Modern Techniques in Bridge Engineering

Edited by Khaled M. Mahmoud

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Editor

Khaled M. Mahmoud

Bridge Technology Consulting (BTC), New York City, USA

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Preface

Due to significant economic growth in the last few decades, increasing traffic loads impose tremendous demand on bridge structures. This coupled with ongoing deterioration of bridges; introduce a unique challenge to bridge engineers in maintaining service of these infrastructure assets without disruption to vital economic and social activities. This requires innovative solutions and optimized methodologies to achieve safe and efficient operation of bridge structures.

Bridge engineering practitioners, researchers, owners, and contractors from all over the world presented on modern techniques in design, inspection, monitoring and rehabilitation of bridge structures, at the Sixth New York City Bridge Conference; held in New York City on July 25–26, 2011. This book contains select number of papers presented at the conference. This group of papers is state-of-the-art in bridge engineering and is of interest to any reader in the field.

The Firth of Forth Estuary separates the Scottish capital of Edinburgh from the Kingdom of Fife to the north. The downstream crossings of the Forth at Queens ferry are two historic bridges, the iconic cantilever rail bridge constructed in the 1880's, and the Forth Road Bridge, Britain's first long span suspension bridge, which was opened in 1964. A recent comprehensive study into the future transport needs concluded that a third crossing is required to maintain the crucial link for the economies of Fife, Edinburgh and the East coast of Scotland. The Proceedings lead off with a paper by Kite et al., on the "Design of the Forth Replacement Crossing, Scotland, UK". The authors discuss design of the new transportation link, a cable stayed bridge with 3 towers and a pair of 650 meter main spans. In the centre of each main span the stay cables will overlap to stabilise the central tower, a unique design feature for a bridge of this scale. The design of the crossing has been carried out in accordance with the Eurocodes and project specific design criteria. The freeway E75 runs through Belgrade connecting central and southeast Europe. Known as "Autoput", the freeway is notorious for traffic congestions, which present a major obstacle for further economic growth in the region. Traffic problems extend to all roadway bridges across the Sava River. In order to solve these traffic problems and in order to improve the infrastructure of the surrounding area a ring road was planned. In "The Cable-stayed bridge across the River Sava in Belgrade", Walser et al. provide details of the design of the new cable-stayed bridge which is the key structure of the infrastructure improvement project. The bridge connects New Belgrade in the North with the Radnicka Street in the south and is located four kilometers upstream from the confluence of the Sava and the Danube Rivers. The new landmark of the city of Belgrade will carry a needle type tower with a total height of 200 m. With 40 stays on each side it holds up the main span of 376 meter length and backspan of 200 meter length. The total length of the structure including the approach spans will be 967 meter with a 45 meter width. The authors present details of design and construction of the structure. The existing bridge across the Indian River Inlet in Delaware has long been plagued by extreme scour conditions within the channel. To improve the safety of the crossing, construction of a new bridge is ongoing under a design-build contract. The new design addresses two key challenges of the site, presented by Nelson in "Indian River Inlet Bridge—A design-build in progress". First, the main span was set to keep the substructure for the new bridge out of the channel in order to mitigate the threat posed by scour. Second, the bridge must be highly resistant to corrosion due to its proximity to the Atlantic Ocean. These requirements dictated the design of a concrete cable-stayed bridge, which has a 290 meter long main span across the inlet. A significant portion of the bridge is constructed on false work using precast floor beams. Only the central two-thirds of the main span is erected with a form traveler, thus simplifying and accelerating the construction. Moisture-imposed deterioration of suspension bridges main cables has been recognized as a very serious problem. The safety of a great number of suspension bridges is compromised due to moisture-imposed deterioration that reduces the load carrying capacity of the main cables. In "Main cable corrosion protection by dehumidification: Experience, optimization and new developments", Bloomstine describes the development of corrosion protection of steel bridge components by dehumidification over the last 40 years. The paper focuses on the application of dehumidification technique to the main cables of suspension bridges.

For a number of years, bridge designers have been made aware of the need to design for future maintenance. In the UK and other European countries there are now statutory obligations in force that require engineers to ensure that safe access and methods of working to carry out maintenance are determined for all projects from the design stage. A gap exists in knowledge between bridge design engineers and bridge maintenance engineers. In the current financial climate, there is stronger pressure on design engineers to minimize construction costs, potentially at the expense of future maintenance. For maintenance to be effectively considered in bridge design, the knowledge and experience of maintenance engineers should be harnessed at the design stage to improve the future maintainability of bridges. In his paper "Bridge design for maintenance", Colford examines this issue with illustrations from two case studies. The Lake Champlain Bridge, also known as the Crown Point Bridge, was an historic steel truss bridge stretching 667 meter (2,187 feet) across Lake Champlain between Crown Point, New York, and Chimney Point, Vermont. After significant deterioration was discovered in the unreinforced concrete bridge piers in the fall of 2009, the bridge was abruptly closed to all traffic. An emergency declaration was issued by the Vermont Agency of Transportation and by New York State. A Safety Assessment Report was prepared, and it was concluded that the existing truss bridge should be demolished. The two regions connected by the bridge are economically linked and share services, including hospitals and fire departments. Given the criticality of the bridge for the regional economies, plans for the demolition of the original truss bridge and the design and construction of a new network tied arch bridge were developed. Utilizing design-bid-build contracting methods, the demolition of the existing deteriorated bridge, and the design and construction of a new signature crossing was to be completed within twenty months. In "The New Lake Champlain Bridge design process", Potts et al. discuss reasons for the bridge closure and the design-bid-build approach that enabled an expedited replacement of the Lake Champlain Bridge. Xinjin is a historical town located 12 km south of Chengdu, China. It has attracted many residents and visitors over the years, and is geographically centered on the convergence of five rivers. Over the years poets have exalted its natural beauty. To elevate the image of Xinjin and attract business and investments, the City of Xinjin decided to construct a signature pedestrian bridge, the Nanhe Landscape Bridge, in the center of the city. The pedestrian bridge has 5 spans with a total length of 229 meters, and features two intertwining steel-box girders in a 3-D space, creating a symbol of the five rivers that converge at the city. In "A signature footbridge for Xinjin, China", Ye et al. present the design details of this signature bridge. The St Laurent Bridge, part of the new A30 highway in Quebec, Canada, is a low-level crossing over the St .Laurent River. This concrete structure supported on short stocky piers has been designed with seismic isolation to reduce the otherwise high seismic demand due to its support configuration. In "Comparative of single pendulum and triple pendulum seismic isolation bearings on the St. Laurent Bridge, Quebec, Canada", Barbas et al compare the use of friction pendulum bearings in two of their modalities, single and triple pendulum, to determine which was most effective. In a study of bridge failure causes in the United States, over 60% were due to hydraulic factors. Changes in sea-level rise, precipitation, and storm frequency may have important implications in the hydraulic design, analysis, inspection, operation and maintenance of bridge structures. Climate change predictions for the northeastern United States are particularly alarming as many coastal cities would be affected. Shields discusses "Incorporating climate change predictions in the hydraulic analysis of bridges", using the New York State coastal region as a case study.

Innovations in design, fabrication, construction, inspection methodologies, and material technology provide more efficient bridge structures. A Concrete Filled Tubular Flange Girder (CFTFG) is an I-shaped girder that uses a concrete filled hollow structural section (HSS) as the top flange. A key advantage of the CFTFG is the increased torsional stability that reduces the need to brace the girders under construction loading conditions. Time and cost savings in fabricating and erecting the bridge girder system can be realized by a reduced number of diaphragms along with the locating the field splices over the piers. Combined with span-by-span construction, CFTFGs provide a robust steel structure that can be erected much more efficiently than conventional I-girder bridges. In "Concrete filled tubular flange girder bridge", Bondi presents a, first of its kind bridge in the world, two span structure over Tionesta Creek, located in western Pennsylvania, USA, using CFTFGs system. The Northeast Extreme Tee (NEXT) Beam is a new beam standard developed by the Precast/Prestressed Concrete Institute Northeast (PCINE) Bridge Technical Committee. Over the past five years the technical committee, which is comprised of the New England States and New York State Departments of Transportation, Precast Manufacturers and Consultants, has focused their activity on developing regional standards for use on accelerated bridge construction. In "The Queens Boulevard Bridge—the NEXT generation", Tuckman and Deitch provide details of the benefits the NEXT Beam delivers in the design, fabrication and construction process, utilizing the case study of Queens Boulevard Bridge over the Van Wyck Expressway in New York City. The authors briefly discuss the LRFD Design requirements for the NEXT beam. After the collapse of the I-35W Bridge in Minneapolis, Minnesota on August 1, 2007, gusset plate connections now must be evaluated by transportation agencies. Connection evaluations require accurate as-built drawings. In "New tools for inspection of gusset plates", Higgins and Turan present a new methodology that permits rapid collection of accurate field measurements of connection plate geometry. The method uses close-range photogrammetry techniques to rectify field-collected digital images. Consumer-grade cameras are used to produce scaled orthographic photographs (orthophotos) of the bridge connections.

The Delaware River Turnpike Bridge is a 2003 meter (6,571 foot) long viaduct over the Delaware River with 14 open deck truss spans of 75 meter (245 feet) in average length, flanking a central three span arch truss arrangement with a center suspended span. The bridge was constructed in 1954 and is owned by the New Jersey Turnpike Authority and the Pennsylvania Turnpike Commission. Following the collapse of the Minnesota I-35 Bridge, truss bridges and their gusset plate connections around the United States were subject to intense scrutiny. A detailed evaluation of this structure was promptly directed by the NJTA. Subsequent analysis indicated inadequate connector (rivet) strength as a structure wide concern. A retrofit contract was immediately commissioned by the bridge owners to correct this condition. In "Delaware River Turnpike Bridge—Riveted connection analysis and retrofit", Schaefer et al. discuss the approach to the project and provide details on the large scale rivet replacement, which became a primary focus of the project. The 31st Street Arch is a reinforced concrete arch bridge that was constructed around 1914. It is part of the Hell Gate Viaduct, which is comprised of a series of concrete arch bridges, steel truss bridges and retained earth structures. The bridge is approximately 21 meter (70 feet) wide, 18 meter (60 feet) high and spans approximately 30 meter (100 feet) over 31st Street and an elevated New York City Transit track structure. It carries four tracks, three of which are active with Amtrak and CSX trains. A series of investigations, including visual inspection, material testing and structural analysis was conducted. The visual inspection revealed that the extent of deterioration was more extensive than anticipated. In his paper "Rehabilitation of the 31st Street Arch of Hell Gate Viaduct, New York City", Abrahams provides details of the repair work on this old structure. The Sandesund bridge, located about 100 km south of Oslo, Norway, was built around 1972–77. The total bridge length is 1521 meter and is divided in four sections. The section of the structure crossing over the Torsbekkdalen Valley contains the largest 97.7 meter span and is where a change in the static system was considered. The main spans were built by the cantilever construction method and the Torsbekkdalen Valley span was designed and constructed with a hinge in the middle of the span. Over the years, the main span has deflected about 200 mm in the middle. Due to increasing heavy traffic, the hinge was increasingly subjected to live load impact. It was therefore desirable to investigate whether it was possible to introduce continuity in the main span. In their paper "Main span hinge elimination and continuity retrofit of the Sandesund Prestressed Box Girder Bridge in Norway", Fjeldheim and S. Kr. Heggem provide details of the retrofit design. The Benjamin Franklin Bridge, owned by Delaware River Port Authority, is a suspension bridge that spans the Delaware River between the states of New Jersey and Pennsylvania, USA. It was opened to traffic on July 1, 1926. The bridge carries vehicular traffic, pedestrian traffic and trains. The Camden approach, on New Jersey side, consists of seven truss spans, ten girder spans, twelve stringer spans and an abutment. The Philadelphia approach, on Pennsylvania side, is comprised of five truss spans, nine girder spans and an abutment. The existing steel truss bearings have severe corrosion problems and need retrofit. In "Deck truss bearing rehabilitation for the Benjamin Franklin Bridge", Ye et al. discuss different retrofit options, such as replacing steel rocker nests only or installing seismic isolation bearings, which were evaluated based on costs, as well as seismic performances. Built in 1901, the Atlantic Avenue Viaduct carries Long Island Railroad's (LIRR) Atlantic Tracks 1 and 2 connecting Jamaica to Downtown Brooklyn in New York City. In March 2008, a $69 million design/build contract was awarded to demolish and replace 176 spans and rehabilitate 86 spans of the elevated viaduct. One of the requirements for the rehabilitated and replaced spans is to provide uplift restraint at points of bearing. The design called for low profile uplift restraint disk type bearings. The vertical load requirements for these bearings range from 712 to 836 kn (160 to 188 kips) with horizontal load capacities of 30%. In addition an uplift restraint capacity of 62 to 124 kn (14 to 28 kips) was required and due to the low profile limitations a dovetail uplift restrainer was utilized. In "Low profile uplift bearings for the Atlantic Avenue viaduct, New York City", Watson and Conklin provide details of the selection, design, manufacture and testing of these specialty devices.

Accelerated Bridge Construction (ABC) applications in the U.S. have developed two different approaches; accelerated construction of bridges in place using prefabricated systems and the use of bridge movement technology and equipment to move completed bridges from an off-alignment location into the final position. Pre-engineered modular systems configured for traditional construction equipment could promote more widespread use of ABC through reduced costs and increased familiarity of these systems among owners, contractors and designers. In "Rapid replacement of bridges using Modular Systems", Sivakumar discusses the development of modular ABC systems, design of the modular replacement bridge and chronicle the ABC technologies and design innovations. On the theme of ABC, Kaleghi et al. present an overview of the latest ABC connection details being considered for use in moderate to high seismic regions in "Accelerating bridge construction with prefabricated bridge elements—The latest in new technology for moderate to high seismic regions in the United States". The authors focus on implementation and constructability of the "Next Generation" of seismic details. The Hog Island Channel and Powell Creek Bridges, located along the Long Beach Branch of the Metropolitan Transportation Authority (MTA) Long Island Rail Road, served the Railroad for almost 90 years. The existing Hog Island Channel Bridge carried two electrified train tracks and consisted of timber pile bents and wide flange steel stringers. The existing Powell Creek Bridge also carried two electrified train tracks and consisted of timber piles and beams. In "Replacement of the Hog Island Channel and Powell Creek Bridges", Fernandez provides details of replacing the bridges while maintaining railroad service on the existing bridges. Construction started in September 2009 and completed by the end of May 2010.

Monitoring of bridges can serve wide range of purposes, providing continuous records of variation in a structure's condition. This may be of special interest to owners of older bridges, who must ensure the safety of their structures. Safety could be compromised because of the ageing process, and due to the increased traffic loading that exceed the bridge design live loads. An automated monitoring system can be utilized to provide real-time information on the structure's condition, allowing gradual changes over a period of time to be identified and offering notification over the internet of any sudden changes. Having such a system in place ensures that any changes in the structure's condition will be recognized immediately, allowing appropriate strengthening work to be planned. In "Remote structural monitoring systems for long-term confidence in a structure's condition", Spuler et al. provide details of automated structural health monitoring systems and the benefits in keeping track of structure's condition. Most highway facilities in the United States are governed by design, construction, maintenance, inspection, and operations codes and regulations of the American Association of State Highway and Transportation Officials (AASHTO) and the U.S. Federal Highway Administration (FHWA). However, to date highway tunnels in the U.S. do not have comparable national codes and regulations. Recent events such as the July 2006 ceiling collapse of the I-90 Central Artery Tunnel in Boston, Massachusetts, and the 2005 International Scan on tunnels, have called attention to the need for such national standards. The "Best Practices for Roadway Tunnel Design, Construction, Maintenance, Inspection, and Operations" domestic scan was a key activity initiated to assist in addressing this need. In "Strategies in tunnel design, construction, maintenance, inspection and operations", Capers et al. provide a summary of the domestic scan 2009 findings and recommendations, as well as an up to date overview of efforts underway by the AASHTO Subcommittee on Bridges and Structures in advancing national standards and guidance for designing and constructing roadway tunnels. Over the past two years, multiple studies have been conducted through the National Academies Cooperative Research Programs on the topic of determining best bridge management practices by various state departments of transportation. In their paper "Identifying and sharing best practices in bridge maintenance and management", Myers and Capers discuss NCHRP 20-24(37)E and Domestic Scan 07-05. Both of these studies focused on states' approaches to continuous bridge maintenance and concluded in recommendations for ways to incorporate the most effective bridge maintenance programs in practice around the United States. Recommendations focused on proper use and allocation of resources and funding, preservation and preventative maintenance, advancing and improving effectiveness of inspection, and developing closer tracking of bridge condition and managing information.

The Alfred Cunningham Bridge, in Craven County, North Carloina, U.S., was designed in approximately 1953 and built in 1955. It was 537 meter (1763 feet) long and consisted of a 107 meter (350 foot) long New Bern approach, a 67 meter (220 foot) swing span, and a 363 meter (1190 foot) long James City approach. The approaches consisted of 11 meter (35 foot) long spans comprised of a reinforced concrete deck supported on rolled steel beams with a substructure composed of reinforced concrete pier cap bents on octagonal precast piles. The swing span was a center bearing through truss span with a concrete filled grating deck and reinforced concrete safety walks supported on a reinforced concrete circular pivot pier founded on octagonal precast piles. The swing span provided two 24 meter (78 foot) navigational channels, between the fenders with unlimited vertical clearance when the span was in the open position. An approximate clearance of 4.25 meter (14 feet) was provided with the swing span in the closed position. The bridge was in a deteriorated condition and had been classified by the North Carolina Department of Transportation as structurally deficient and functionally obsolete. In "New bascule bridge for historic New Bern, North Carolina", Kelly provides explanation of the details and materials used for the historic structure, various components, and more specifically the control house. The Hudson River is one of the most historically rich rivers in the United States and defines one of the most important regions of development in the U.S. history. It is also home to a variety of bridge types that reflect the development of structural engineering. In "Site and sight: Examining the forms of Hudson River Bridges", Draper et al. briefly discuss structural forms for bridges, and the relationships bridges have to their environments and to the other bridges nearby. The authors focus on the Tappan Zee Bridge and its planned replacement. The St. Louis Bridge, built by Captain James Eads in 1874, and the Brooklyn Bridge, completed by Colonel Washington Roebling in 1883, shared innovative techniques and established precedents for designing long-span bridges in the years ahead. Despite the St. Louis Bridge being a steel arch bridge which differed from the wire suspension construction of the Brooklyn Bridge, both of these structures were considered the largest bridge of the time, and continue to hold historic significance. In spite of the achievements of both Eads and Roebling, their individual successes were overshadowed by a dispute between the two men. The root of their disagreement was the design of caissons used for the foundation, as each engineer claimed to be the first to design the caisson for his bridge foundation. In "The St. Louis Bridge, the Brooklyn Bridge, and the feud between Eads and Roebling", Gandhi provides an overview of the dispute between these two legends of bridge engineering.

The editor would like to acknowledge with gratitude the contributions of authors and reviewers of the papers contained in these proceedings. The materials presented in this volume are of interest to any reader in the field of bridge engineering.

Khaled M. Mahmoud, PhD, PE
Chairman of Bridge Engineering Association
President of Bridge Technology Consulting (BTC)
New York City, USA

New York City, July 2011

1 Cable-supported bridges

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Chapter 1

Design of the Forth Replacement Crossing, Scotland, UK

S. Kite
Arup, London, UK

M. Carter
Arup, Hong Kong

B. Minto
Forth Replacement Crossing Project, Transport Scotland, Glasgow, UK

ABSTRACT: The Forth Replacement Crossing road bridge will be built across the Firth of Forth in Scotland to maintain and enhance a vital transport link. The wide estuary will be crossed by a cable stayed bridge with 3 towers and a pair of 650 m main spans. In the centre of each main span the stay cables will overlap to stabilise the central tower, a unique design feature for a bridge of this scale. The scheme design of the crossing has been carried out by the in accordance with the Eurocodes and project specific design criteria. The structure will provide a fitting 21st century icon, to stand alongside the existing Grade A listed bridges: the cantilever rail bridge from the 19th century and the road suspension bridge from the 20th century.

1 INTRODUCTION

The Firth of Forth is a dramatic estuary which separates the Scottish capital of Edinburgh from the Kingdom of Fife to the north. The downstream crossings of the Forth at Queensferry are two historic bridges, the iconic cantilever rail bridge constructed in the 1880's (Mackay, 1993) and the Forth Road Bridge (Anderson et al., 1965), Britain's first long span suspension bridge, which was opened in 1964.

Despite significant investment and maintenance over its lifetime, the Forth Road Bridge is showing signs of deterioration as a result of increased traffic and the influence of weather. In 2005, investigations of the main cables (Colford, 2008) revealed serious corrosion which, if left unchecked, could lead to the bridge being closed to heavy goods vehicles as early as 2014, and to all traffic by 2019. A comprehensive study into the future transport needs concluded that a replacement crossing is required to maintain this crucial link for the economies of Fife, Edinburgh and the East coast of Scotland.

The replacement bridge will be slightly to the west of the existing bridges (Figure 1), making use of a natural granite outcrop, Beamer Rock, in the middle of the Forth to

Figure 1. Visualization: Three centuries of bridge engineering in the Firth of Forth.

allow the wide estuary with two navigation channels to be crossed by a cable stayed bridge incorporating a pair of 650 m main spans, with an approach viaduct to the south. Selection of the scheme and the reasoning behind the overall structural layout are described in Carter et al., 2009.

2 PROCUREMENT

The client, Transport Scotland, is procuring the crossing through a Design and Build contract, with particular constraints and criteria set out which ensure the bridge will satisfy the project requirements. This will bring experience from contractors into the project at a suitable stage and allow the chosen contractor to tune the detailed design to suit his preferred methods of working and maximize efficiency. A period of dialogue with tenderers was run in parallel with the parliamentary process leading to an Act enabling powers for construction.

Tenders were submitted in January 2011, with contract award and start of construction in April 2011, aiming for completion of the bridge in 2016.

3 THE SPECIMEN DESIGN

The Specimen Design of the crossing is a scheme design incorporating a high level of detail which has been produced by the Jacobs Arup Joint Venture. Transport Scotland required this Specimen Design for several purposes:

– in order to verify the feasibility of the bridge arrangement,
– to define the overall form and geometry of the crossing,
– to inform the environmental assessment and the Bill of Parliament,
– to enable a detailed cost build up to be calculated, and
– to be an illustrative design as a starting point for the tendering contractors from which to prepare their design proposals.

