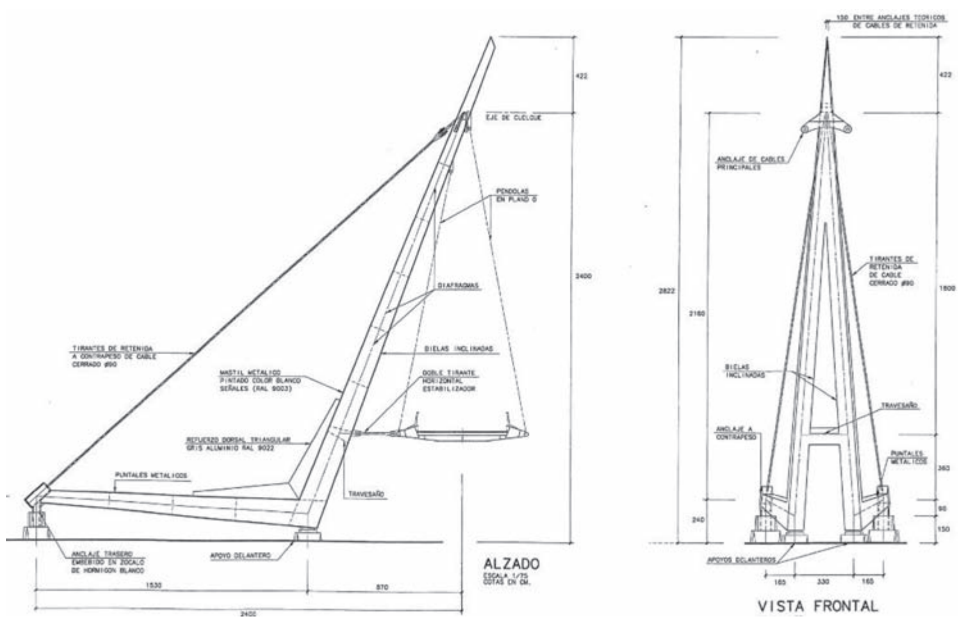


Figure 19. Drawings of the inclined mast configuration.

Figure 20. Image of the footbridge during nightfall.

Main cables are full locked cables of 90 mm diameter, and hangers are 30 mm spiral strand cables. The hangers are anchored to the main cables using bolted elements whose design was included in the project. Tensioning of the structure takes place using jacks on the main cables, with no need to tensioning in the hangers except for final force verification.

Previous experience of Arenas & Associates with this typology of footbridge can be found in the access footbridge to a Commercial Area in Sanchinarro (Madrid), where the 8 m wide deck hanged from two main cables anchored to the building structure, with 45 m span (Figure 21) (Arenas et al. 2005a, b).

Special studies have been carried out to ensure the vibrations on the deck will be below acceptable range of acceleration. The lowest natural frequency for vertical vibrations is 0.8 Hz, below the range of 1.5 to 4 Hz to avoid because of human-induced vibrations. All analysis results guarantee that the structure stiffness and mass (with a composite deck with 15 cm concrete slab) is enough to limit deformations and accelerations efficiently.

5.2.2 *Access ramp*

The access ramps solve the technical problems related to be supported on the Station concrete slab, and they constitute urban design elements, at the same time their presence and dimensions do not compete with the architecture of the station.

Ramps design search for the maximum lightness, to reduce visual impact, and to limit the load of its supports lying on the pre-existing slab of the station over the railway lines.

Figure 21. Image of the footbridge in Sanchinarro (Madrid).

Figure 22. Image of the access ramp with the main span of the footbridge and the mast at the background.

Figure 23. Access ramp (left) and main span during construction on temporary supports (right).

Inclined steel piers with "7" shape reflect the asymmetry of the approach, in which the piers flee from the station building, in the same time they establish architectural dialogue with the building. The increasing height of these elements and the final curve of the ramp give a very dynamic character to this access structure (Figure 22).

The ramp structure is made out of steel except for a compression slab of less than 10 cm.

5.3 *Method of erection*

The method of erection for the main span consists in positioning of steel portions of the deck on temporary supports, and welding of these portions together. The 15 cm concrete slab is poured while resting on these supports. The mast is the first element to be erected after completing the foundations. It is also helped by a small temporary support until the counter stays are installed and tensioned to initial force (Figure 23). Once the deck structure is completed on temporary supports, main cables and hangers are installed and tensioned using jacks at both ends of the deck. Tensioning of the main cables sets the hangers in tension and allows removing the temporary supports. Construction of the footbridge and ramps is completed in seven months. On the opposite end of the footbridge of the station, a small modular building is built including stairs and elevators to communicate with ground level and the park below. The footbridge surface is made out of wooden Tali deck.

6 CONCLUSIONS

Designing and constructing the two similar cable stayed bridge of Las Rozas and PCTCAN in a very short period of time has allowed us to compare different solutions and procedures in cable stayed bridges. The erection method followed in both cases for the deck using temporary supports has worked well, and it has allowed fast construction times for the complete bridge of less than a year in both cases. Asymmetric cable stayed design has proved to suit very well the site conditions, as for deck depth, as structural behavior, and aesthetic integration. As much as for differences, rigid counter stays helped to stiffen the structure and simplified tensioning procedure, but they implied the use of more steel. Deck hinged in the abutment with elastomeric bearing simplified erection procedure but it will require long term maintenance, while the deck fixed to the concrete abutment ensures a very good durability. Stays technology and materials in both cases guarantee long term durability and resistance, and the use of the proper tensioning procedure and instrumentation during the whole process have given high accuracy levels in the values of forces and deformations, which is also a guarantee for future structural behavior and durability. As a result of all the design and erection work carried out by Arenas & Associates, both bridges have become the iconic images wanted with a unique and original character of their own.