The general arrangement is shown in Figure 2. The total length of the bridge is 2,638 m. Although the crossing is divided into a cable stayed bridge and a southern approach viaduct, the structure is continuous from abutment to abutment with no intermediate expansion joints. Longitudinal fixity is provided by a monolithic connection at the Central Tower located on Beamer Rock with transverse support provided at all towers and piers.

The towers are vertical reinforced concrete elements located on the centerline of the bridge. Two planes of stay cables have deck anchorages located centrally in the "shadow" of the towers between the carriageways. The stay cables overlap in the center of the main spans so that the longest cables pass beyond the mid-span locations. This arrangement provides stability to the central tower, avoiding alternative arrangements with heavy looking towers or more complicated stay cable arrangements. The deck itself is a streamlined box girder and stay cables are multi-strand type.

The key design requirements for the approach viaduct are long spans to minimize environmental impact, and visual continuity with the cable stayed bridge. One span in particular

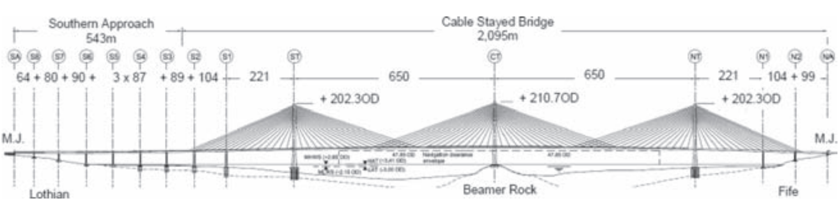

Figure 2. General arrangement (orthotropic deck variant) [dimensions in meters].

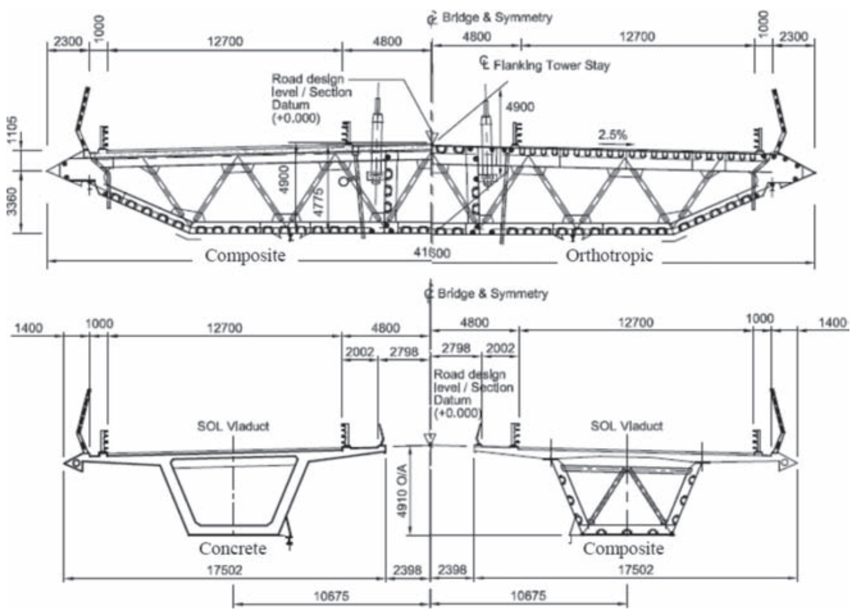

Figure 3. Deck sections, showing all variants [dimensions in millimeters].

must cross the Port Edgar Barracks and adjacent road in a single span of 90 m. The aesthetic requirements are achieved by a pair of constant depth box girders supported on V-shaped piers. The transverse separation of the carriageways is constant, and this also suits the road geometry to either side of the main crossing.

During preparation of the Specimen Design it became clear that there was no clear advantage to distinguish between all-steel orthotropic and steel-concrete composite construction for the cable stayed bridge deck box. Therefore both options have been worked up as design solutions, and the contract permits either to be adopted. The heavier composite deck variant has stay cables spaced at typically 16.2 m whereas for the orthotropic variant a typical spacing of 25 m is adopted.

Similarly for the approach viaduct the choice between composite and prestressed concrete construction for the twin boxes was not driven by significant cost difference, so designs for both variants were completed and the option left open. For the composite option, incremental launch construction is assumed, whilst for prestressed concrete option a construction sequence using in-situ balanced cantilevering is assumed.

Figure 3 shows all variants for the deck sections—the cable stayed bridge deck box at the top, composite option on the left, orthotropic option on the right; and the approach viaduct deck below, concrete option on the left and composite option on the right. The tower elevation is shown in Figure 4.

4 BASIS OF DESIGN

The Forth Replacement Crossing is the first major bridge in the UK to be designed to the Eurocode, implemented in April 2010. Work on the Specimen Design commenced in early 2008 when not all of the UK National Annexes to Eurocode and other implementation documents were available. The design criteria to be used for the structural design are set out in a project specific Design Basis document which was updated and simplified as more national documents were published. The final version forms part of the Design and Build

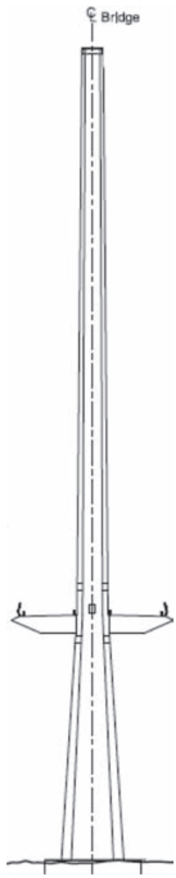

Figure 4. Tower elevation.

contract, providing additional rules and criteria appropriate to the bridge as well as clarifying how some of the Eurocode rules should be interpreted.

Aspects such as the site specific wind climate and the design criteria for ship impact have been defined. Historic wind data from measurements on the existing Forth Road Bridge over a period of 30 years was analyzed along with models to account for the local terrain to define the minimum design wind speed and turbulence characteristics. The design mean hourly wind speed at deck level was taken as 31.9 m/s. A risk assessment on the likelihood of ship impacts and their potential effect, accounting for the complex geometry of the navigation channels in the vicinity of the site, is described in Carter et al., 2010.

Due to the marine environment, and considering unexpected deterioration of the existing road bridge, the design for durability requires careful consideration. The choices left open to the contractor for aspects of the concrete mix designs, and the steelwork corrosion protection system are therefore more restricted than in some Design and Build contracts. Low grade concretes are not permitted, and the minimum cover to reinforcement is specified. Stainless steel reinforcement will be used in outer layers of bars at the base of the towers within the splash zone. The outer surface of the steel deck will be protected with a paint system comprising a zinc-rich epoxy primer, two layers of MIO epoxy, and a polyurethane top coat. The inside of the deck which house the stay cable anchorages will be dehumidified so that future touch-up and repainting operations here will not be required. The upper towers will similarly incorporate dehumidification to protect the steel anchor boxes and stay cable anchorages.

5 ANALYSIS AND BEHAVIOR

The overall structural analysis was carried out using 3D global computer models (Figure 5). Additional local and semi-local analysis models were established to examine more closely the distribution of stresses and to aid in calibration of the behavior of the global models.

5.1 *Global structural models*

Separate models were assembled for each of the main Specimen Design variants to account for the variation in geometry and properties associated with each scheme. The global models encompass the entire length of the crossing from South abutment to North abutment.

Where a range of values exist for the most appropriate definition of structural properties, sensitivity analyses have been carried out adopting different values. This is the case for the foundation stiffnesses due to variability in ground conditions, and the stiffness of the towers due to potential cracking in the reinforced concrete.

The bridge is subject to movements under different loading conditions. The reference condition is defined as the completed bridge, subject to a uniform temperature of 10°C. It is in this condition that the geometry of the bridge and the road alignment are defined. Therefore the structure has virtually no resulting deformation when loaded with permanent actions and those time dependent effects which have occurred up to the end of construction.

5.2 *Stay cable tuning*

The stay cable forces were determined at the reference condition using an iterative tuning procedure with the objective to minimize the flexural moments in the deck and towers as well as deflections. With a conventional stay cable bridge arrangement, the tuning procedure is relatively determinate given these objectives, as each pair of main span stays carries the vertical component of load for a deck segment, and each side span stay pair balances the horizontal force at the tower. For a bridge with overlapping stay cables, a degree of indeterminacy is introduced in the crossing zones since each support point on the girder is now provided with two pairs of stay cables rather than one. The Specimen Design solutions have targeted an approximately equal sharing of the vertical component of load by the stay cables meeting at the common point.

5.3 *Design effects*

The design envelopes of most live load effects were determined using the 3D global model. Particular load cases were investigated in more detail, for example those which maximized the bending effects in the towers were re-analyzed including p-delta effects to find the second order moments caused by deflection.

Wind effects were calculated with a buffeting analysis to capture the interaction of the gusty wind and the dynamics of the structure, and this proved to be one of the governing actions.

Figure 5. Global analysis model (orthotropic variant).

The effects of ship impact were the subject of detailed investigations, including non-linear analyses calculating the plastic hinge rotations of the piles under the "collapse prevention" criteria.

5.4 *Semi-local structural models*

FE models with 2D plate elements representing sections of the deck were used to study the effects in the generalized span regions and at the connection of the deck to the Central Tower as shown in Figure 6.

No alterations to the global model were found necessary for overall behavior, but the effects of shear lag in the span regions result in a small increase in peak stress in the top plate above the stay cable webs. At the Central Tower connection, the expected concentrations of stresses were quantified for critical loadcases to allow the design of this junction to be carried out.

5.5 *Construction stages*

The construction sequences assumed for orthotropic and composite variants differed so that alternative techniques could be studied. In reality, either of these sequences for the main spans could be adopted for either option:

- balanced cantilever erection to mid-span, resulting in 325 m long cantilevers
- balanced cantilever method to 257 m from all towers, with a heavy lift segment, 136 m long, for the main span closures.

In both cases it was assumed that the installation of the overlapping stay cables commences after the main span has been closed. A total construction schedule of around 60 months is expected.

6 WIND TUNNEL TESTING

As wind effects play an important role in the design, extensive wind tunnel tests were carried out throughout the Specimen Design phase. The aims of the studies were to:

- determine suitable cross sectional shapes for the deck and the towers,
- determine the aerodynamic properties,
- identify effective wind shields, and
- confirm the aeroelastic stability of the bridge.

Preliminary tests were carried out at BMT Fluid Mechanics in Teddington, UK. More extensive tests at larger scales were then undertaken in the 14 m wide by 4 m high wind tunnel at the Politecnico di Milano in Italy.

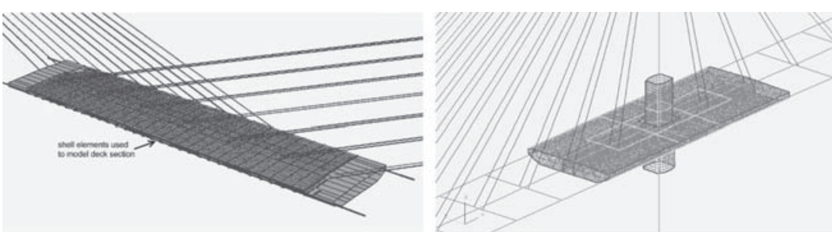

Figure 6. Semi-local analysis model (orthotropic deck variant).

6.1 *Preliminary wind tunnel studies*

As part of the option selection process, four different types of deck sections were tested at 1 to 50 scale to investigate the aerodynamic stability and force coefficients. At the early stages, ladder beam decks were included in the investigation as they may have proved cost effective.

Mitigation measures were required to improve stability including edge fairings and partially open central vents. The risk of aerodynamic problems for these decks was reduced, but not eliminated, and they were not progressed beyond the preliminary stage once the box option had been selected.

6.2 *Wind shielding study*

The existing road bridge is subject to restrictions and occasional closures due to the risks that strong winds pose to traffic. In the past, accidents have occurred where high-sided vehicles, such as empty curtain-walled trucks, have toppled over and caused damage and significant disruption. Therefore the design criteria for the replacement crossing include enhanced reliability so that the bridge can remain open in strong winds.

Wind shielding along the edges of the deck will be provided, but a balance is required to determine the level of protection to vehicles without increasing the forces that the structure must carry beyond reasonable levels. Performance criteria were set to select wind shields which would achieve conditions on the bridge which are no worse than the conditions that would be expected on typical approach roads around the site. A wide range of wind shield geometries were tested on a 1:40 scale model of the deck section. The wind shields selected were 3.44 m high with 6 horizontal slats, each 300 mm high, which provide the required reduction in overturning moment for vulnerable high sided vehicles.

Additional localized shields are proposed near the towers to improve conditions for vehicles travelling in the fast lanes, which may be affected by local changes in conditions due to the three-dimensional air flow around the towers.

6.3 *Confirmatory study*

Large scale tests of the final deck and tower sections at 1:30 scale were conducted to confirm the aerodynamic stability and the force coefficients. The deck (Figure 7a) was found to be aerodynamically stable up to and beyond the ultimate design wind speed expected at the site. Cut outs in the corners of the tower section were found to greatly improve the aeroelastic behavior of the towers.

6.4 *Full aeroelastic study*

A 1:170 scale model of the cable stayed bridge was used for the full aeroelastic tests. Key stages of construction were investigated (Figure 7b) as well as the completed bridge in smooth and turbulent wind flow proving aerodynamic stability up to and beyond the design criteria.

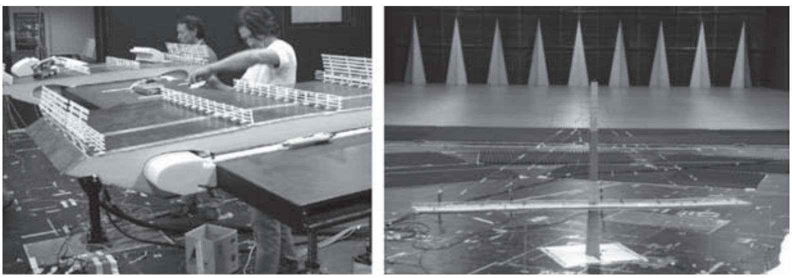

Figure 7. Wind tunnel testing: a) confirmatory deck section. b) balanced cantilever from central tower.

7 VERIFICATION AND DESIGN DETAILS

7.1 *Stay cables*

The sizing of the stay cables is generally governed by the serviceability limit state (SLS) load combination comprising permanent actions together with traffic load model 1 as the leading variable action and wind as an accompanying variable action.

The stay cables will incorporate surface treatment to limit rain-wind induced vibrations, and additional damping has been specified to ensure vibrations from other phenomena are minimized. As both the surface treatment and the details of the damping are dependent on the preferences of the stay cable supplier, a detailed solution has not been presented as part of the Specimen Design. Minimum performance requirements specified in the contract will need to be achieved.

7.2 *Decks*

7.2.1 *Steel panels*

A major part of the deck section checks of both variants of the cable stayed bridge is the verification of slender steel stiffened deck plates. There is a significant transverse stress in the steel plates due to the transverse bending, which for the bottom flange reduces the overall section longitudinal capacity. BS EN 1993-1-5 gives two methods for checking plates. Generally, the effective area method was used to check the capacity of the deck section with an additional reduced stress method check of the individual sub-panels which are subject to significant transverse stress.

The top plate of the deck is typically 14 mm thick with 336 mm deep 8 mm thick trough stiffeners detailed for fatigue loading. The bottom plate varies in thickness along the bridge from 10 mm to 24 mm to account for the different load effects at different sections. Typical stiffeners are troughs 314 mm deep and 6 mm thick. Diaphragms are formed as trusses rather than plates as this was found to be more economical and provides a much more open space within the deck. For the orthotropic deck, diaphragms are spaced at 5 m, whereas for the composite deck a spacing of 4.05 m is adopted, to provide 5 or 4 sub-panels between each stay cable support point, respectively.

7.2.2 *Concrete slabs*

Since the deck elements have significant compression forces, the effects of slenderness are important. The global axial compression can induce additional bending moments in the concrete slab due to initial imperfections. Further bending moments can be induced due to deflection of the slab under local wheel loads and long term displacements of the slab due to creep. The Eurocode gives three methods for checking the second order effects—nominal stiffness, nominal curvature and the general method. It was found that the nominal stiffness and nominal curvature methods were very conservative and there is sufficient benefit, in terms of reinforcement quantities, to perform a refined analysis using the general method.

The composite deck option incorporates transverse prestressing in the deck slab to prevent cracking and maintain the torsional stiffness of the box.

Table 1. Stay cable data.

	Orthotropic deck variant	Composite deck variant
Number of stay cables	192	288
Cable sizes	56–123 strands	42–126 strands
Total cable tonnage	3640T	5670T

7.3 *Towers*

The analysis and subsequent design of the towers was carried out using the general method in BS EN 1992. The effects of geometric imperfections were assessed by analyzing the tower with a deformed shape equivalent to the first bucking mode. The magnitude of the deformation was equal to the maximum construction tolerance estimated to be no more than 200 mm at the top of the tower. The effect of imperfections was found to be very small compared to the static deflections due to wind load.

The variation in stiffness of the tower cross section about each orthogonal axis due to the cracked section properties was determined for various load cases. An average profile of cracked section stiffnesses representative of the critical load cases was adopted in the analysis.

7.4 *Piers*

The piers to the main bridge side spans and approach viaducts are conventional reinforced concrete elements. A V-shape has been chosen to support the decks while allowing a single foundation at each pier location. Steel tie-beams are provided between the pier legs at pier head level.

The inclination of the pier legs results in bending and therefore there is flexure cracking under quasi-permanent load combinations. As a result, the crack width check is the most critical design check for these elements. Crack widths at the serviceability limit state were determined by the method reported in the UK National Annex to BA EN 1992.

7.5 *Approach viaducts*

The overall scheme has been sized to maintain the same structural depth for the approach viaduct as for the cable stayed bridge deck, and both variants for the twin box girders have the same external shape. The composite boxes have cross frames typically every 8 m, with stiffening ring frames in between these. Trough stiffeners similar to those in the main deck are used. The concrete boxes consist of segments between 3.6 m and 4.0 m long, depending on the length of individual spans.

7.6 *Foundations*

Ground conditions vary considerable across the site. The 3 main towers generate the highest foundation loads. There are in addition 10 side span and approach viaduct piers with smaller loads, seven of which will be within the estuary and the remainder on land. In addition to service loads, the foundations within the estuary are required to resist accidental ship impact (Carter et al., 2010).

Beamer Rock is a dolerite outcrop which will support the Central Tower. A pad foundation with overall dimensions of 35 m by 25 m has been designed to be recessed into the rock. Construction works within the estuary can be minimized by using a cellular structure cast off site. After blasting the rock and positioning the foundation, the cells would be infilled to complete the base.

For the flanking towers, piled foundation solutions have been designed incorporating large diameter piles. As large foundations are required, it was efficient to maximize the size of the piles to limit the number of construction operations. Hence 3.4 m diameter piles were incorporated into the design. The South Tower is located in about 22 m of water, with rockhead at around –40 m OD. At the North Tower the water depth is around 5 to 8 m, and the rockhead at typically –34 m OD.

At pier locations with rockhead near the surface, pad footings are the simplest form of foundations. The pads vary in size to suit the different loading conditions and ground

Figure 8. Nighttime visualization of the Forth Replacement Crossing.

conditions at each pier. A number of the piers require deeper foundations and piled solutions have been designed with typically 2.5 m diameter piles.

Under the Design and Build contract, the contractor has significant scope to adopt foundation solutions according to his preferred methods of working.

8 CONCLUSIONS

The Forth Replacement Crossing is a major infrastructure project for Scotland, designed to safeguard a vital connection in the country's transport network. The crossing is located in a historic and environmentally constrained area, and great consideration has been given to selecting an appropriate scheme. Specific design criteria have been determined to complement the new Eurocode standards. Different variants of a Specimen Design have been produced, with options left open to the contractor for an orthotropic or composite main deck, and a composite or prestressed concrete approach viaduct. The Design and Build contract arrangements allow the contractor sufficient freedom in his design to adopt his preferred methods of working and bring economy to the project, whilst ensuring the final form of the bridge will not deviate from the client's expectations. Figure 8 shows nighttime rendering of the Forth Replacement Crossing.

ACKNOWLEDGEMENTS

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Chapter 2

The cable-stayed bridge across the River Sava in Belgrade

P. Walser & S. Hopf
Leonhardt, Andrä und Partner GmbH, Stuttgart, Germany

M. Steinkühler
Porr Technobau und Umwelt Aktiengesellschaft, Vienna, Austria

ABSTRACT: A new bridge in Belgrade over the Sava River illustrates unique design features, e.g. composite steel and concrete spans with a 45 m wide deck, caisson type of foundation, a slender tower and seismic design.

1 GENERAL

Belgrade, the capital of Serbia, has about 2 million residents and stands for 35% to 40% of the gross domestic product. The infamous freeway E75 known as "Autoput" runs through Belgrade connecting central with southeast Europe. The increasing traffic jams are one of the principal obstacles for further growth in the region. All the existing road bridges across the Sava River are permanently bottlenecks for the commuter traffic.

In order to solve these traffic problems and in order to improve the infrastructure of the surrounding area a ring road was planned where the new cable-stayed bridge is the key structure. The most important bridge structure under construction on the Balkans connects New Belgrade in the North with the Radnicka Street in the south and is located four kilometers upstream from the confluence of the Sava and the Danube River.

The new landmark of the city of Belgrade will carry a needle type tower with a total height of 200 m. With 40 stays on each side it holds up the main span of 376 m length and the backspan with 200 m. The total length of the structure including the approach spans will be 967 m with a width of 45 m! Side Spans and Back Span are from prestressed concrete executed by the incrementally launched method on auxiliary piers and the longer main span consists of a steel section with orthotropic deck, crossing the river at 20 m height.

In spring of 2008 the city of Belgrade awarded the contract to plan and build this structure to a joint venture of PORR, DSD and SCT under the overall control of PORR and with Leonhardt, Andrä und Partner GmbH (LAP) as Main Designer.

2 DESIGN

2.1 *Concept design*

During 2005 the city of Belgrade issued an international design competition in order to get an outstanding solution for the new Sava crossing at the tip of the Ada Ciganlija Island within eye-shot distance of downtown Belgrade. The crossing had to carry 3 lanes in each direction, 2 tracks for tramways and pedestrian and bike paths on either side.

The winner of this competition was that single tower cable-stayed bridge from Victor Markelj (Ponting consultants, Maribor, Slovenia) who developed with the support of architect Peter Gabrijelcic the typical form of the extremely slender A-shaped tower (Hopf et al., 2010; Steinkühler et al., 2010/2011). In spite of the extremely wide deck the superstructure should give a light and slender appearance and cross the river with clear and orderly lines (Fig. 1). Cross-section, cable arrangement, pier form and pier locations all where defined under this premise and in order to study in detail the appearance of the tower a large scale wood model was used. To avoid a cut off effect the tower shaft extends above the cable anchorage zone with a steel structure encased in stainless steel.

2.2 *Tendering*

With some minor modifications the concept design was the basis of a European wide tender in 2007. The deck was defined with all outside dimensions and a steel superstructure was proposed over the full length being ballasted in the side span. As according to the contract (following FIDIC yellow book) the bidders could optimize the outline design within the geometrical and aesthetical limits given in the tender, which left to the tenderers to fine tune foundations, to chose the material (steel/concrete) and last but not least to optimize for the construction procedures.

The tender requirements did not allow for temporary structures within the clearances for ship traffic nor for transport of material to the tower using the roads of the protected Ada Ciganlija Island. Site activities near the river banks had to cope with variable water levels up to 4 m.

2.3 *Tender design*

During tender the optimization focused on the foundations and on the superstructure. For the tower foundation a combination of piles and slot walls was developed. The superstructure with different spans on main and back span side was modified to avoid ballasting. In consequence the "light" steel section in the main span had to be balanced with a heavier back span deck in concrete which avoids to some extending huge uplift forces on the side span piers. On the south side there is one additional 50 m span before reaching the abutment. On the north side

Figure 1. Artists render of bridge according to tender design.

the bridge continues over 4 spans with 70–108–80–80 m. As well as the back span on the south side this portion will be a post-tensioned concrete section. In spite of two changes in material the superstructure is continuous over the full 967 m length with the fix-point at the tower.

In the tender design true care was given to the appearance of the superstructure. In spite of the two changes in material the under sight of the deck as well as the fascia should look similar throughout the length in order to make it acceptable to the client. To show the minimal effect of the material change the offer was complemented by a realistic visualization of those details.

The cross-section consists of a 14.50 m wide box of 4.75 m depth and a deck with 45 m width. The cantilevering deck sections are supported every 4 m by steel struts and similar to the steel section the concrete deck got ribs in the same rhythm of 4 meter distance. This way the wide deck allows for the requested number of lanes, rail tracks and foot and bike paths (Fig. 4).

In order to anchor the stays between roadway and tramway the box section is divided in 3 cells two smaller ones at the outside with the anchorages and a wider one in the center to carry all utility lines.

2.4 *Detailed design*

According to the tender documents the wind and seismic loads had to be determined according to local conditions. Loss of a pair of cables and the loss of a strut below the deck were defined as accidental situations with the requirement of no overall failure of the structure. The design follows generally the conditions of the Eurocodes. Where EC-Code refers to

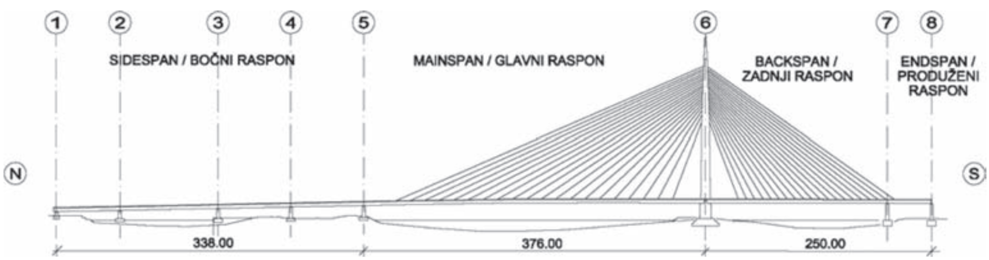

Figure 2. Elevation.

Figure 3. 3D Animation of steel-concrete transition.

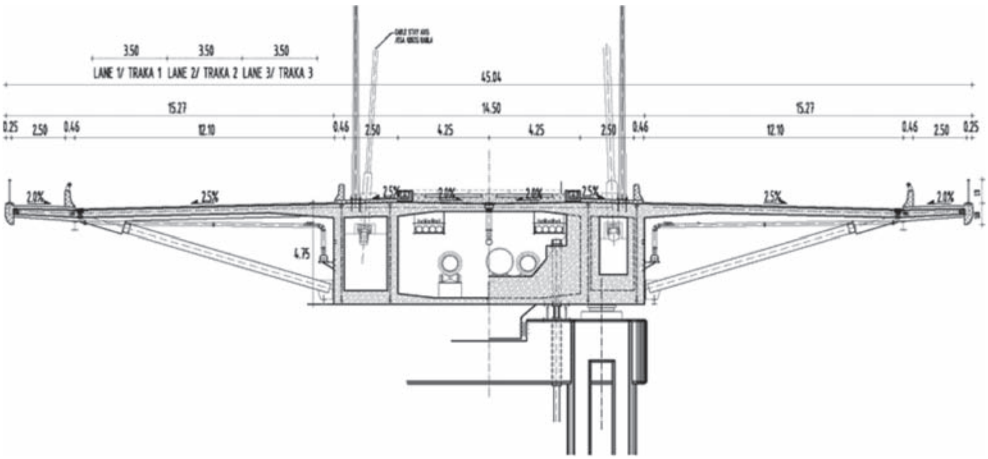

Figure 4. Cross-section of Back Span (span and support situation).

national annexes it was agreed to use the German DIN-FB as annex, as the Serbian codes have not yet reached that standard and cannot provide answers from a national annex.

2.4.1 *Seismic considerations*

Belgrade is located in an area with a ground acceleration of 0.08 g, just below the limit for minor seismic activity according to EC8. However, in the neighboring countries like Greece, Al-bania and Rumania the seismic activity is much higher. Magnitudes up to grade VII had been registered before. In spite of moderate accelerations at rock level the amplification through the soil characteristic is important at the location of the Sava Bridge and leads to considerably higher excitations of the structure. This is reflected by very large soil parameters S, which are above the highest recommended values according to EC8. By this effect the accelerations of the foundations reach more than 0.2 g and this is clearly above the limit for minor seismic activity ($ag \cdot S < 0.1$ g).

After extensive investigations in cooperation with the university of Belgrade a ground acceleration of $ag = 0.083$ g was established for a return period of 475 years and an importance coefficient of $\eta = 1.3$. The response spectra were defined by numerical simulations considering the actual subsoil situation. These simulations were made on the basis of typical registered events and as well on the basis of synthetic earthquakes for rock (soil class A). Upon request of the client response spectra for locally registered quakes had to be developed too, however, those were at the end irrelevant for the sizing of the structure.

To take the forces from seismic effects the piers in axes 1, 5, 7 and 8 were chosen to be connected to the superstructure by shear keys protruding from the deck into the piers due. At axis 6 the pylon shaft is connected monolithically to the deck. The calculations have shown that a lateral connection at all pier axes would generate higher lateral forces due to the large difference in the span lengths. To avoid uplift in axis 7 under each event vertical holddown cables are placed inside the pier shaft. Those cables with 3 times 70 strands at each pier shaft are normal stay cables which are able to accept considerable local bending caused by the horizontal displacement of the deck structure.

2.4.2 *Aerodynamics*

In order to check the aerodynamic stability tests were performed at the University of Bochum. Windtunnel tests with section models of the superstructure (Fig. 5) as well with a full model of the tower (scale 1:200) proved the stability and gave the relevant shape factors. The tower model was tested with and without crane, free standing and with cables installed. The data

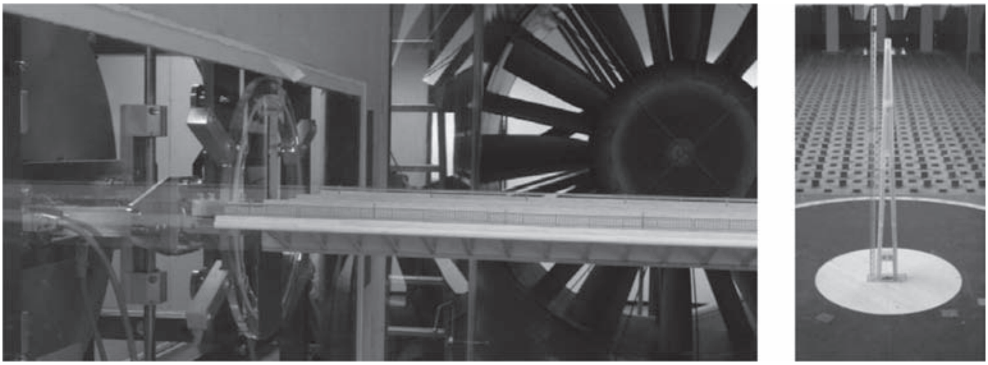

Figure 5. Wind tunnel test at section model of superstructure and a full tower model.

derived from these tests formed the basis for the analysis of the aerodynamic stability and wind loadings which could then be applied as equivalent static loads at the global model.

2.4.3 *Independent checking of detailed design*

According to the contract the detailed design had to be checked independently by a checking engineer and all documents supplied as certified documents. The independent checker (ICE) who was hired by the construction joint venture was the local consultant Ponting from Maribor as he had won the design competition, had been the author of this design and had already the responsibility to check the general compliance with his original concept design.

3 EXECUTION

3.1 *Foundation*

Under alluvium layers follow the quaternary with a sequence of silty, sandy clays over fine and middle sand even up to gravel. Down in the tertiary follows after marly soil a layer of limestone with transitions to alternating sandstone and siltstone layers (Steinkühler et al., 2010).

Based on those preliminary geotechnical investigations the characteristics of the subsoil could be defined and the suitable foundation type determined. Deep foundations were chosen for axis 1 to 5 and 7 with bored piles of diameter 1.20 m and 1.50 m up to a depth of 30 to 40 m. The foundation was verified during construction by 4 pile tests (Osterberg Load Cells used in 3 level tests). The resulting small settlements confirmed all upfront made assumptions.

For the foundation of the 200 m high tower a sort of caisson was chosen at an early stage of tender design. A ring wall, up to 40 m depth, of 1 m thickness and with 35 m diameter was constructed as slot wall. This served to stabilize the 10 m deep open cut and as well to transfer loads to a lower level. Inside the excavated pit piles of 1.5 m diameter were placed to the same depth as the ring wall, providing together with the encased soil a monolithic pot or embedded caisson. The 9 m deep pile cap, placed in three stages, transfers loads to the piles and the ring wall.

The settlements are smaller than by groups of piles and the seismic safety is considerably higher. A pile cap with the depth of 9 m distributes the load to the tower. Special attention was given to the connection between the pile cap and the surrounding wall in order to allow for a load sharing between the piles and the wall. The tower foundation consists in total of 16,000 m^3 concrete.

Temporary piers were used for the launching of the concrete spans and those were founded on pile groups with pile diameters 1.50 m. Of the seven foundations five were built inside the water which required the use of steel casings driven into the ground. For excavation of the piles a bentonite suspension was used. After launching and installation of the stays those temporary structures will be removed. Therefore the temporary piers were built by prefabricated elements, connected by dry joints and prestressed in vertical direction. It is envisioned to cut the piles under water by sawing technique right at the elevation of the river bed.

3.2 *Superstructure back span*

The 200 m long post-tensioned structure spans the Cukaricka Bay, a small branch of the Sava River. It is cast in situ on the south side about 20 m above ground and launched by the incrementally launching method (ILM) over temporary piers at 50 m distance towards the tower. The slope is 0.2% upwards and the segment length being cast is typically 18 m. For each segment about 630 m³ of concrete must be placed.

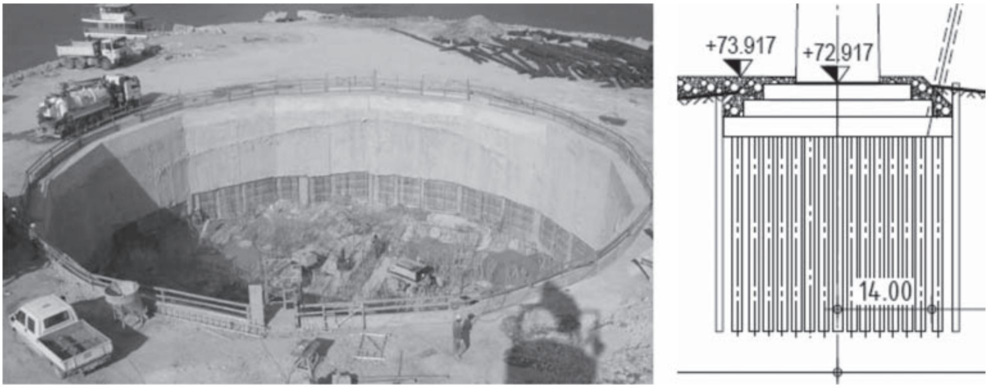

Figure 6. Tower foundation, construction pit with 35 m diameter, slot wall and top end of bored piles.

Figure 7. Incrementally launched Back Span, elevated construction yard.

Due to the size of the section and the time constraints, the elevated construction yard was divided into 3 zones. The 1st zone to produce the 14.50 m wide bottom plate of the box, the 2nd for the webs and top flange including the stay anchor area and in the 3rd zone the section was completed by the rest of the roadway with the cantilevering slabs supported by struts. This way the activities could run parallel in all 3 zones. During the last launch the complete concrete superstructure weighed about 20,000 tons. For the launching operation two twin hydraulic launching equipments were placed on axis 7. Due to the exceptionally high stiffness of the superstructure and the jack capacity of $4 \times 800 = 3{,}200$ tons the vertical lifting capacity was limited to a few millimeters. As this is typically needed to remove the structure from the breaking saddle, the saddle itself was transformed to be used alternatively as sliding support.

At the temporary piers one launching bearing was required under each web and as all 4 webs had to participate fully in carrying the loads, always two supports were coupled by a rocker block, which is the simplest way to guarantee a 50/50% distribution.

Once the last temporary support was reached, the launching nose could be dismantled in several parts. The launching nose had been designed and produced especially for this bridge as none of the used ones available in the market could provide the high bending capacity of 200 MNm. It was applied twice within this project at the Back Span with 9 m distance between girders and at the Side Span with 13.5 m distance between girders. The bracing between the two girders of the nose could be adjusted to both girder separations.

The closure pour between the Back Span structure and the pier table of the tower was done by the help of traditional formwork. After transferring the fixed point from axis 7 to the tower axis the closure section could be poured in two steps, one for the boxes and a second for the slabs. After that closure the back span is completed by another 50 m structure towards the transition pier axis 8 constructed within the elevated construction yard as before. The auxiliary piers of the Back Span can be removed after installation of the stays, shortly after completion of the main span.

3.3 *Tower*

The tower with its tip in the form of a needle and over 200 m height above ground will become the new icon of Belgrade. It has the form of a tapered cone resting on two tower shafts which pass through the deck below. The single cone starts at elevation +98 m and carries all stay anchors. The radius of the cone varies between 8 m at the bottom and 2 m at 175 m height, which is quite a challenge for the climbing form (Figs. 8 and 9).

Up to the height of the deck the tower stem was cast in 5 steps of 4.39 m and then continued with 34 steps of 4.59 m until reaching the final height of the concrete section at +175 m. In order to save time the power joint zone (connection zone with the superstructure) was built using the same climbing formwork. The connection to the deck is made by mechanical couplers and post-tensioning bars fed in later on. During construction of the 2 separate stems of the tower 4 temporary struts were installed in between. At the end, one of the two forms had to stay behind, as there was not sufficient space for both of them.

The anchorages of the first 10 stay levels correspond to rather steep stays and were chosen to be anchored right in the concrete and closing the horizontal force component with a circular post-tensioning. For the next 10 levels of anchors a solution with steel cages was adopted working as composite section with the surrounding concrete shaft. Due to limited space in the upper part of the cone and the spacious situation below two different anchorage systems were the appropriate answer.

Concrete grade C50/60 was used for the tower with higher grade C55/67 in the power joint area. All placement of concrete was done by the tower crane with the help of a remote controlled bucket of 3 m^3. At 150 m height one cycle took about 15 minutes which means to place 120 m^3 it took about 10 h. The rather slow process of concrete placement resulted in a high quality concrete. The crane is self-climbing and attached at several elevations to the

Figure 8. Climbing form of the upper cone at stay anchorage area.

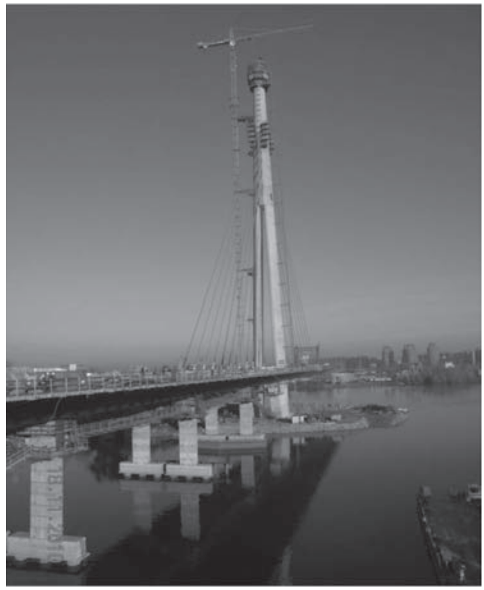

Figure 9. Tower, stage 36 of 39, elevation +161 m, November 2010.

tower. It reaches with its hook to a final elevation of +210 m allowing the installation of the steel tip of the tower too.

The final section of 25 m height composes of a steel structure covered with stainless steel sheets. Due to the limited lifting capacity of the crane the steel section is subdivided in 3 elements of less than 12 t weight. The upper end section is purely an architectural element without any structural function. For sizing of that steel tip the seismic loads were governing due to the high dynamic acceleration at this elevation. The size of the top platform is of diameter 1.5 m and can be reached from below, carrying aircraft warning lights and all sorts of lightning protection and fixations for inspection and maintenance.

One of the two shafts carries an elevator, the other one a series of ladders.

3.4 *Superstructure main span*

3.4.1 *General*

The 7,800 tons of steel grade S355 J2+N of the superstructure came from Chinese fabricators. The steel sections were fabricated with the necessary float and were shipped in 7 transport units of about 1000 tons in suitable sizes to Amsterdam/Rotterdam. There they were loaded on barges and shipped through the Rhine river, the Main river and then through the Main-Danube canal to the Danube and finally to Belgrade to be unloaded at the site itself.

On the island Ada Mala near the side span a 200 m long area was prepared for site fabrication at an elevation above highest water. The area is served by a portal crane with 48 ton capacity and a span of 30 m. With these on site fabrication facilities the sub-elements could be stored, preassembled to larger elements and finally the complete segments of 16 m length produced. The steel section (Fig. 10) consists of a 3-cell section from orthotropic plates, the wide cantilevers including secondary longitudinal beams which carry as well the rails for the inspection trolley and inclined struts every 4 meters.

The two smaller boxes at each side carry the stay anchorages. The stays apply their forces on transverse box beams which are sealed airtight. The web thickness of the main boxes reaches 35 mm in the area of the stay anchors and the minimum thickness of the plate of the

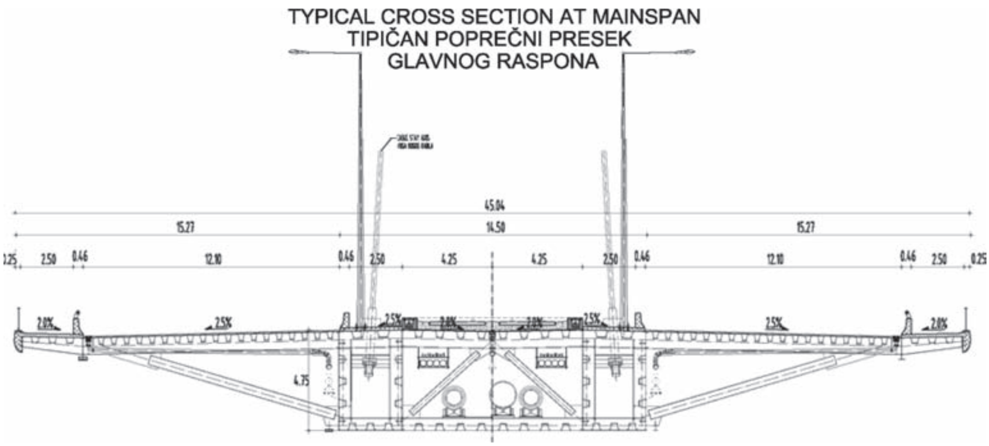

Figure 10. Typical main span section of 45 m width.

orthotropic deck is 14 mm which is stiffened by trapezoidal stiffeners and supported every 4 m by cross-girders.

The 8.5 m wide central box receives diagonal stiffeners every 8 m and this wider box is planned to carry many utility lines hanging from the top and 3 water pies up to 1 m diameter resting on the bottom flange.

The concentration of stresses in longitudinal and transverse direction, especially in the area of the connection of the diagonal struts to the box required a close cooperation between designer and fabrication. In order to confirm the feasibility of those complicated details in the shop a simulation of the fabrication was done upfront and in parallel to the design.

3.4.2 *Free cantilevering erection*

With the erection of the transition element in front of the tower started the free cantilevering deck erection. The 17.1 m long transition piece weighs 505 tons and was placed with the help of mobile cranes in the formwork for the cast-in-situ pier table. Continuity will be achieved by tendons anchored in the steel and concrete sections and by shear studs (Fig. 11). Later on all 21steel segments were erected as typical with the free cantilevering method, lifting the segments from barges.

As said before the prefabricated sub-elements delivered by barge are assembled to complete segments on the site near the bridge location. The corrosion protection of the 45 × 16 × 4.75 m elements was applied as well on site and then loaded by Kamag transporters on a barge of the size of 80 × 15 m with a capacity of 1300 tons (Fig. 12). By help of tug boats and winches the segments were pulled to the lifting location. The lifting is done by a heavy lift system on top of the derrick and located at the tip of the cantilever. The lifting capacity of 400 t (Fig. 13) corresponds to segment weights up to 360 t.