Referring to La Cava and Delicias footbridges, we can see that this kind of structures allow very different design solutions, with less restrictive geometric inputs than road or railway bridges. As a consequence a very wide range of solutions is possible with high potential of expressiveness. Nevertheless it is wrong to think that no matter which design will suit a location, nor that a specific impressive design is right no matter where, and then give up to empty spectacular designs. Urban integration, or to set a proper relationship within the parts of the structure and between the structure and its surrounding, is vital to achieve a good design. Doing less or being less visible is a relevant choice to make in a delicate environment. Another idea which comes out of these works is that it is very relevant to take highly in account the users and its comfort when using our structures, avoiding too steep slopes, long paths or unsafe or weather or traffic exposed areas. Following these basic lines, we find that the footbridges built in Logroño and Zaragoza are a good example of urban integrated footbridge designs in signature structures being built in Spain designed and directed by our firm, Arenas & Associates.

REFERENCES

Aguiló, M. 2008. *Puentes para una exposición. Zaragoza 2008.* Madrid: Abada Editores.

Arenas, J.J. 2002. *Caminos en el aire: Los Puentes.* Madrid: Colegio de Ingenieros de Caminos, Canales y Puertos.

Arenas, J.J. & Capellán, G. 2006. Pasarela peatonal sobre la circunvalación: Parque de San Adrián. La Cava (Logroño). *Proyectar Rioja* 36: 115–118.

Arenas, J.J. & Capellán, G. 2007. Nueva pasarela de La Cava en Logroño. *Cauce 2000: Revista de la Ingeniería Civil* 136: 49–55.

Arenas, J.J., Beade, H. & Ortega, A. 2008a. El Puente del Tercer Milenio: un hito simbólico de la ciudad. *Cauce 2000: Revista de la Ingeniería Civil* 141: 18–25.

Arenas, J.J., Capellán, G., Beade, H., Martínez, J. & Ortega, A. 2008b. El Puente del Tercer Milenio sobre el Río Ebro en Zaragoza. *Revista de Obras Públicas* 3490: 75–98.

Arenas, J.J., Capellán, G., Beade, H., Martínez, J. & Ortega, A. 2008c. El Puente del Tercer Milenio sobre el Río Ebro en Zaragoza. *Rutas: Revista de la Asociación Técnica de Carreteras* 127: 61–64.

Arenas, J.J., Capellán, G., García, P., Sacristán, M. & Merino, E. 2008. Puente atirantado "Puerta de Las Rozas" sobre la A-6, Madrid. *Revista de Obras Públicas* 3488: 11–30.

Arenas, J.J., Martínez, J., Merino, E., Sacristán, M. & Guerra, S. 2005a. Proyecto de tres puentes en el centro comercial Sanchinarro. Madrid. *III Congreso de ACHE. Zaragoza,* 14–17 *November 2005.* Madrid: ACHE. Asociación Científico-Técnica de Hormigón Estructural.

Arenas, J.J., Ortega, A., Hue, P. & Arias, J. 2005b. Puentes de acceso al Hipercor de Sanchinarro. *III Congreso de ACHE. Zaragoza,* 14–17 *November 2005.* Madrid: ACHE. Asociación Científico-Técnica de Hormigón Estructural.

Casas, F.J. 2008. Deshaciendo el nudo. *Europ'A. Acero Arquitectura* Especial Julio 2008: 18–19.

Velando, C. 2008. Pasar, pasear, estar. *Europ'A. Acero Arquitectura* 8: 14–15.

Author index

Recent surveys of the U.S. infrastructure's condition have rated a staggering number of bridges structurally deficient or functionally obsolete. While not necessarily unsafe, a structurally deficient bridge must be posted for weight and have limits for speed, due to its deteriorated structural components. Bridges with old design features that cannot safely accommodate current traffic volumes, and vehicle sizes and weights are classified as functionally obsolete. Such deficiencies may adversely affect the performance of transportation systems in emergency situations or for disaster response. This narrative has become part of the public debate sparked by the collapse of the I-35W Bridge over the Mississippi River in Minneapolis, Minnesota, USA, on August 1, 2007. Ever since, numerous technical and news articles have been written to answer the persistent question, *why did the bridge collapse?*

Exhaustive examination of the details of a specific bridge failure, typically, reveals the reasons for the collapse and lessons are drawn from the experience. Each bridge failure, since the Tacoma Narrows Bridge disaster in 1940, has served as a wakeup call for the bridge engineering community, initiating radical changes in the design and construction standards. However, a paradigm shift is necessary in the inspection and monitoring practices of the bridge engineering community to provide preventive maintenance and restore the public's confidence in the safety of bridges.

Concerns about bridge safety and reliability go beyond geographical boundaries and are shared by bridge engineers from different countries. This book contains a number of selected papers that were presented at the Fifth New York City Bridge Conference, held on August 17–18, 2009. These papers cover a wide range of topics in the design, construction, maintenance, monitoring and rehabilitation of bridge structures.

CRC Press
Taylor & Francis Group
an **informa** business
www.crcpress.com

6000 Broken Sound Parkway, NW
Suite 300, Boca Raton, FL 33487
Schipholweg 107C
2316 XC Leiden, NL
2 Park Square, Milton Park
Abingdon, Oxon OX14 4RN, UK

ISBN-13: 978-0-415-56484-7

9 780415 564847

an **informa** business