The derrick is moved to the new segment once the welding is completed. After this operation the new pair of stays can be installed and the load transferred to tower and back span. The segment length was given by the regular stay distance.

3.5 *Stays*

Design, testing, procurement and installation of the stays follow the rules established in Fib-Bulletin 30 (2005) and PTI-Recommendations (2007).

The High Am Cona stays, up to 373 m long, are provided by VT-BBR (Fig. 14). Acceptance tests were performed for 3 different sizes, a small one, a midsize and a large one. Each

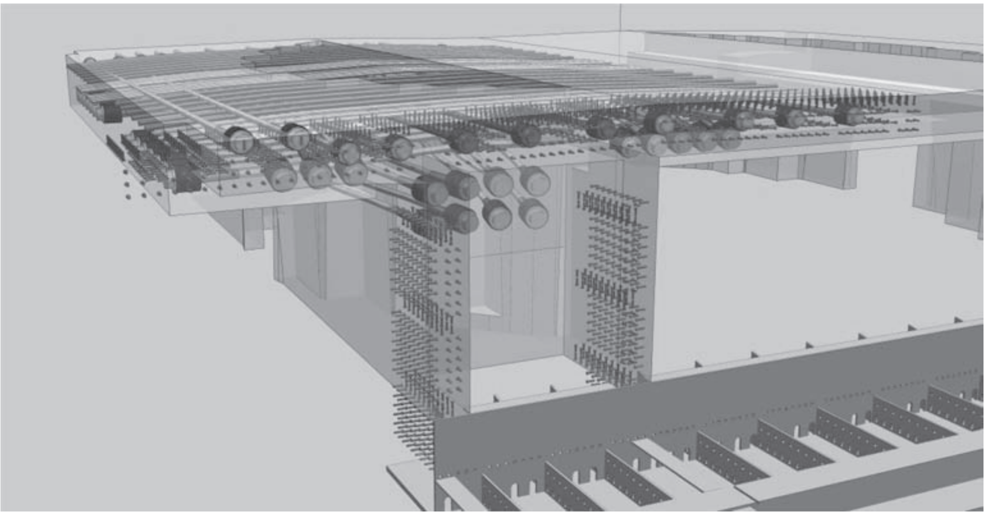

Figure 11. Visualization of the transition steel—concrete.

Figure 12. Transport of a 350 ton segment on Kamag transporters.

strand is galvanized, the interstices greased and sealed by individual PE sleeves. The bundles of strands are protected by a HDPE pipe with diameters up to 300 mm. All 80 stays together consist of 1,280 tons of steel and the biggest one carries 88 strands. Due to carefully planned guides and deviation devices in the anchorage area any angular deviation is kept outside of the wedge anchorage of the strand which reduces the combined stresses due to tension and bending considerably.

The dead anchors are located in the tower and the live anchors in the superstructure. Each live end anchor rests on a ring nut which allows for later force adjustment, if required. The initial stressing of the strands is done by mono strand jacks. After a pair of tendons is tensioned the achieved cable forces and uniformity is checked by lift-off tests of a number of strands. In order to avoid effects from temperature and solar radiation, the tests are made

Figure 13. Free cantilevering erection, lifting of segment M7.

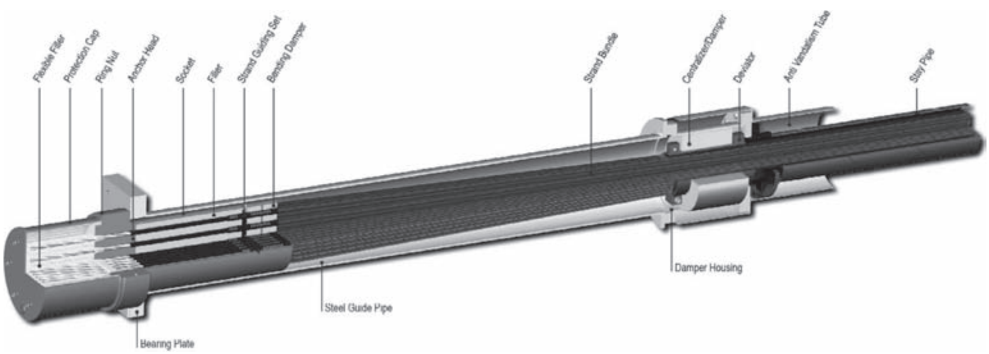

Figure 14. BBR High Am Cona (BBR/VT) anchorage details.

early in the morning and are completed by parallel measurements of the geometry (tower and deck).

First the Back Span cables are installed, followed by the Main Span cables. Due to the extremely unusual ratio between self weight of the steel (200kN/m) and superimposed dead load (242 KN/m) the installation has two steps of tensioning, a first one during free cantilevering. After completion of the erection a portion of the superimposed dead load (SDL) is applied and the overall geometry checked. Then every 2nd stay will be tensioned by mono strand jack to final length, a new measurement of the achieved geometry done and the rest of the SDL placed on the deck. Then the remaining stays will be re-tensioned by mono strand jacking. Between the 2 (3) stages of stressing the as-built situation (geometry and forces) is analyzed and respected by the next step of stressing.

An extensive program to control geometry and forces is followed up, deformations resulting from each new erection stage are reported to the designers and deviations are analyzed.

To prevent cable vibrations of the long stays each one is equipped by friction dampers at the lower end, just above the deck.

3.6 *Superstructure side span*

This 358 m long section of the superstructure will be launched with a slope of 2% upwards in 19 increments with segment lengths of 16 and 20 m (Fig. 15). Between pier 1 and 2 the alignment requires a change in transverse slope and a spiral curve is starting. This is achieved by changing the width and inclination of the cantilevering deck sections without any modification of the boxes itself for reasons of launching.

Between the normal piers one or two temporary pier were placed which reduced the span lengths for launching to 36 and 40 m. The superstructure of the Side Span consists of a single box and is prestressed longitudinally with straight and curved tendons in the webs. The latter ones are fed in after launching and then the auxiliary piers can be removed. A special challenge

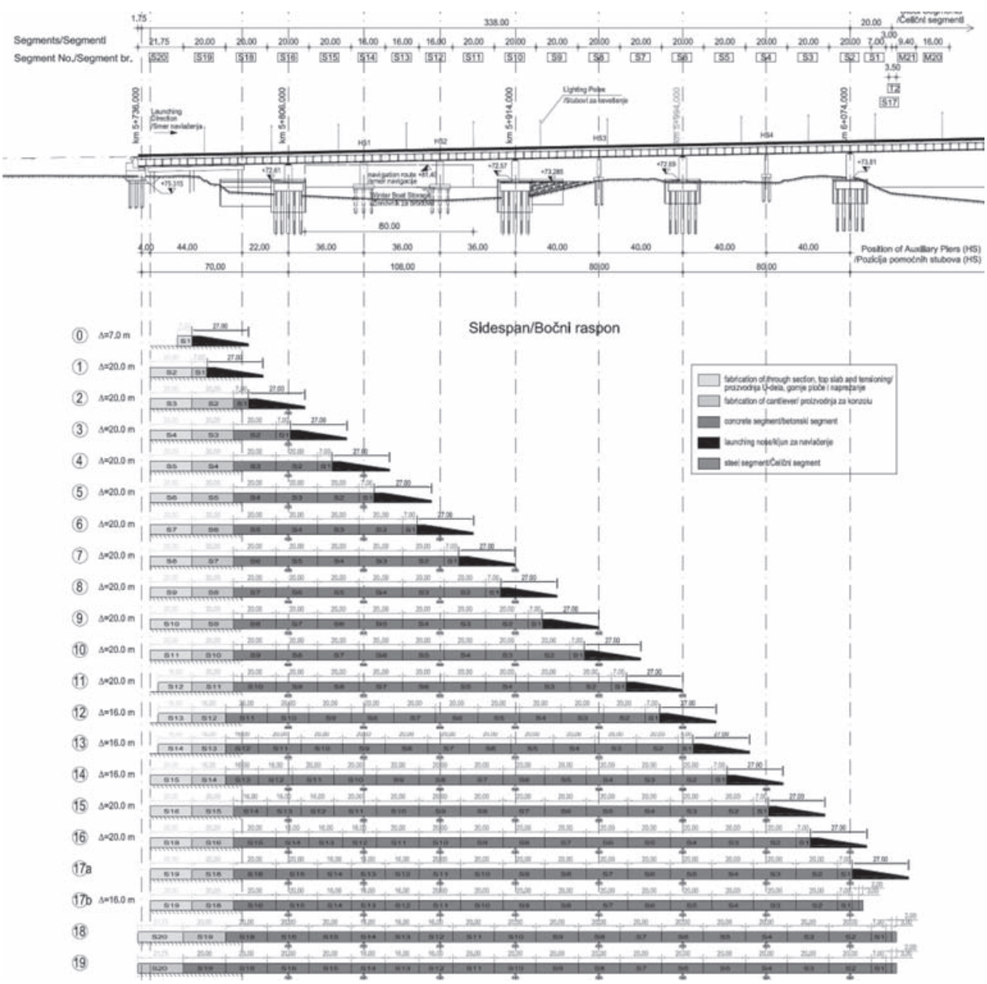

Figure 15. Incremental launching in 19 steps and segment lengths of 20 m.

results from the camber sizes for up to 108 m long spans of variable length, launched over shorter spans than final. To compensate the adverse effects from camber during launching, wedge plates up to 30 cm thickness were used. To produce the structure with the required camber the stationary form had to be adjustable in elevation.

The deck slab of 45 m width is prestressed from edge to edge. As in the Back Span the section is cast in 3 stages but on two zones, since more construction time was available: bottom slab with webs in stage 1, followed at the same place by the deck slab of the box, than launching took place and the cantilevers supported by the struts were erected in zone 2. The cycle time was 2 weeks.

For this 30,000 ton superstructure unit two twin hydraulic launching equipments with a lifting capacity of 4400 t were installed at pier axis 2 (Fig. 16). In order to support the launching reaction of 1800 t, the pier in axis 2 had to be braced against the casting yard.

After reaching pier 5 the steel nose will be dismantled and the deck launched into its final position. Then the last segment will be produced in the casting yard. For closure in the main span the last steel segment is lifted and welded. With temporary connections between both structures—the concrete Side Span and steel Main Span—they will be fixed in their position and the closure gap poured and prestressed by tendons. This closure is planned in August 2011.

3.7 *Finishing*

Towards the end of Side Span construction the remaining activities start on the deck. Fascia and barriers at both sides of the roadway will be from precast elements. Steel fascias in the main span were avoided for reasons of balance between main and Back Span and for continuity reasons. After the placement of these finishing elements the auxiliary piers are removed and the actual weight situation in both spans can be evaluated based on the deformations from removal of temporary supports. As both, concrete and steel deck can deviate from the theoretical weight it is planned to compensate any deviation by ballast concrete in the backspan. This ballast would be poured in the two smaller boxes with the stay anchors.

Besides the normal street light the bridge gets an installation of architectural lighting which will underline the tower, stays and superstructure without disturbing the traffic on deck.

Figure 16. Launching of side span situation in November 2010.

4 CONCLUSIONS

In 2008 a consortium of contractors PORR, DSD and SCT was awarded the contract to design and build the bridge over the river Sava. Start of construction was in April 2009.

The size of the superstructure, the combination of steel and contract in the stayed area, the use of incremental launching combined with elevated casting yard and a construction time of only 3 years are a multiple challenge for design and execution. The handover is planned in summer 2012. You can observe continuously the progress under www.savabridge.com

ACKNOWLEDGEMENTS

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Construction:	JV Ogranak Sava Most: PORR (Vienna, Austria), DSD (Saarlouis,Germany), SCT (Lubjana, Slovenia)
Tender, detailed Design and Construction Engineering:	Leonhardt, Andrä und Partner GmbH (Stuttgart, Germany)
Preliminary Design and Checking:	Ponting (Maribor)
Stay Cables:	VT Vorspanntechnik GmbH & Co. KG (Salzburg, Austria)

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Chapter 3

Indian River Inlet Bridge—A design-build in progress

E. Nelson
AECOM, Nashville, TN, USA

ABSTRACT: The existing bridge across the Indian River Inlet in Delaware has long been plagued by extreme scour conditions within the channel. To improve the safety of the crossing, construction of a new bridge is ongoing under a design-build contract. The new design addresses two key challenges of the site. First, the main span was set to keep the substructure for the new bridge out of the channel in order to mitigate the threat posed by scour. Second, the bridge must be highly resistant to corrosion due to its proximity to the Atlantic Ocean. These requirements dictated the design of a concrete cable-stayed bridge, which has a 950-foot main span across the inlet. A significant portion of the bridge is constructed on falsework using precast floorbeams. Only the central two-thirds of the main span is erected with a form traveler, thus simplifying and accelerating the construction.

1 HISTORY OF THE SITE

The Indian River Inlet Bridge is located on the Delaware coast near the town of Bethany Beach as shown in Figure 1. It is part of Delaware State Route 1 (SR1) and is the primary north-south link along the coast. The inlet itself connects the Indian River Bay to the Atlantic Ocean and is subject to substantial outflows. The location of the inlet naturally meandered until 1938 when the U.S. Army Corps of Engineers constructed parallel jetties to create a stable 500-foot-wide inlet that provided navigation for recreational boats. The first bridge across the inlet was a timber structure constructed in 1934. Since that time there have been four bridges at the site, with the current steel girder bridge being built in 1965.

2 BACKGROUND AND OVERVIEW OF NEW BRIDGE

The need for replacement of the current bridge was primarily driven by the strong scour in the channel, which has degraded the existing bridge foundations. An aerial photo of the

Figure 1. Bridge location.

existing bridge is shown in Figure 2. The Delaware Department of Transportation (DelDOT) has been monitoring the condition of the existing bridge closely, while at the same time was pursuing the design of a new bridge that would place the foundations out of the current channel and any future anticipated channel widening. The design process ultimately culminated in a request for Design-Build proposals, with the team of Skanska USA Civil Southeast and AECOM being selected in the fall of 2008.

The replacement bridge has a total length of 2,600 feet. The approach structures on either end consist of four bulb-T girder spans, each 106'-3" in length. The main bridge is a three span concrete cable-stayed structure with a main span of 950 feet and side spans of 400 feet. The general plan and elevation of the structure is illustrated in Figure 3 below. The remainder of this paper will focus on design features of the cable-stayed structure.

Figure 2. View of existing bridge.

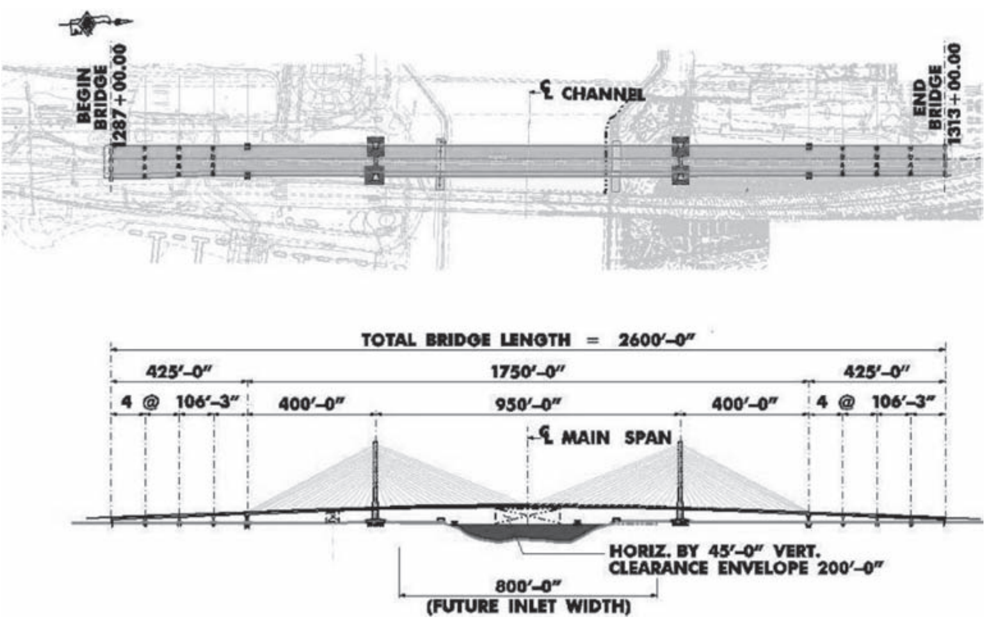

Figure 3. General plan & elevation.

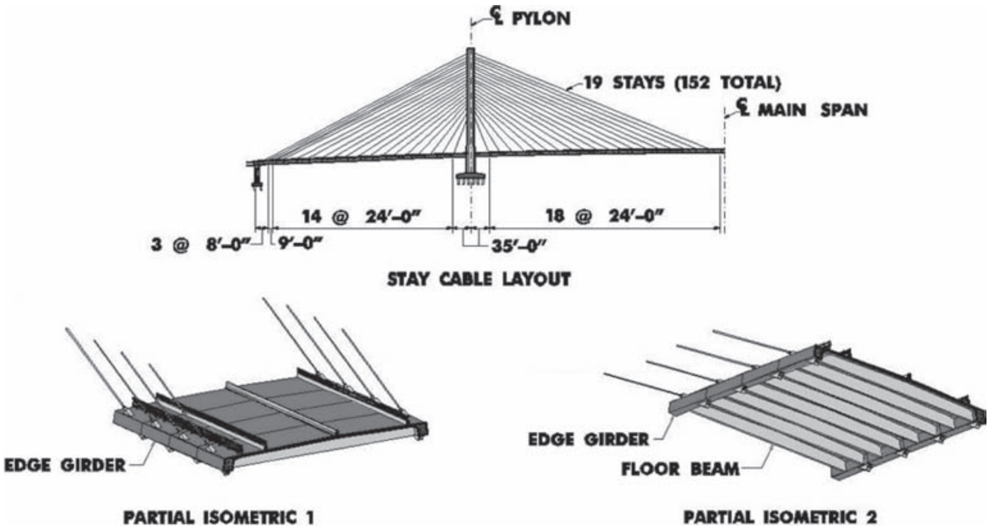

Figure 4. Cable stay configuration.

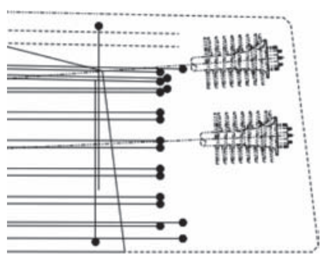

Figure 5. Floorbeam connection at edge girder.

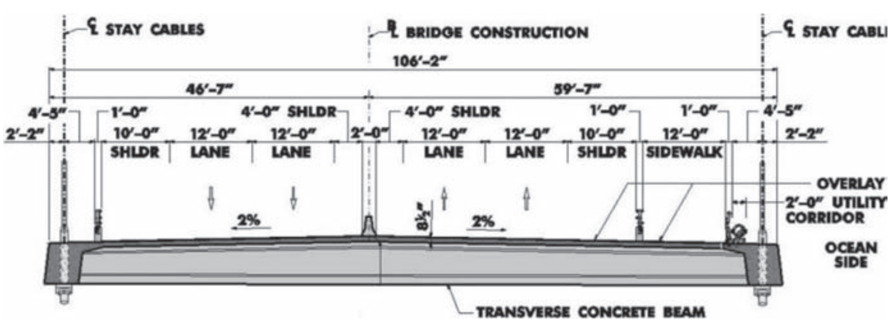

Figure 6. Typical section.

The main span structure is supported by two planes of stays, each anchored into longitudinal edge girders. Anchors are typically located at 24 foot intervals along the deck, as shown in Figure 4 below. The edge girders are connected at 12 foot intervals by concrete floor beams, which are either precast or cast-in-place. The upper ends of the stay cables are anchored into hollow concrete pylons, which consist of steel boxes that are connected to the concrete pylon with shear studs. The two pylons are connected transversely only at the foundation leve.

Both the precast and cast-in-place floorbeams are connected into the cast-in-place edge girders with mild reinforcing that extends into the girders, as shown in Figure 5 below. All floorbeams are also transversely post-tensioned with the anchors located within the edge girders.

Figure 6 illustrates the roadway cross section of the bridge, which includes four lanes of traffic with shoulders and a 12-foot sidewalk.

3 KEY DESIGN CONSIDERATIONS

The new bridge was designed in accordance with the Fourth Edition of the AASHTO "LRFD Bridge Design Specifications" (AASHTO 2007), and supplemented by the Post-Tensioning Institute "Recommendations for Stay Cable Design, Testing and Installation", Fifth Edition (PTI 2007) as well as the project specific design specifications that were included with the DelDOT Design-Build Request for Proposals (DelDOT 2008).

3.1 *Design life*

Given the proximity of the bridge to the Atlantic Ocean, the ability of the structure to withstand the corrosive marine environment was a high priority. This was clear in the project specifications that included several requirements intended to extend the life of the structure.

One of the primary requirements related to the durability of the structure is the allowable stress in the concrete deck. The deck is essentially designed as a fully post-tensioned structure during all stages of erection and during the design life. This includes both longitudinal and transverse stresses. Additionally, the deck is designed for zero tension under dead loads (including the effects of creep and shrinkage). For other concrete elements, such as the piers, pylons and footings, the crack control provisions of the AASHTO LRFD code are considered for the most stringent conditions.

In addition to the concrete design parameters, the project specifications allow for minimal exposed steel. In the areas where exposed steel is unavoidable, such as at the bearings, all replaceable steel elements are metallized, and all permanently embedded items are made of stainless steel. Metallizing is a process to thermally spray zinc coatings directly onto the steel surfaces, which significantly enhances corrosion protection of the steel over common paint systems. Further measures that provide an extended design life include a modified latex concrete overlay, low permeability concrete, and epoxy coated reinforcement throughout the structure.

3.2 *Scour*

Given the history of significant scour at the site, the long-term potential for additional scour was included in the design. Contours of the existing scour depths at the inlet are shown in Figure 7. Extensive studies were carried out by AECOM, including studies of the storm histories in the area, as well as other parameters that would influence the potential for scour. In the final analysis, it was determined that the maximum anticipated contraction and local scour was approximately 35 feet below finished grade. As a result of this, the design team analyzed the bridge with both scoured and un-scoured foundations, and used both sets of results to envelope the design loads.

3.3 *Wind*

Wind is often one of the controlling parameters in the design of any cable-stayed bridge, both in terms of the design forces for the bridge and for ensuring stability. The location of the Indian River Inlet Bridge is susceptible to Atlantic hurricanes, so the wind actions applied to

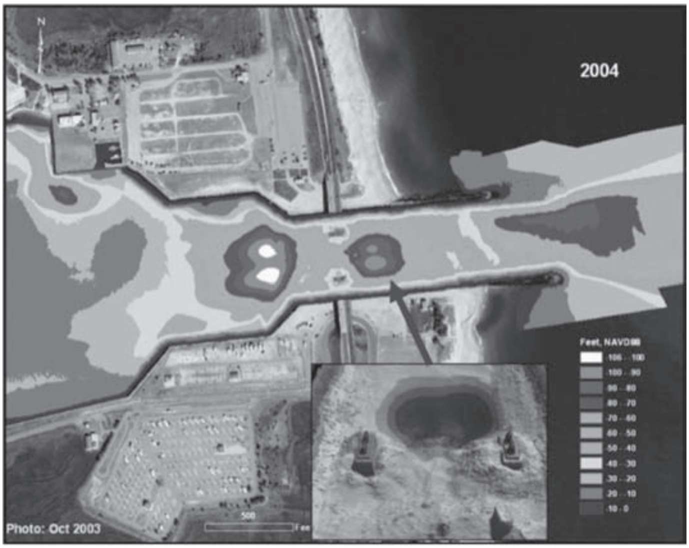

Figure 7. Inlet scour depth contours.

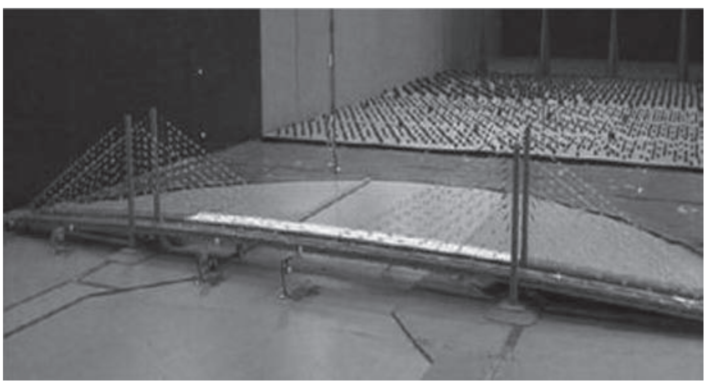

Figure 8. Wind aeroelastic model.

the bridge were chosen conservatively. The bridge was designed for wind at the STRENGTH limit state for a 100-year return period, and at the EXTREME limit state for a 2,000-year return period. In addition, the stability of the structure was verified for wind with a 10,000-year return period.

Wind analyses were carried out by the specialist sub-consultant Rowan Williams Davies & Irwin, Inc. (RWDI), and included tests on small sectional models, testing on aeroelastic models of the full bridge, and development of the design loads for the bridge. A picture of the aeroelastic model completed for the bridge is shown in Figure 8.

3.4 *Foundations and Geotechnical*

The geotechnical conditions at the site were a major influence on the design. The soils at the site are composed of four principal layers. The first is a sand layer, which offers some resistance to vertical loads, but is subject to scour. The second is a clay layer, which is not extremely effective in carrying vertical loads. The third layer is another sand layer, which is much more compact than the upper sand layer and is the primary load bearing layer. The

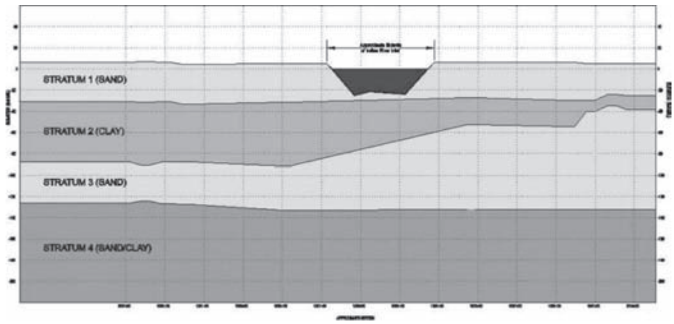

Figure 9. Soil strata.

fourth layer is composed of sandy clay that exhibits lower strength than the sand layer above it. The different soil layers at the site are illustrated in Figure 9.

The foundations for the bridge consist of 3-foot square precast driven piles, with a maximum depth of approximately 100 feet. Because of the soil conditions described above, the design of the piles required balancing several factors. Obviously, it was desirable to found the piles in the stronger sand layer in order to develop adequate vertical capacity. Additionally, it was necessary to include the effects of the predicted 35 feet of scour in the pile capacity calculations, which had a particularly strong influence on the lateral design. Running counter to these influences, though, are the undesirable aspects of deeper piles. These primarily include significant downdrag loads that are developed in the unscoured condition in the second clay layer, as well as drivability of the deep piles.

4 DECK CONFIGURATION

The bridge is close to the ground, with a top of footing elevation at 14 feet above sea level and the top of deck is approximately at 45 feet. Given that the deck is both cast-in-place and highly compressed, the effects of creep and shrinkage were an important design consideration. These two factors drove the bridge articulation at the pylons.

Because the deck is close to the foundation, restraining the bridge longitudinally would create a very rigid frame. As the deck shortens, not only would it induce large shears into the foundation, it would also create a significant net tension in the deck. This is not desirable, given the stringent allowable stress requirements.

To mitigate the effects of creep, there is only one longitudinal connection to the substructure. At the north pylon, elastomeric bearings transfer longitudinal forces from the deck to the pylon. Bearings are located on each longitudinal face of the pylon, so that they are acting only in compression. At the south pylon, the deck is free to translate relative to the pylon. This results in creep movements of approximately 6 inches at the transition pier closest to the south pylon, and approximately 2 inches at the transition pier closest to the fixed north pylon. Allowing this movement significantly reduces the axial effects in the deck and eliminates the high shear in the foundations.

However, these benefits are not without some cost. While it may intuitively feel that the fixed pylon would see high demand from creep movements, the analysis revealed that the opposite is true. Unlike a beam bridge, in this case the free pylon yields significant moments due to creep. The reason lies in the longitudinal stiffness of the stays. As the deck translates, the stays try to translate along with it. However, they are restrained by the free pylon. This results in a scenario where a single large shear at the base has been exchanged for multiple small horizontal loads near the top of the pylon. While the magnitude of these shear loads are not large, they are applied high up in the pylon, resulting in a significant creep moment at the base. The free pylon thus sees a creep moment at the base roughly twice that of the fixed pylon.

To reduce this moment in the free pylon, the deck will be jacked apart prior to closing the bridge at mid-span. Jacks will be placed between the deck and the free pylon immediately prior to casting the mid-span closure. The jacks will be engaged and the deck will be pushed back away from the main span by approximately 2 inches. The deck will be held in this position while the mid-span closure is cast and then released. This will incline the free pylon away from the main span, yielding a more symmetrical demand envelope in the pylons over the life of the structure.

5 PYLON AND EDGE GIRDER CONFIGURATION

As is the case with most cable-stayed bridges, development of the deck section in relation to the pylon geometry is typically an exercise in tradeoffs. From the perspective of pylon construction, a straight pylon without transverse cross beams and a constant section is desirable. Inclined pylon legs or large cross beams cast above the deck can add both time and cost.

On the other hand, it is also desirable to align the stay anchorages in the pylon with the anchorage plane in the edge girders. Again for construction, the most desirable configuration is to keep the anchors in a single plane. This reduces the complexity of the anchorage details and simplifies anchorage installation and alignment.

The third factor that often drives the overall deck and pylon configuration is the required roadway width. The limiting condition for this factor is at the pylons, where the required roadway must pass between the pylon legs.

For the Indian River Inlet Bridge, the design-build team determined that a vertical tower with no cross beam was the preferred solution. Additionally, the stays were to be oriented in a vertical plane. With these parameters, the challenge to the designers was to minimize the required width of the deck. The solution consisted of two key elements.

First, the stay cables were offset transversely within the pylon column. The stay cable work points are located 1'-3" away from the pylon centerline towards the centerline of the deck. This reduces the intrusion of the pylons into the deck width. However, offsetting the stay cables is clearly not without cost, as the vertical component of the stay cable reaction now creates transverse bending in the pylon. This was partially mitigated by varying the thickness of the pylon transverse walls, with the inner wall one foot thicker than the outer wall. This has the effect of shifting the pylon center of gravity towards the stay cables. While this did not completely eliminate the effect of the stay offset, it did result in an economic design. The pylon dimensions are illustrated in Figure 10 below.

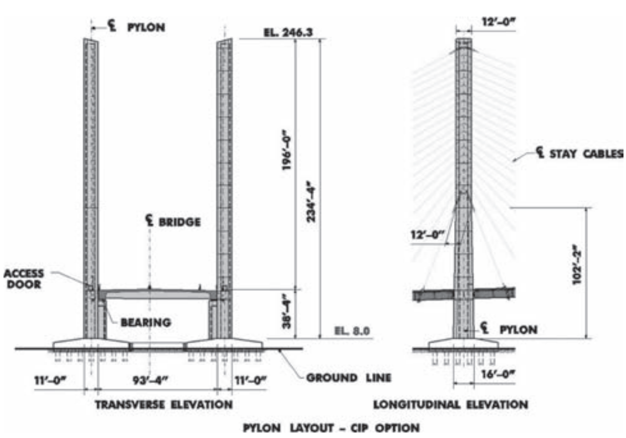

Figure 10. Pylon dimensions.

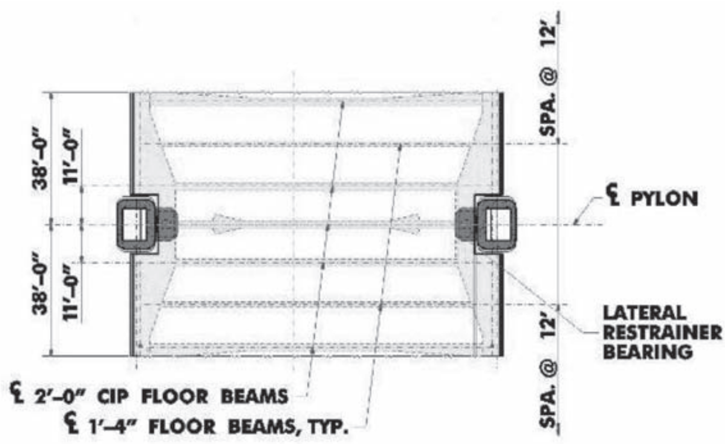

Figure 11. Deviation of edge girders at pylons.

The second part of the solution involves the configuration of the edge girder in the vicinity of the pylons. For the majority of the bridge length, the edge girder is centered on the plane of the stay cables. At the pylon, however, this would mean that the edge girder would need to pass through the pylons, which would have resulted in an undesirable fixity condition. The solution was to deviate the edge girder around the pylon, as illustrated in Figure 11. This allows the edge girder to be aligned with the pylons in the regions where the stay cables are anchored, while still allowing the deck to move longitudinally at the free pylon.

6 TRANSITION PIER AND BALLAST

As with many cable-stayed bridges, the ratio of the back span to the main span is slightly less than 1:2. This is often desirable for the global performance of the structure, as it provides some additional longitudinal stiffness to the pylons while minimizing edge girder bending near the transition pier. However, this all raises two design challenges. First, it is desirable to balance the longitudinal shear in the pylons. And secondly, it is necessary to provide a stable vertical support at the transition piers.

To satisfy these design conditions, the solution for the Indian River Inlet Bridge involved the extensive use of ballast near the transition piers. The ballast is poured between the floor beams, which are designed to carry the additional weight in this area. The pier cap itself is a heavy concrete member, adding to the total weight of the ballast. Additionally, the pier cap was designed with a shelf on the back side that accommodates the bearings for the approach span girders. In this manner, the weight of the approach span contributes to the stability of the transition pier bearings. An elevation view of the transition pier cap and ballast region is shown in Figure 12.

7 STAY CABLE ANCHORAGE

Anchoring the stay cables in the pylons is an important aspect of the design. For the Indian River Inlet Bridge, it was decided to use a steel anchor structure embedded in the hollow concrete pylon as illustrated in Figure 13. Each anchor structure consists of a U-shaped steel box, with a stay cable anchored at either end. One side of the box is composite with the side wall of the pylon via shear studs, as are the front faces where the stays are anchored. The steel anchorage box serves two primary purposes. For the typical service condition, the box

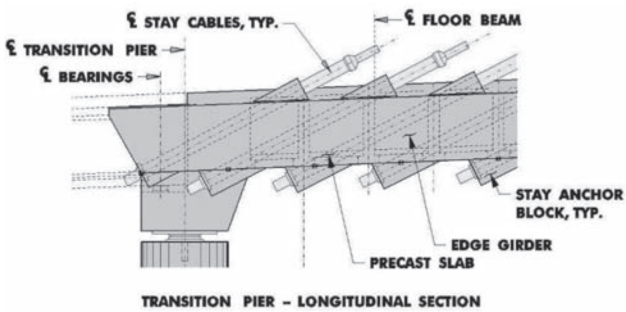

Figure 12. Transition pier cap.

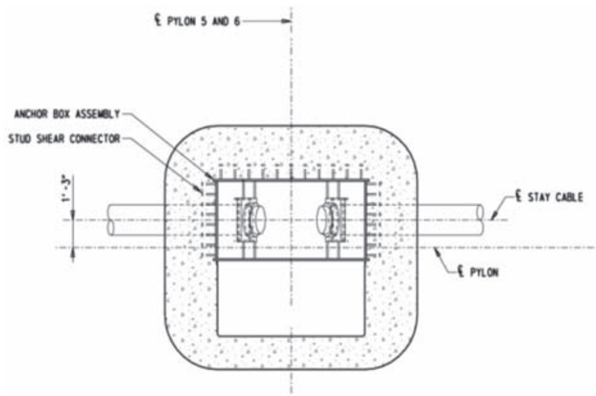

Figure 13. Stay anchor at pylon.

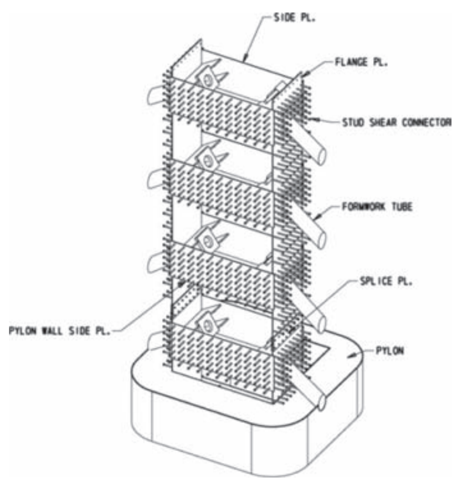

Figure 14. Anchorage spacing.

acts to transfer the horizontal component of the cable tension directly to the opposite stay while transferring the vertical component into the pylon. In this condition, only the unbalanced portion of the horizontal stay cable load is transmitted from the anchor structure to the concrete pylon.

For the stay cable loss events, both the required sudden loss and replacement cases, the entire force in the unbalanced stay cable is transferred to the pylon. The box connection in these cases is designed to transfer the unbalanced stay load from the steel box through the embedded studs and into the concrete section.

Contractor input was important in developing a cost-effective tower anchorage. One key modification was the approach to laying out the stay cable work points. Typically, a designer will lay out the stay cables based on the intersection point at the pylon centerline, spacing the intersection points at regular intervals. In this case, however, a regular spacing of the theoretical intersection points was not desirable, but there was a desire to standardize the spacing between anchor boxes. The stay cable work points were adjusted so that the boxes could be placed at regular 6 foot intervals. Additionally, the contractor requested that the boxes be assembled in groups of three, matching the 18 foot lift heights selected for the forming system of the pylon, as illustrated in Figure 14 below. This allowed the complex details of the anchorage region to be standardized from lift to lift.

8 CONSTRUCTION DRIVEN DESIGN DECISIONS

As the Indian River Inlet Bridge was designed under a design-build contract, there were many opportunities to tailor the design to the preferred construction methods. These early efforts were important in creating a constructible and cost-effective design. Two aspects of the design that were influenced by the construction methods are described in more detail below.

8.1 *Construction Sequence and falsework*

The site for this bridge presents a unique advantage seldom seen in cable-stayed construction in that more than half of the deck is accessible from the ground. This presented an opportunity to construct a significant portion of the deck on falsework. This method of construction is clearly preferable, as it is both less expensive and significantly faster than traditional form traveler erection. The erection sequence is a combination of cantilever erection over the inlet using a form traveler and cast-in-place construction on falsework for the portions over land. As illustrated below in Figure 15, the entire back span and the first 8 segments of the main span are cast on falsework. Construction in this manner has the added advantage of providing stability during erection. Because the back span is typically built several segments in advance of the main span leading edge, it acts as built-in ballast for both erection and wind loadings.

8.2 *Precast beams*

By constructing much of the deck on falsework, the opportunity to use precast elements was created. In this case, the floor beams in the falsework sections were typically designed as

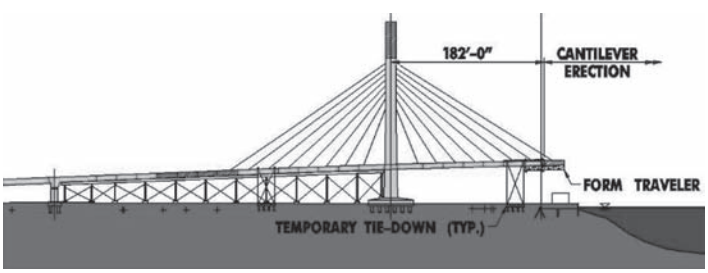

Figure 15. Construction on falsework.

precast elements. The primary advantage of using such precast beams is schedule, with the precast beams eliminating an additional concrete pour per lift wherever they are used. Cast-in-place floor beams were used in the pier table and ballast regions, due to the size and low number of these beam types.

9 CONCLUSIONS

The design-build procurement method has proven to be successful for the design and construction of the new Indian River Inlet Bridge. The new concrete cable-stayed bridge will significantly improve the safety of the crossing and long-term maintenance concerns of the existing bridge.

Final design of the bridge was completed in 2009 and construction is nearing completion. The main bridge foundations and pylons were completed in late 2009/early 2010, and all of the deck sections cast on falsework were completed in late 2010. Deck construction in the main span with the form traveler began early this year and is anticipated to be completed this fall. The bridge is scheduled to open in early 2012.

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Chapter 4

Main cable corrosion protection by dehumidification: Experience, optimization and new developments

M.L. Bloomstine
COWI A/S, Denmark

ABSTRACT: Corrosion of main cables on suspension bridges is a wide spread, well known and very serious problem. The safety of a great number of suspension bridges is compromised due to hidden corrosion that reduces the load carrying capacity of the main cables. Corrosion protection of main cables by dehumidification has over the last 13 years been proven to be in all ways superior to earlier so called traditional corrosion protection systems. Dehumidification is the only method that actually prevents corrosion, whereas other methods can at best only slow down corrosion. This paper describes the development of corrosion protection of steel bridge components by dehumidification over the last 40 years and in particular the application of this technique to the main cables of suspension bridges. Experience from existing systems, the world-wide status for main cable systems, optimization methods and new developments are also described.

1 INTRODUCTION

Numerous examples of serious suspension bridge main cable corrosion have been discovered in USA, Europe and Japan. Many of the suspension bridges in USA are quite old; several are over 100 years, so it is not especially surprising that there is serious corrosion in the main cables of many of these bridges. More surprising are reports from Europe and Japan, where relatively young bridges; 5–30 years, have serious corrosion problems, Figure 1. Even more surprising is the fact that the main cables on many of these bridges have been well maintained and regular external inspections have revealed no signs of corrosion. This clearly shows that all suspension bridge owners/operators, no matter the age of the their bridge, should instigate measures to determine the condition of the main cables and protect them from corrosion by the best means possible, which has been proven to be dehumidification. This is the only

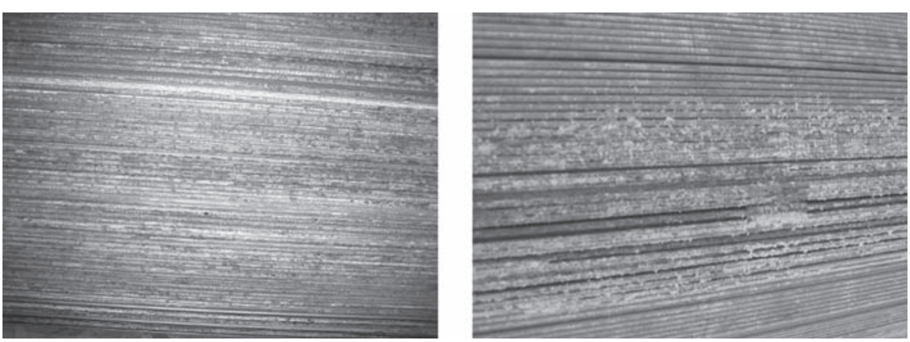

Figure 1. Corrosion on main cables aged roughly 30 and 8 years.

method that actually prevents corrosion and does so by providing a dry and noncorrosive atmosphere inside the cables.

Due to the above mentioned serious corrosion problems the use of existing dehumidification technology was further developed in Japan and Denmark during the 1990s for application on suspension bridge main cables. The goal was to completely prevent corrosion. There is currently 13 years of experience with dehumidification of suspension bridge main cables in Asia and Europe and the systems are showing excellent results.

2 GENERAL DESCRIPTION OF CORROSION PROTECTION BY DEHUMIDIFICATION

Dehumidification has been utilized as corrosion protection for more than 50 years. Dehumidification is based on the fact that steel does not corrode when the relative humidity (RH) of the local atmosphere is below 40%. This was proven by research at MIT Corrosion Laboratory lead by Professor H.H. Uhlig. Between 40% and 60% RH corrosion can occur, though at a very low rate. When the relative humidity exceeds 60% the rate of corrosion increases dramatically. The relationship between RH and the rate of corrosion is illustrated below in Figure 2.

A dehumidification system for a steel bridge structural element (see examples in section 3) is composed of relatively few elements. A dehumidification plant provides sufficiently dry air and circulates it inside the structural element, ensuring that the inner surfaces are protected from corrosion. The main components of the dehumidification plant are a electrical/control panel, a dehumidification unit and a fan unit as illustrated in Figure 3.

The dehumidification unit is generally based on active sorption, as it is efficient for virtually all air conditions, i.e. there are practically no temperature and relative humidity limits. This method works by binding the moisture in the process air to a hygroscopic material (a sorbent). A dehumidification unit based on active sorption contains a rotor which is built up of many small pipes, coated with a sorbent, most commonly lithium chloride. The process air is forced through the rotor and its moisture is absorbed under this process, resulting in dry air. The rotor turns very slowly, allowing time for the process. On the opposite side of the rotor, intake air is heated and blown through, which dries out the sorbent coating. This air becomes moisture laden and is subsequently discharged. A dehumidification unit

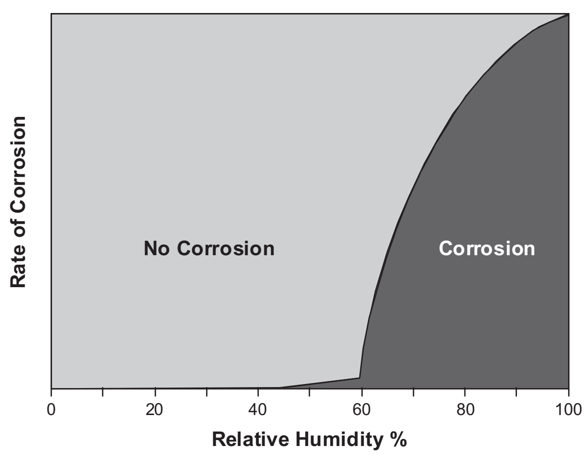

Figure 2. Relationship between RH and rate of corrosion (Prof. H.H. Uhlig, MIT Corrosion Laboratory).

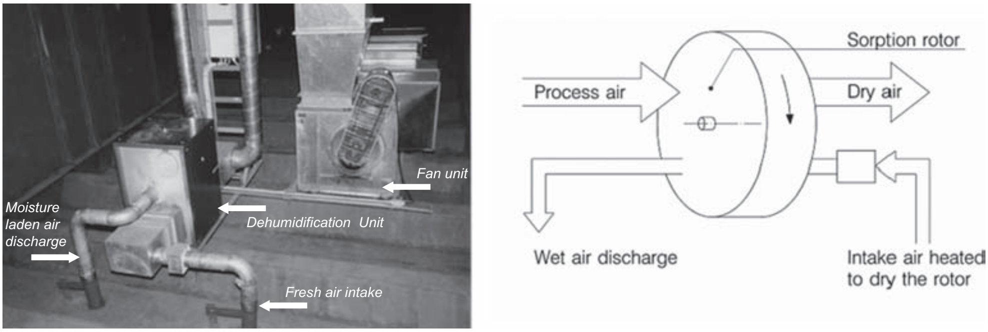

Figure 3. Typical dehumidification plant and diagram of active sorption dehumidification unit.

with active sorption is shown in the picture in Figure 3 and the principal is illustrated in the diagram.

3 EXPERIENCE WITH DEHUMIDIFICATION OF BRIDGE STRUCTURES

Corrosion protection of steel bridge structures by dehumidification was pioneered and developed by COWI in Denmark and there is currently over 40 years of experience from various steel bridge structures. This development started with the Little Belt Suspension Bridge that was constructed during 1965 to 1970. Dehumidification systems were designed and installed in the box girder and the anchor chambers (protection of cable strands) during construction. The dehumidification systems each include a dehumidification plant that produces dry air and a circulation system that ensures that dry air is circulated in all areas of the structures. The original systems perform effectively are still in service and have required a minimal amount of maintenance. The condition of the strands, splay saddle and other elements in the anchor chamber is as new. In the box girder shiny untreated steel plates were hung up at various isolated locations and these are also still as new, see Figure 4.

The development continued on the next major bridges to be built in Denmark in the 1980s, the Faroe Bridges, where further applications were developed and applied. These two adjoining bridges have steel box girders with lengths of 1.6 and 1.7 km that are protected by dehumidification. The southernmost of the bridges includes a cable-stayed bridge and the lower cable anchorages are protected by the dry atmosphere in the bridge girder, whereas the upper cable anchorages are protected by a separate dehumidification system in the anchorage boxes at the top of the pylons. Systems were also installed in the abutment rooms on both bridges to protect the expansion joints and other structures located here.

These successes were followed up by systems on the Great Belt Bridge in Denmark, the Humber Bridge in England, the Högakusten Bridge in Sweden, the Oresund Bridge between Denmark and Sweden, the Pont de Normandie Bridge in France, the Stonecutters Bridge in Hong Kong and numerous other major bridges in many countries. It has virtually become a worldwide standard to apply dehumidification to bridge box girders and anchor chambers, as it is recognized as the most effective and economical means of corrosion protection. Dehumidification can be incorporated in the design of the bridge, or it can be applied as a retrofit, such as in the case of the box girder of the Humber Bridge in England.

All this experience from dehumidification of steel bridge structures over a period of about 30 years led to the development of dehumidification systems for main cables as the optimal system for corrosion protection. Since the first application on main cables in 1998 dehumidification of main cables has been continuously developed and is now internationally accepted as the optimal method for main cable corrosion protection.

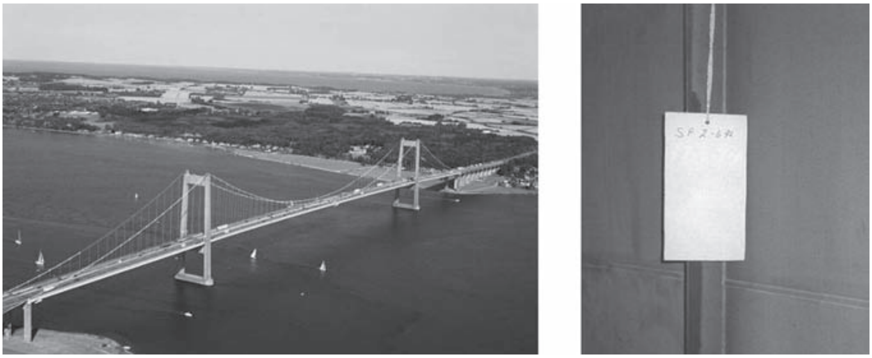

Figure 4. Little Belt Suspension Bridge and untreated steel plate in bridge girder.

4 CURRENT WORLD-WIDE STATUS FOR DEHUMIDIFICATION OF MAIN CABLES

4.1 *Systems in service*

There are currently 21 suspension bridges in a total of 8 different countries where dehumidification has been installed on the main cables as described below. As far as we know these systems are generally performing well, though some problems have been reported and suitable modifications have been carried out.

The development of dehumidification systems for main cables began in the 1990s with studies being carried out in both Japan and Denmark. The systems for main cables were based on tried and proven technology as a natural extension of earlier bridge projects.

The first bridge with dehumidification of the main cables was the Akashi-Kaikyo Bridge in Japan, the world's longest suspension bridge with a main span of 1,991 m opened in 1998. The dehumidification system was integrated in the bridge's design and installed during construction. Based on the experience from the Akashi-Kaikyo Bridge the operator Honshu-Shikoku Bridge Expressway Company Limited (HSBE) decided to install dehumidification on the main cables of all their suspension bridges. To date they have installed systems on 5 new suspension bridges and 7 existing suspension bridges in Japan.

In 2008, after 10 years of service, the main cables of the Akashi-Kaikyo Bridge were opened and wedged for inspection. The purpose of the inspection was to verify the effectiveness of the dehumidification system. According to HSBE Newsletter No. 38, July 2009, the cables were completely dry and the wires were in the same condition as at the time of construction.

The first suspension bridge outside of Japan to be retrofitted with dehumidification of main cables was the Little Belt Suspension Bridge in Denmark with a main span of 600 m, see Figure 4. The bridge opened in 1970 and the dehumidification system was installed on the main cables in 2003. Research and testing of sealing systems and on-site testing of injection and flow parameters were carried over several years before the final project was developed and tendered. This system has been performing well for 8 years and no leakage has developed, as documented by the monitoring system. The drying out process was also documented by the monitoring system and took only a few days. The dehumidification plants have been inspected once a year and no maintenance has been necessary yet.

Also during 2003 a dehumidification system was installed on the main cables of the 40 year old Aquitaine Bridge in France during a retrofit project where the entire cable system was replaced. This system has also been performing well since installation.

The Högakusten Bridge in Sweden has a main span of 1,210 m and opened in 1998, see Figure 5. There were already serious problems with water intrusion and corrosion of cable

Figure 5. The Högakusten Bridge during installation of dehumidification system.

wires after 5 years. A dehumidification system was installed on the main cables during 2005 and a massive amount of water in the cables was dried out and the relative humidity brought down below the corrosion threshold. Data from the monitoring system has shown that the amount of water removed from the cables corresponds to about 3% of the void volume in the cables and this took about 1.5 years. The system has been performing well, though there has been a problem with one of the transition shrouds outside one of the anchorage chambers, see section 6.2 for further information.

In Great Britain there are three major suspension bridges and dehumidification systems have been installed on the main cables of all three of these during 2007 to 2010. These are the Forth Road Bridge opened in 1964 with a main span of 1,006 m, the Severn River Crossing opened in 1966 with a main span of 988 m and the Humber Bridge opened in 1981 with a main span 1,410 m. The Humber Bridge was also earlier retrofitted with dehumidification systems for the box girder and the tower saddles in the 1990s.

Dehumidification of main cables has also been installed on suspension bridges in China and Korea. In China dehumidification of main cables was incorporated in the design of the Rung Yang Bridge. In Korea dehumidification of main cables was incorporated in the design of the Gwan Yang Bridge and installed on the existing Yong Jyong Bridge.

4.2 *Current projects and standards*

At the current time there are at least six projects for dehumidification of main cables under various states of progress as described below. Furthermore, authorities in two countries, Japan and Norway, currently require dehumidification of main cables as a standard. As mentioned in section 4.1, HSBE in Japan requires dehumidification of main cables on all their suspension bridges. In Norway the state authorities have required dehumidification of main cables in their bridge design standard (Bruprosjektering, Normaler, Håndbok 185) since 2009. Current projects in Norway include dehumidification, see below.

There are currently at least 6 different projects commencing in 4 different countries as described below. Together with the systems already in service there are at least a total of 27 systems in 11 different countries.

In Sweden a system is currently being installed on the Älvsborg Bridge in Gothenburg. The project is expected finished in August this year. This is an integrated system and further information is included in section 7.2.

In Norway two projects are currently progressing. The Hardanger Bridge with a main span of 1,310 m is currently under construction and is expected finished in 2012, see Figure 6. This is an integrated system and further information is included in section 7.2. The Hålogaland Bridge with a main span of 1,145 m is currently preparing for tendering, see Figure 6.

Figure 6. The Hardanger and Hålogaland Bridges, Norway.

Figure 7. The Messina Bridge, Italy—Future world record span of 3,300 m.

In Qatar 2 twin suspension bridges with unique circular towers, the Lusail Bridges, are to commence construction this year. The system for these bridges is a fully integrated system as described in section 7.2.

In Italy the Definitive Project (Progetto Definitivo) for the Messina Bridge with a world record span of 3,300 m has been completed, see Figure 7. An integrated system has been designed encompassing the 4 main cables, each with a diameter of 1.1 m, the triple box girder and the steel towers. The anchorage chambers are designed with separate dehumidification systems.

5 DETAILS OF DEHUMIDIFICATION SYSTEMS FOR MAIN CABLES

A system for corrosion protection of main cables by dehumidification consists of the following three main components:

- A dry air system capable of producing and blowing dry air through the main cables.
- A sealing system for the main cables, including cable bands, saddles and other connected components.
- A control and monitoring system.

5.1 *Dry air system*

The dry air system produces dry air and blows it through sections of the main cables. The system assures overpressure inside the sealed cable system. While the sealing system may have minor imperfections in the form of small leaks, no water or moisture will enter the cables, as the overpressure will prevent this. The dry air system is made up of the following main components:

- Dehumidification plant(s).
- Injection and exhaust points.
- A layout, such as shown in Figure 15, in Section 7.2, is developed for the dehumidification system. The layout defines the locations of the dehumidification plant(s), buffer chamber(s) (see description of buffer chamber in section 6.4), injection and exhaust points as well as the flow sections.

The main components of a dehumidification plant are a dehumidification unit, a fan, an electrical board, filters and ducting, such as illustrated in Figure 8. Injection points are established by either modifying existing bridge components, such as the saddles or by designing purpose suited injection and exhaust collars, see Figure 8.

5.2 *Sealing system*

We have carried out extensive research, development, workshop testing and on-site testing to determine the best systems for sealing the main cables, cable bands, saddles and other connected components. This has been supplemented by eight years of experience with sealing systems installed on bridges with dehumidification of main cables. We have concluded that the best system to seal the cable panels is the Cableguard™ Wrap System from the D.S. Brown Company. This is an elastomeric wrap with a thickness of 1.1 mm and a width of 200 mm. It is applied with slightly more than 50% overlap, so the total thickness is 2.2 mm. It is applied under tension with a special wrapping machine. After wrapping a section it is heat bonded with a special heat blanket, which melts the two layers together and shrinks the material slightly, giving an even tighter fit. The wrapping and bonding work is illustrated below in Figure 9. Special details have been developed to ensure sealing at the transition to the cable bands and to give a uniform appearance.

5.3 *Control and monitoring system*

The control and monitoring system allows remote control/adjustment of the system and data from the system documents that the system is performing properly and that the cables are

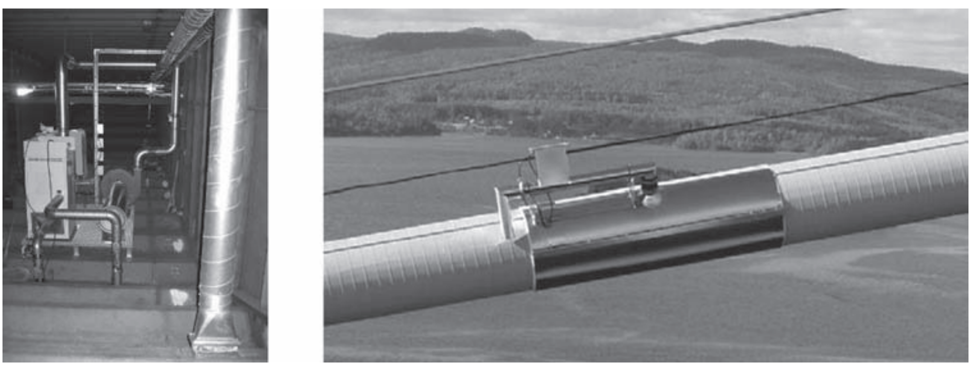

Figure 8. Dehumidification plant and exhaust sleeve.

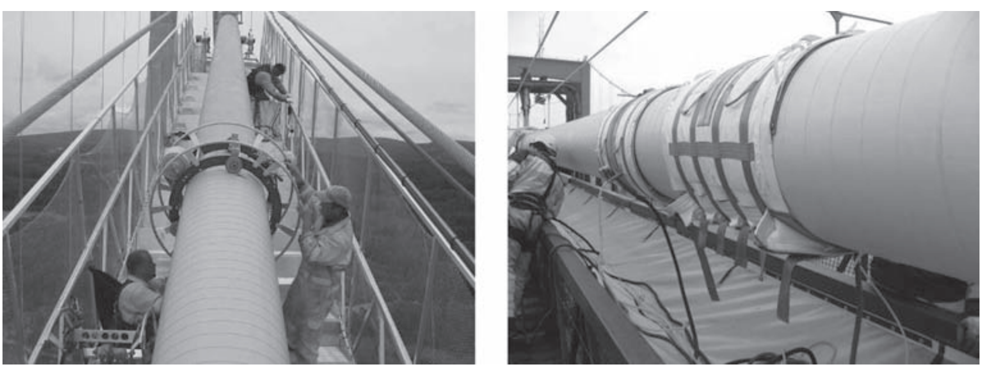

Figure 9. Wrapping with wrapping machine and bonding with heat blankets.

protected from corrosion. Instrumentation is arranged at the dehumidification plant(s), in the buffer chamber(s) and at injection and exhaust points. Key data to be monitored includes system functionality, relative humidity, temperature, flow and pressure.

Generation of a number of standard and custom graphs should be integrated in the monitoring system allowing an even more effective overview of the systems functionality. One example of a valuable standard graph is the absolute water content in the injection air and exhaust air, which is calculated on the basis of the relative humidity and the temperature. This allows generation of a graph that clearly illustrates the drying out process with the ratio between the absolute water content for corresponding injection and exhaust points. Another valuable graph is a comparison of the corresponding injection and exhaust flows, which can indicate if any leakage is developing.

6 EXPERIENCE WITH SYSTEMS FOR MAIN CABLES AND LESSONS LEARNED

Extensive experience and valuable information concerning dehumidification systems for main cables has been obtained through numerous project designs, on site testing, full scale model testing and up till now 13 years of service. HSBE in Japan has opened and inspected the main cables of the Akashi-Kaikyo Bridge after 10 years of service in 2008. As mentioned in section 4.1 the wires were dry and the condition as at the time of construction, verifying that the dehumidification system is functioning well. From other bridges, such as the Little Belt Bridge in Denmark and the Högakusten Bridge in Sweden, the monitoring systems have during respectively 8 and 6 years confirmed the effectiveness of the systems.

6.1 *Drying out process*

The drying out process has been monitored on various bridges and it can vary greatly in time, depending on the type of cable, how much water is accumulated in the cables and the local climate. Generally it can be concluded that cables made up of strands dry out much quicker than cables of parallel wire. Strand cables have much larger voids and the water is easier to remove as the pathway from each void is more open than for parallel wire. On the Little Belt Bridge the strand cables were dry a few days after the dehumidification system was turned on. Earlier inspections had however shown that the cables were relatively dry, so this has also contributed to the short drying out period.

On the other end of the scale the parallel wire cables on the Högakusten Bridge took about 1.5 years to dry out. There are two reasons for this. Firstly the cables were very wet as the original sealing system had many defects. On the basis of data from the monitoring system it was calculated that roughly 500 liters of water were removed from a 300 m long stretch of the cable, corresponding to app. 3% of the void volume. Secondly the bridge is located in the far north, as far north as Alaska or Siberia, which means long freezing winters. Data from the monitoring system showed that the drying process progressed mainly in the summer, but during the winter much less drying out occurred due to the frozen condition of the water. This is illustrated by the graph in Figure 10 where the ratio between the absolute moisture content in the exhaust air and injection air is shown. A high ratio indicates that a relatively large amount of water is being removed from the cables. When the ratio falls to one over the second summer no more water is being transported out of the cables, as they have become dry. Reports from Japan have indicated that a drying out period of roughly 3 months is more normal when the bridge is not located in such a cold climate and the cables are not so extremely wet.

6.2 *Sealing of cables*

Data from the monitoring systems have also indicated that the sealing system described in section 5.2 is durable. The ratio between the exhaust flow and the injection flow is constantly

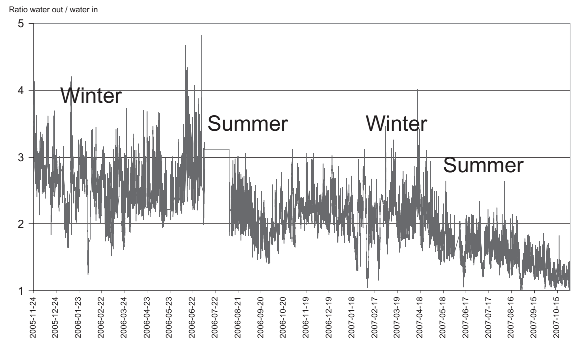

Figure 10. The drying out process on Högakusten Bridge, ratio of absolute water content.

monitored. This ratio is noted for each stretch of the main cable in connection with handing over the works. The current ratio is regularly compared with the original ratio and this has in all cases been constant, indicating that no leakage has developed.

Experience has shown that the elastomeric wrapping system described in section 5.2 seals the cables excellently. The wrapping work requires some initial training of the workers, but once they are trained and approved to do this work they have no problems meeting the quality requirements. The original heating blanket for bonding the two layers of elastomeric wrap has been improved in recent years. The original heating blanket was stiff and could not adapt to the shape of the main cable, which is usually not quite circular. This gave a less than 100% bond of the two layers, but still provided sufficient air tightness. The new heat blankets for the system are both flexible, so they adjust to the shape of the main cable, and inflatable, so they exert a uniform pressure during heating. The new heat blankets ensure a virtually 100% bond between the two layers.

Whereas the elastomeric wrapping system between the cable bands is quite standard and straightforward to apply, other details require more attention. These details include the transition between the wrapping system and the cable bands, the cable bands, saddle lids, shrouds and other encasings of the main cables.

The details of the face of the cable band towards the wrapping system vary from bridge to bridge, so there is not one solution that suits all bridges. The cable bands on some bridges have a groove in this position and it is possible to fit the elastomeric wrap into the groove and then caulk the groove to provide a good seal. On other bridges the bands have no groove and the wrap should be laid as flush with the band as possible. In this case a larger amount of caulk is required to seal this detail and the void under the neoprene wedge should be completely filled. Furthermore, the two halves of the cable band have gaps in this face that have to be sealed. Usually there is a sufficiently wide gap, 1–2 cm wide, which provides enough room to seal with caulk. On some bridges the joint in the band is stair shaped and there is virtually no gap, which makes sealing difficult.

The cable bands have several details that require sealing. The large highly tensioned bolts that prevent the cable bands from slipping need to be sealed. Full scale tests have shown that leakage occurs at several positions on the bolts if they are not sealed, i.e. between the washer and the band, between the nut and the washer and along the threads of the bolt. All these positions must be sealed and several solutions can be applied. The whole area can be painted with a thick, durable paint system with good adhesion. Caulk can also be applied to prevent leakage. Both of these solutions require regular maintenance and must be removed and reapplied if the cable band bolts need re-tensioning. It is also possible to design and mount air tight caps that protect the bolts and minimize maintenance requirements, a prototype for the Hardanger Bridge is shown in Figure 11.

Experience from a number of bridges has led us to conclude that details that design engineers have believed were watertight are not at all watertight and are often one of the major

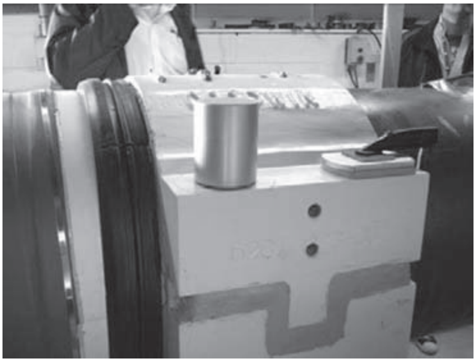

Figure 11. Prototype of sealing cap on cable band bolts.

Figure 12. Shroud that is currently being replaced by a new airtight shroud.

sources of water ingression in main cables. This conclusion is also backed up by full scale testing that proves what type of detailing is necessary to provide air/water tightness. Our general conclusion is that all these types of details should be evaluated and replaced by proven watertight details in connection with a main cable dehumidification retrofit. These details include saddle lids, shrouds and other encasings of the main cables. The sealing system for theses structural details should be twofold. The inner and main sealing should be of a flexible foam neoprene that is compressed and can effectively fill out and seal gaps of varying thickness, such as between the rough surface of a saddle and the saddle lid. The outer and secondary sealing should be of a durable, highly adhesive and UV-resistant caulk, which protects the inner seal from the elements as well as providing a double seal. All structural details should be designed to accommodate this double seal.

An illustrative example of the need for well designed sealing details is the transition shrouds adjacent to the anchorage chamber on the Högakusten Bridge see Figure 12. These shrouds are placed adjacent to the exhaust points, where the overpressure from the dehumidification system is minimal and therefore more susceptible to water ingression. In the tendered project it was assumed that it was possible to sufficiently seal the existing shrouds, but experience has shown that this was not practical. One of the shrouds is currently being replaced after which the other three shrouds will follow later.

6.3 *Length of flow sections*

The maximum viable length of the flow sections is the key factor when starting design of a dehumidification system for main cables. The length is limited by the level of allowable pressure and the flow resistance of the actual cable. We generally recommend a maximum overpressure of roughly 2,000 Pa with regards to the durability of the sealing system and to limit leakage and power consumption. Main cables of parallel wire generally have a flow resistance of roughly 10 Pa/m. The resistance is affected by the condition of the wires and the level of the flow of the dry air. Wires that are in poor condition, i.e. high level of corrosion, will generally increase the resistance. A higher level of flow will also increase the resistance in a twofold manner, directly and indirectly. There is a direct theoretical relation between the flow and the level of resistance. In order to increase the flow it is also necessary to increase the pressure, which results in increased leakage and therefore additional resistance. The maximum recommended length of flow sections for parallel wire cables is generally about 200 m, depending on the actual resistance. Main cables made up of strands have a much lower flow resistance, roughly 1 Pa/m. Therefore it is possible to have much longer flow sections with strand cables.

In all cases the flow resistance should be measured during testing prior to design. If this is not possible or practical it can be done in connection with the works and the results incorporated in the final detailing of the system.

6.4 *Buffer chamber solution*

In 1995 we developed the buffer chamber solution in connection with dehumidification of the tower saddles on the Humber Bridge in England. There were problems with water ingression in the saddles and it was decided to dehumidify these as well as the main cables in the vicinity of the saddles. This would require constant injection of dry air in the saddles, as the dry air would flow into the main cables and eventually disappear through leaks in the existing sealing system. It would be relatively expensive in electrical consumption if the dehumidification unit was to run constantly, so the buffer chamber solution was developed. The air coming directly from the dehumidification unit has a very low relative humidity, just over 0%, which is much drier than necessary. Therefore a tower leg was utilized as a buffer chamber, where the air from the dehumidification unit is mixed up with ambient air to about 40% RH before injection in the saddles. In this manner the electrical consumption was significantly reduced, as well as wear on the dehumidification unit. This was also the first application of dehumidification on main cables, although the extent of the protection is not documented.

Following this successful application the buffer chamber solution has been integrated in all our main cable dehumidification projects. An existing structure is always utilized for the buffer chamber, as described in section 7. The volume of the available existing structures varies, so it has not in all cases been possible to have a chamber of optimal size. For example there are buffer chambers in the cross beams of the towers on the Högakusten Bridge, which provide a energy savings of roughly 50%. On the same bridge there is also a buffer chamber with a much larger volume arranged in part of the box girder, which gives an energy saving of roughly 75%. Optimally the buffer chamber should be large enough to give maximum savings in electrical consumption. The buffer chamber solution has a further advantage, as the dehumidification plant is protected from the elements when it is located inside the chamber, which minimizes maintenance requirements.

7 OPTIMIZING SYSTEMS INCLUDING INTEGRATED SYSTEMS

Optimising a system for main cables starts in the earliest design phase. At this time the layout of the main cables should be reviewed and a layout for the dehumidification system should be developed that suits well to the layout of the main cables. In the case of a new bridge, such as the above mentioned Messina, Hardanger, Lusail, and Hålogaland Bridges the design of the dehumidification system should be integrated in the design of the individual elements in close cooperation with the respective structural engineers. The design of the dehumidification system has interface to and influence on the design of most of the bridge elements, including the main cables, cable bands, saddles, cable shrouds, towers, bridge girder and the anchorage chambers. In the case of a new bridge it is also recommended to include full scale testing of critical sealing details, such as was done on the Hardanger Bridge, see Figure 13. The wrapping system including details at cables bands, the tower saddles including adjacent transition shrouds, the cable bands and the transition shrouds at the anchor chamber were all tested and adjusted to achieve maximum sealing.

In the case of an existing bridge the bridge documents including drawings, specifications and reports from main cable inspections should be reviewed, summarised and incorporated in a preliminary design. On this basis a specific inspection should be planned and carried out and the results should also be incorporated in the design. Furthermore, the bridge operator's knowledge of the bridge should be fully utilised. The operator should be involved in the process by providing additional specific knowledge, providing local supplemental requirements and reviewing the preliminary design.

It is also advisable to carry out on site testing of the main cables on an existing bridge, as the features of every bridge are unique and resistance to the air flow varies according to cable type, cable diameter and cable condition. If there is any doubt as to whether voids in the cables have been filled during earlier works, the ability to inject air and cause it to flow the length of the cables should also be tested. The tests can also provide valuable information about the current sealing of the cables and how much supplemental sealing will be required. Such tests have been carried out on a number of existing bridges. Pictures illustrating trial injection on the Älvsborg Bridge are show in Figure 14.

The layout for the main cable dehumidification system comprises the placement of the injection and exhaust points, flow directions and lengths, the placement of buffer chamber(s)

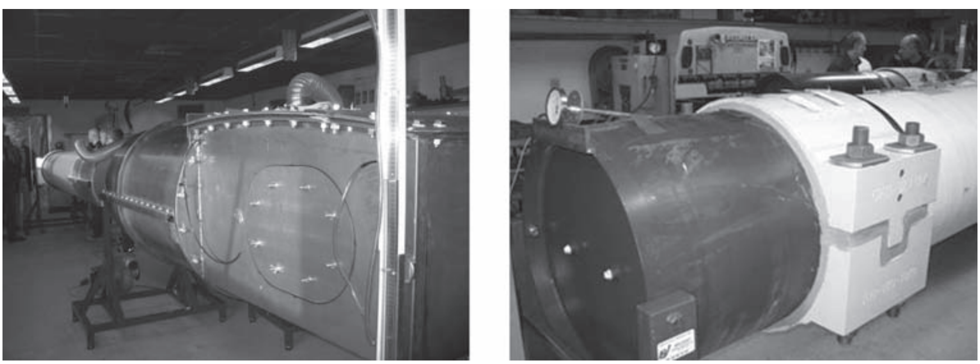

Figure 13. Full scale testing of sealing, left saddle and shroud, right cable band and transition at band.

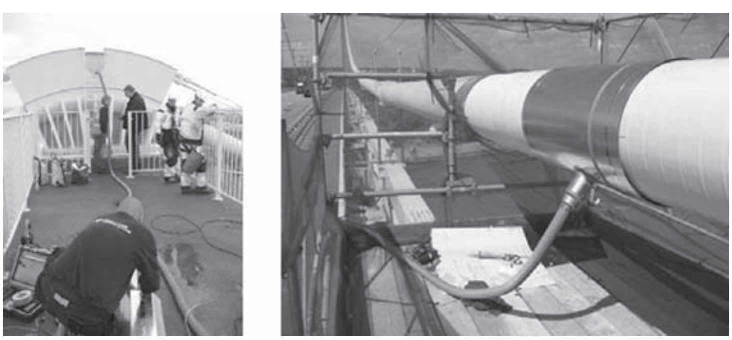

Figure 14. Injection and flow testing at tower saddle and mid span on the Älvsborg Bridge, Sweden.

and dehumidification plant(s) and ducting connecting the buffer chamber(s) with the injection points.

The lengths of main cables in the main and side spans (and back spans if these are found) and the type of main cable (parallel wire or strand and the diameter) are dictating for the placement of the injection and exhaust points. As described in section 6 it is possible to work with much longer flow stretches for main cables of strands than main cables of parallel wire. The flow stretches should be as long as possible to minimise the amount of equipment, but at the same time respect the maximum length.

The buffer chamber(s) including the dehumidification plant(s) should be placed inside an existing structure if possible, as this will save the cost of constructing a chamber. If the bridge girder is a box girder it is very convenient to isolate an area of this structure and utilise it as a buffer chamber. A part of an anchor chamber can also be utilised as a buffer chamber, though this will usually require constructing at least two walls to isolate a portion of the chamber. As mentioned in section 6.4, the volume of the buffer chamber must be sufficiently large in order to obtain the full effect and maximise energy savings.

Depending on the design of the bridge it might be relevant to apply an integrated system, i.e. a system that dehumidifies and protects two or more bridge elements. An integrated system is the ultimate of optimisation, as one set of equipment protects two or more structural elements, resulting in minimal construction, operation and maintenance costs, i.e. lowest life cycle cost. A dehumidification system for main cables can be integrated with protection of one or more of the following elements: steel box girder, closed steel towers and anchor chambers. Systems have already been designed that cover all these structural elements. Examples of integrated systems are further described below in section 7.2.

7.1 *Optimization criteria*

Depending on the layout of the main cables and if an integrated system is relevant, it may be possible to develop a number of alternative layouts for the dehumidification system. In order to evaluate the optimal system we have developed the following criteria to evaluate and compare the individual layouts. These criteria can also be used to further optimise the chosen layout.

- Feasibility—The layout must be within the limits described in this paper.
- Construction cost—A layout that has less equipment and requires less construction work will have a lower price.
- Maintenance friendliness—Equipment should be easy to access and easy to maintain.
- Maintenance costs—Maintenance costs are minimized by maintenance friendliness, a minimal amount of equipment to maintain and placement of equipment in the protected environment of a buffer chamber.
- Operation costs—Operation costs are minimized by using as few dehumidification plants as possible, plants with as little equipment as possible and a suitably large size of buffer chambers. These will all reduce power consumption.
- Necessary overpressure to force dry air through all the flow stretches—The overpressure should be below app. 2,000 Pa (0.3 psi) so as not to overload the sealing system and to minimize leakage.
- Monitoring—It should be possible to fully monitor the system's functionality and the durability of the sealing.
- Aesthetics—The bridge's appearance should not be not disturbed. This is achieved by utilizing a minimal amount of visible equipment and slender elements that are hardly visible.
- Individual bridge requirements—There may be other requirements on the actual bridge that have influence on the dehumidification system. These should be discussed and determined in cooperation with the bridge owner/operator.

7.2 *Examples of integrated dehumidification systems*

The following three examples of the Hardanger Bridge in Norway, the Älvsborg Bridge in Sweden and the Lusail Bridges in Qatar illustrate some of the possibilities for integrated dehumidification systems, especially the Lusail Bridges, as they have a fully integrated system.

A retrofit of the 44 year old Älvsborg Bridge in Gothenburg, Sweden is currently under construction. The main cables are made up of helical strands and the strands are only coated with paint, i.e. no galvanisation. An in-depth inspection of the main cables in 2005 showed serious corrosion on the bottom of the lower strands, though with negligible reduction of the load carrying capacity. The corrosion protection of the main cables needed rehabilitation and a design study was carried out to determine the optimal method of rehabilitation. Dehumidification was chosen, as the study concluded that it was superior in all aspects.

The integrated system encompasses the main cables and the strands in the anchorage chambers. Part of the southern anchor house is enclosed as a buffer chamber and a dehumidification plant that serves the entire system is located here. Ducting connects the buffer chamber with injection points at the middle of the main span. Dry air flows through the main cables about 400 m in both directions and finally flows through the anchorage chambers. At the southern end the dry air returns to the buffer chamber and is re-circulated, giving a highly effective system.

The Hardanger Bridge in Norway is currently under construction (completion in 2012) and has a main span of 1,310 m, see Figure 6. The stiffening bridge girder is a steel box girder and the main cables are made up of parallel wires. The two dehumidification plants in the box girder protect the inner surface of the box girder and utilise the volume as a buffer chamber for provision of dry air to the main cables. Ducts run up through the towers to injection points at the saddles and along two of the shorter suspender cables to injection points on the main cables. In this manner one integrated dehumidification system protects the box girder and the main cables. A separate system provides for each anchorage chamber, as it was not feasible to integrate them in the system.

Construction of the twin Lusail suspension bridges in Qatar is commencing with a fully integrated dehumidification system, see Figure 16. The inner surfaces of all steel elements are protected from corrosion by one integrated system with just one dehumidification plant.

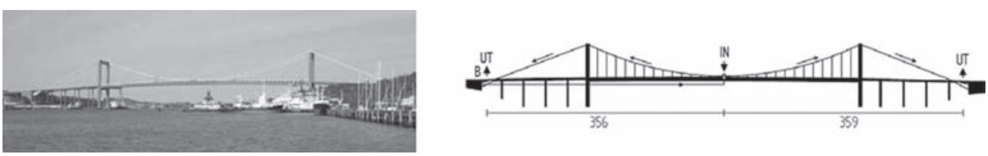

Figure 15. Älvsborg Bridge in Sweden and layout of dehudmification system.

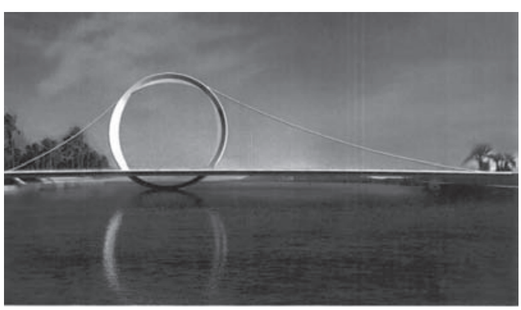

Figure 16. The Lusail Bridges in Qatar.

The bridges each have two steel box girders, a circular steel tower and main cables made of strands. A dehumidification plant is located in one box girder and it produces dry air and blows it through one box girder, then through a duct at the end of the bridge connecting the two box girders, through the other box girder and finally back to the plant through a duct at the other end of the bridge. The entire collective volume of the box girders serves as a buffer chamber. Part of the dry air in the box girders is injected into the tower from one side and circulates through the tower to the opposite box girder. The air in the tower is slightly over pressured, which causes a controlled amount of air to flow through the main cables, where it finally flows out through the anchor chambers. In this manner the insides of the box girders and the towers, the main cables and the cable strands in the anchorages are all protected from corrosion by just one integrated system. This is a fully optimised and extremely economical system.

8 CONCLUSIONS

International experience from USA, Japan and Europe has proven that corrosion of suspension bridge main cables is a very serious problem that can lead to reduction of load carrying capacity or even closure of the bridge. The degree of corrosion is generally worst on old bridges, but even some relatively young bridges between 5 and 30 years of age have been shown to have serious corrosion problems. As there are so many corrosion problems with main cables it is an obvious conclusion that the earlier applied traditional protection systems do not provide sufficient protection. All suspension bridge owners/operators should instigate measures to inspect and evaluate their main cables and upgrade the corrosion protection to the only system that actually prevents corrosion—dehumidification, such as described in this paper. A dehumidification system controls the atmosphere inside the cables and keeps the relative humidity below the critical level of 40%.

There are currently dehumidification systems for main cables in service on 21 suspension bridges in a total of 8 different countries with up to 13 years of service. Furthermore, there are currently at least 6 different projects commencing in 4 different countries, which gives a total of at least 27 systems in 11 different countries. The technology is well proven and developed and should be applied to all suspension bridge main cables. When designing dehumidification systems for main cables the experience and guidelines laid out in this paper should be utilized.

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2 Bridge analysis & design

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Chapter 5

Bridge design for maintenance

B.R. Colford
Chief Engineer & Bridgemaster, Forth Road Bridge, Forth Estuary Transport Authority, South Queensferry, Scotland

ABSTRACT: For a number of years now, engineers working in design offices on new bridge projects have been made aware of the need to design for future maintenance. Indeed in the UK and other European countries there are now statutory obligations in force that require engineers to ensure that safe access and methods of working to carry out maintenance are determined for all projects from the design stage. Minimising the use of joints and bearings in bridges and facilitating their replacement when specified; the use of weathering steels; specifying longer life paints; increasing the quality of and cover to concrete and the use of epoxy and stainless steel reinforcing steel in concrete are all methods used to minimise future maintenance. However, there is still work to be done in improving the design process. There does still seem to be a gap in knowledge between engineers working in the design office and those working in the maintenance field. There is now even stronger pressure in this financial climate for design engineers to minimise construction costs, potentially at the expense of future maintenance, especially in design and construct projects. The only effective way that maintenance can truly be considered at the design stage is if the knowledge and experience of those engineers working in the field can be harnessed at the design stage to improve the future maintainability of bridges. This paper will examine this issue with illustrations from two case studies.

1 INTRODUCTION

Bridges do not last forever. This is particularly true of modern bridges which are built from materials such as steel and concrete. Yet the expectation of society when these structures are first constructed is that they will have an infinite life. This expectation is often set against the reality that funding of the maintenance of bridges ranks very low against other funding requirements such as health, education and housing.

In the UK, it was recognized that the design philosophy of minimizing initial capital cost was having a significant effect on durability. Design standards, were changed around 2001 to promote measures to improve durability (Design Manual for Roads & Bridges 2010). These changes included the use of continuous decks over intermediate supports and the promotion of integral bridges with abutments connected directly to the deck. Eliminating the use of corrodible reinforcement in concrete is now encouraged and where this is not possible, cover to reinforcement is increased and quality of concrete is improved. The use of mass concrete in abutments and wing walls is also encouraged where possible. In addition, adequate provision is now made for access for maintenance; cleaning and painting; bearing replacement and inspection of closed cell and box members.

The standards and guidelines are now quite specific regarding certain requirements such as the provision of continuous decks and integral bridges. However, when dealing with access, there is only a need to ensure there is an adequate provision to allow maintenance and inspection.

There is a question over how robust some of these standards and guidelines are, especially with regard to access, when engineers are faced with the conflicting requirement of minimizing initial capital costs but trying to ensure bridges are durable and maintainable in an economic climate where cuts being vigorously chased through every part of the procurement process.

The prioritizing of durability, ease of inspection and maintenance into the design process is not helped by the design and construct form of contract which is now widely used.

In this form of contract, the funding authority will normally employ a consulting engineer to produce an outline design and specification for the works. This outline design will include standards on durability and maintenance. However, whether or not these are specific enough to ensure a significant change in the design philosophy towards durability and ease of maintenance rather than initial cost remains an issue.

Contractors will be invited to bid for a tender, usually following a preliminary process, when an evaluation of competency and experience is carried out. Bids are then submitted from those tenderers who have passed the initial process. Award is usually to the lowest bidder but some further quality element may be included in the bid process. However, it is commonly below 10% and is likely to comprise mainly of meeting environmental, sustainability and carbon targets rather than addressing future maintenance and inspection requirements.

Given that the contractors bidding for the work have the sole purpose of securing the contract, it is not difficult to conclude that their whole focus will be to save money in the initial cost of the project rather than address fully any long term maintenance issues. The engineers designing the bridge or bridges in this process, will be under financial pressure during a short tender period to produce the cheapest and most buildable design possible.

Unless some tangible financial incentive is included at the tender stage to reward durability and maintainability, then this form of contract results in the engineer responsible for the design of the bridge having no real economic or contractual interest in the long term durability and maintenance requirements.

It is not only the design and construct contract that weighs against long term durability and maintenance being uppermost in the mind of the engineers carrying out the design. It is likely that their expertise is rightly in the field of designing and building bridges. They are unlikely to have the experience of the maintenance of structures over a long period.

Ideally the expertise of those engineers who have spent a considerable time dealing with inspecting and maintaining bridges should be brought into the design process. However, it is rarely done and should be seen as a wasted opportunity.

Having been involved in the design of bridges and in their maintenance over a long period of time, I consider that if we are to ever effectively design durable lower maintenance structures then we have to harness the experiences of the field engineer during the design.

There will be a short term cost in this but a considerable long term gain. The engineer involved in maintenance will ask awkward and difficult questions such as:

– How do we get to the top of those piers to inspect or carry out minor maintenance.
– How can areas be contained effectively in order to blast clean and paint.
– Show me how an element can be replaced safely without disrupting traffic.

2 GENERAL

Modern materials such as steel and concrete are formed using naturally occurring materials which are fused together using large amounts of energy to form the new materials.

Unfortunately, as soon as these materials are formed they are on a single course to return to the constituent elements from which they were made. The high tensile wire that forms the cables of cable supported bridges is galvanized and protected by sheathing or painting. However, it will one day break back down to the basic elements of iron ore and carbon from

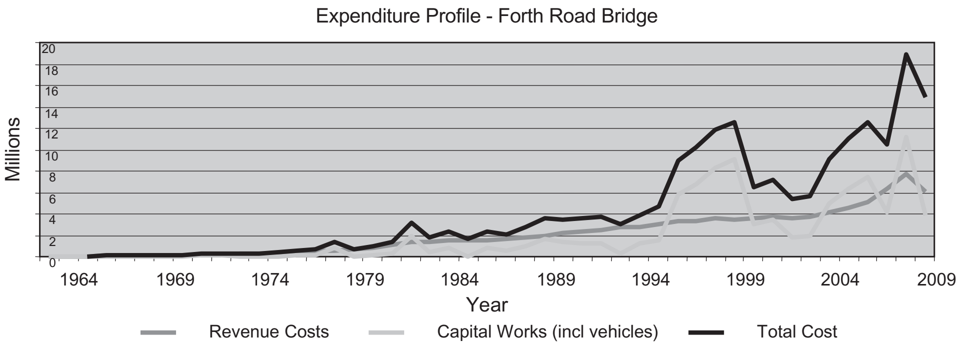

Figure 1. Maintenance on Forth Road Bridge 1964–2009.

which it was formed, in the same way that concrete will return to calcium and silicon. Our task as engineers is to ensure that for the useful service life of the bridge that material is maintained and protected in order for it to perform its function over that service life.

The maintenance of a new bridge, or piece of highway infrastructure, starts the moment it goes into service. However, there is no doubt that there exists a feeling that during the first ten years of the life of a new bridge very little will be required in the way of maintenance. Therefore, a sort of maintenance holiday results, where no funds are allocated. Unfortunately, this is not the case and if maintenance and inspections are neglected in the early years this can result in long term problems for the structure.

Figure 1 shows the funding on maintenance on Forth Road Bridge from the opening year of 1964 onwards. It clearly follows a path which is not uncommon. During the first ten years very little money was spent on inspection or maintenance of the bridge. However, the spend has increased over time to the extent that total money spent in the 46 year history of the bridge is £195 million. It is no coincidence that for most of that period (up until 2008) the maintenance and operation of the bridge was self funded by tolling.

Currently, there is a 15 year capital program of works planned for Forth Road Bridge that is estimated to cost almost £133 million pounds.

There is still considerable work to do by engineers to continue to make the case that the engineering maintenance of bridges, especially cable supported bridges where large amount of public funds have been spent on acquiring a large capital asset, requires funding from day one. Maintenance free as a term is not possible and is meaningless.

My own experience has also shown that large bridges are best maintained and operated if there is a dedicated on site team to ensure the long term life of the asset. Continual inspection and on going preventative maintenance of large bridges is best carried out by a dedicated on site team.

However, such teams are expensive and in times of stringent economic cuts, outsourcing of services by national and local governments continues to be the trend. This can effectively mean that staff trained to look after the asset may change every 3 to 5 years.

3 THE RELATIONSHIP BETWEEN DESIGN, CONSTRUCTION AND MAINTENANCE

Good design must take account not of only safe and cost effective construction but also safe and cost effective future maintenance and inspection.

Access is one if not the most crucial aspect. It involves the safety of those carrying out the inspection or maintenance. It also involves minimising the risk and the disruption to users of the bridge.

Regulations and laws now require engineers to design structures so that future maintenance can be carried out safely but are not specifically concerned about the economics of future maintenance.

Design and construct projects assist in ensuring better buildability at more certain cost but this must not be at the expense of long term maintenance or durability.

4 CASE STUDIES

Two examples of elements of the Forth Road Bridge where future durability, inspection and maintenance were not considered during design will be examined.

4.1 *Case Study 1—Half Joints in the longitudinal deck stringer (floor) beams*

Forth Road Bridge is a long span suspension bridge with a main span of 1006 metres (3300 ft) and links Fife with the Lothians across the Firth of Forth some 10 km west of Edinburgh. The bridge itself consists of three distinct sections, two approach viaducts and a suspension bridge which forms the main section of the structure. The bridge carries two carriageways 7.3 m wide and two footway/cycle paths 4.6 m wide.

The deck structure of the Forth Road Bridge, which carries the carriageways and footways/cycle paths, is suspended from the main cables by means of wire rope hangers. This suspended structure is formed from two modified warren trusses, referred to as the stiffening trusses, which are connected together by lateral bracing and cross girders. (Figures 2 and 3).

The footway and roadway decks on the bridge are separated by three longitudinal air gaps (Figures 2 and 3). The presence of these longitudinal air gaps made it difficult to connect the deck to act with the main truss in order to take global bending effects. Therefore, during the design stage it was decided that the deck should be non–participating.

The road deck panels are supported by longitudinal steel stringer beams and each carriageway deck panel is supported on four of these stringer beams which are plate girders (Figure 2).

The side span road deck is formed of a reinforced concrete deck slab composite with the steel longitudinal stringer beams. The main span road deck is formed of an orthotropic steel deck with transverse beam sections supported by the longitudinal steel stringer beams. The main span orthotropic deck and the steel stringer beams in both the main and side spans are mild steel. It appears that high tensile steel was not used as it was considered that the higher deflections resulting from the higher stresses might affect the wearing surface. The deck is surfaced using 38 mm (1.5") thick mastic asphalt.

Figure 2. Deck structure showing longitudinal stringer beams during construction.

Figure 3. Deck truss showing longitudinal air gaps.

The original design proposal was to have simply supported deck panels sitting on the cross girders at 9.063 metres (approx. 30 ft) centres with joints to permit expansion, contraction and rotation (Figure 4). However, in order to half the number of joints in the carriageway, a design modification was proposed to make the panels 18.126 metres long (approx. 60 ft) and continuous over two spans. This meant connecting cross girders locally, which were designed to accommodate the differential strains applied by the stringer beams.

Subsequently, a further design modification was carried out (what we might call value engineering today) to obtain economy in the design. This involved offsetting the joints by 1.521 metres (5 ft) from the cross girders and using the 1.521 m (5 ft) cantilever to support the 7.542 metre (25 ft) overhang of the next panel (see Figure 5).

This design was adopted throughout the length of the bridge in the main and side spans except at the main towers and mid span where a modification was required.

Because the joints were offset from the cross girders they had to take the form of half joints (see Figure 5). These half-joints are formed from cast steel sections (referred to as bearing blocks) which have been welded to the ends of the stringer beams. The castings were bolted together at the joints to prevent differential vertical displacement of the panels. The bolt is a threaded stud which is screwed into the upper casting. The lower casting is provided with a slotted hole to permit expansion and contraction of the panels across the joint.

Although this cantilevering half-joint aided erection, the main benefit was derived from reducing the bending moment value by between 25 and 30% (see Figures 6 and 7). This meant that the stringer beam size could be reduced and as there are over 14 km of beams on the deck, the cost saving would have been fairly significant in terms of both material and handling costs.

The evolution of the design process did half the number of joints from the first proposal by making the stringer beams continuous over two spans and thus reduce future maintenance problems. However, the final modification to cantilever the joints away from the cross girders appears to have been driven by cost alone and the result of that decision has left a significant maintenance legacy and a noticeably poor ride quality on the road surface of the bridge.

As there are 768 number of these half joints along the length and breadth of the suspension span decks and the cost of dealing with any problems with the design or detail is magnified by this large number of joints.

The primary defect in the existing half-joints is the failure of the stud which ties the bearing blocks together (see Figure 8). This failure has allowed differential movement to occur between the deck panels at the joints. As a result when vehicular traffic passes over a panel a horizontal gap opens and closes at the half-joint. When the gap closes the

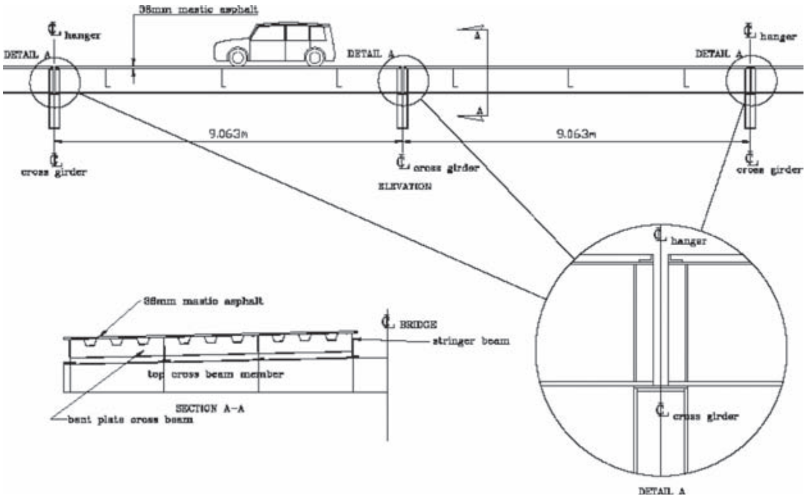

Figure 4. Stringer beams simply supported.

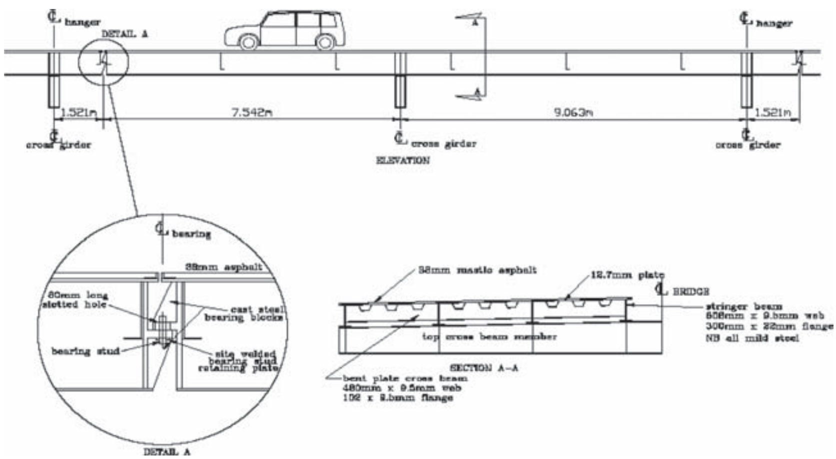

Figure 5. Stringer beams continuous over two spans.

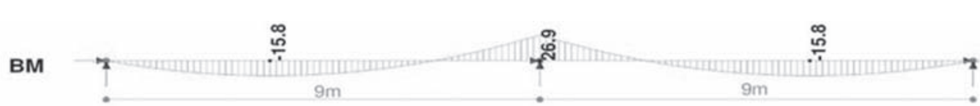

Figure 6. Joints directly over supports (Bending Moment Diagram UDL—2.69 kN/m per meter width).

Figure 7. As constructed—joints offset from supports (Bending Moment Diagram for 2.69 kN/m per meter width).

Figure 8. Existing half joint bearing block detail.

half-joints have been found to impact on one another. Wear is occurring at the half joint with steps in the level of the running surface arising across the joints. This defect has progressively worsened over the years as the weight and numbers of heavy goods vehicles has increased.

It is worth commenting that the when Forth Road Bridge was designed it was envisaged that a maximum number of vehicles that the bridge would accommodate was 11 million per annum and the maximum weight of vehicle on UK roads was 22 tons. Now the bridge carries 24 million vehicles per year and, although lorries only form 6% of the total traffic, their numbers have increased significantly. In addition, maximum lorry weight has now risen to 40/44 tonnes.

The secondary defect arising at the joints is due to water ingress through the open joints which is causing corrosion of the bearing blocks. Due to access constraints it is not possible to reinstate the protective coatings to the affected surfaces of the bearing blocks. The ingress of water has arisen due to the polycast rubber joint sealant, originally installed at the level of the running surface, no longer being present in the joints. Attempts to reseal the joint have not proved successful.

In addition to the primary and secondary defect occurring at the joint itself, the horizontal gap in the joint causes the temporary loss of the propping action of the cantilever stub and when a 40 / 44 tonne vehicle is driven onto the 7.542 meter, effectively unpropped cantilevered deck panel, it deflects impacting onto the cantilever stub. This dynamic loading effect in turn increases the stress levels on other bridge components. These include the surfacing, where the impact effect causes cracking of the mastic asphalt, reducing its service life, and the failure of a number 'King Posts'. These are slender hollow, steel circular sections which prevent excessive deflection of the bottom horizontal lateral bracing and are seen to vibrate significantly when a heavy goods vehicle travels over a defective half-joint.

In fact, what drivers experience is a series of steel upstands, every 18 metres, as they cross the bridge. Some of these upstands are 18 millimetres high therefore, drivers, especially in smaller wheelbase cars with firmer suspension, are experiencing a significant bump at less than one second intervals as they cross the bridge at 80 kph.

As might be expected this ride quality is an issue that drivers frequently comment upon. Unfortunately, due to the wear on the half joint and the deflections on the cantilevered sections damage is increasing with time leading to a greater step between the adjacent panels at the joints, which in turn leads to even greater discomfort for drivers.

In order to try to minimize damage and remedy this defect in the joint, a temporary fix has been developed as follows:

– The deck is leveled across the affected joint to provide as small a step across the joint as possible
– A steel beam section, called a billet, is bolted to the bottom flange of the stringer beam on one side of the half joint
– Stainless steel packing shims are packed into the horizontal gaps left at the interface of the half joint, and into the any gap between the top flange of the billet and the bottom flange of the stringer at the free end.

This work cannot be carried out whilst traffic is running on the affected bridge carriageway. Due to the high volume of traffic on the bridge, it is not practicable to close carriageways during the day so this maintenance work has to be carried out at night. This is obviously more expensive and carries a higher safety risk compared to daytime working. It also means that the maintenance crew are not available the next day to carry out other work. The billet detail is shown in Figure 9.

This billet detail was always intended to be a temporary measure until a permanent detail to remedy the problem could be found. However, like all good stop gaps, it has been around for more than 20 years and has been modified over those years. The first billets used were not long enough, the bolt group failed, and as a consequence, they kept disappearing into the water. A redesign has meant that ultimate failure of the billet does not now occur. Unfortunately, failure of the shims still does occur and these have to be repacked on a regular basis using overnight carriageway closures.

Following work carried out between consulting engineers W A Fairhurst of Glasgow, and Forth Road Bridge's own staff a new prototype detail has been designed by Fairhurst's engineers. This detail is shown in Figure 10.

The proposed arrangement will consist of a 'T' section welded to the bottom flange of the existing stringer beam directly underneath the web plate. Two channels bolted to the 'T' section will cantilever out to support the end of the other stringer beam. The replacement joint will be designed to resist both the downward and uplift force. The web plates of the existing stringers will be required to be stiffened transversely. In order to accommodate the differential movements and rotations of the deck panels a multidirectional bearing will be located in between the channels and the existing stringer beam. Obviously, it would be preferable if a scheme could be developed that did not include a bearing but this does not appear possible with out a more radical refurbishment of the whole deck. To enable the deck panels to be levelled the existing joint bearing block will be removed (Figure 11).

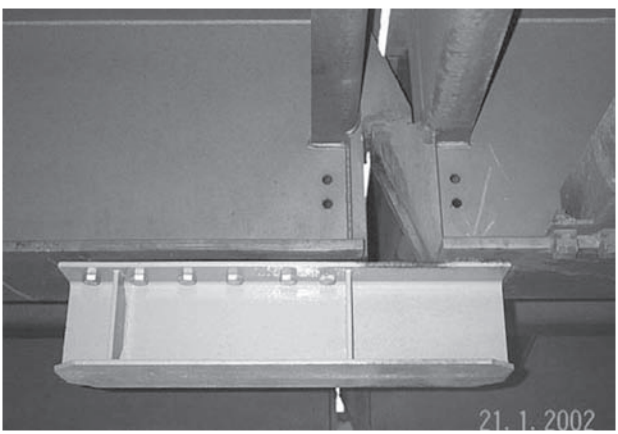

Figure 9. Billet detail.

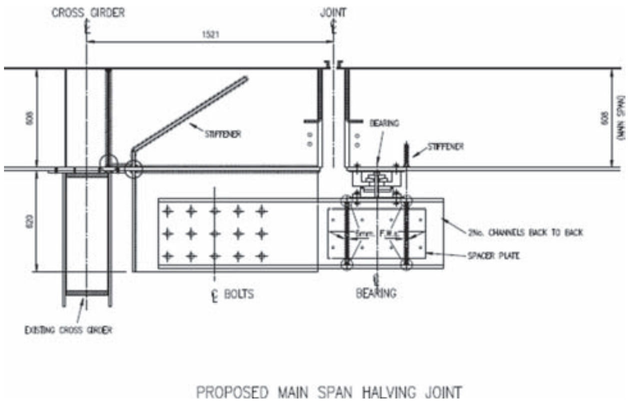

Figure 10. Proposed main span halving joint.

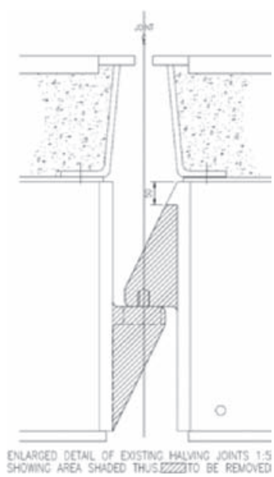

Figure 11. Existing bearing block removed.

A trial to install this new detail on eight joints at a cost of £300,000 is currently out to tender and should be installed by the end of 2011.

The traffic management required will be mainly single carriageway closures. Single footway/cycle path closures will also be required for storing and lifting the components of the replacement joints.

Monitoring of the results will, hopefully, give confidence that the scheme could be rolled out to all the joints. However, cost will be an issue as it estimated that the cost of replacing all the joints (excluding any delay costs) would be between £25 and £30 million pounds.

4.1 *Case Study 2 The anchorages at Forth Road Bridge*

The anchorages of the main cables of a suspension bridge are critical elements of the structure. At Forth Road Bridge, gravity anchorages in the form of massive concrete structures extending up to deck level were originally considered. However, they would have required extensive and very costly foundations, especially on the south side, where the bedrock is at a considerable depth. Therefore, it was decided to take advantage of the natural rock formation on both sides and drive separate tunnels at 30 degrees to the horizontal. These tunnels were formed within the rock at each of the four anchor points and filled with mass concrete.

The mass of the concrete, together with the weight of the overburden was sufficient to resist the 14,000 tonne cable force pulling the mass up the slope against the friction of the base. Additional frictional resistance was provided by the sides of the tunnels against the rock. On the north shore, competent rock is present near the surface of the ground and the rock tunnels extend down for a length of approximately 56.6 meters (186 ft). On the south shore rock is overlain with soil deposits and the rock as a whole is less competent. Consequently the tunnels of the south anchorages were lengthened and are over 79 meters (260 ft) long. It was noted (Proceedings of the Institution of Civil Engineers paper on the Forth Road Bridge 1965) that "although the tunnel anchorages required an increased length of cable there is no doubt that they are economical in this case as compared to gravity anchorages". A schematic of the anchorages is show in Figure 12. So, a decision was made purely on a cost basis to use concrete tunnel anchorages. Although there is no record of the sum saved, the decision seems perfectly sound as it utilizes the naturally occurring rock. Anchorages of this type had been used before and in fact, the anchorages of the Tamar Bridge in England, a 367 meter (1100 ft) span suspension bridge which was completed in 1961 used a similar construction (Anderson 1965). However, the anchorages at Tamar were considerable smaller with a length of around 18 meters (60 ft).

Of course, one main drawback is that mass concrete is not a very useful material in tension and is not strong enough to withstand the forces from the cables. On Tamar, 89 mm (3½") diameter solid steel rods were used to pre-tension the tunnels. However, on Forth, given the length of the tunnels, this was not practicable and the pre-tensioning was provided by using galvanized steel strands. The connection of the main cable to the concrete tunnels is made within the anchorage chambers. The cable, comprising 11618 wires in 37 bundles, is split into 37 separate strands at the splay saddle located just within the chamber. Each of the 37 strands then loops round a strand shoe. The strand shoes are then connected in pairs, by 89 mm ($3^1/_2$") diameter tie rods to mild steel crosshead slabs, two rods per shoe. The cross head slabs are clamped to the tunnel face using 114 pre-tensioned galvanized tendons,

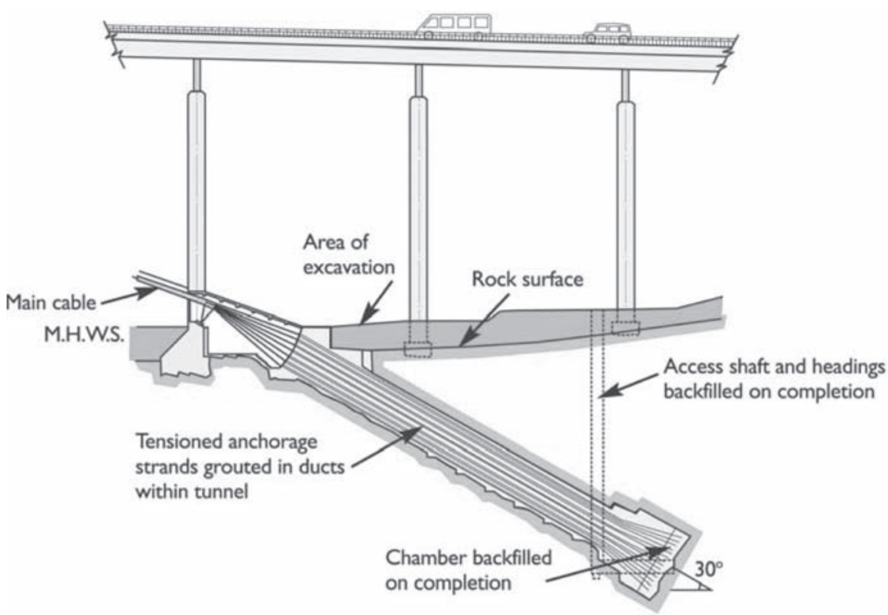

Figure 12. Schematic of anchorages.

of 6 tendons per slab (Figure 13). The strands were tensioned by hydraulic jacks applied at the lower end of the ducts and after tensioning the ducts were filled with grout. Access to these chambers was formed by excavating a vertical shaft which lead to tensioning galleries located to the rear of the tapered tunnels. These shafts and galleries were infilled with rubble and concrete on completion of the works (Figure 12).

This use of pre- tensioning strands in buried concrete anchorage tunnels at Forth was considered innovative at the time. Unfortunately, this form of construction can be vulnerable to corrosion and deterioration, especially in a saline environment such as is found at Forth.

This form of construction has another unfortunate drawback. There was no provision madefor either inspecting or maintaining the strands. Therefore, the most critical elements on the bridge are located in potentially the worst environment, with no access to inspect or maintain.

Records and papers acquired relatively recently relating to the construction of the existing anchorages highlighted various problems during construction particularly in relation to early depletion of the galvanizing protecting the pre-tensioning strands.

The reports on the issues and conditions encountered during construction determined the need to carry out a special inspection or investigation to try to establish the existing condition of the pre-tensioning strands. The design and supervision of the project is being carried out by W A Fairhurst & Partners.

Frequent inspections are carried out to monitor for movement within the anchorage chambers at the tunnel/strand shoe interface by Forth Road Bridge staff and no signs of distress or movement have been recorded. Therefore, this special investigation is about ensuring the long term structural integrity of the anchorages and establishing that the risk of failure remains low and within acceptable levels. The investigation is also a pro-active measure to ensure that all accessible parts of the structure are inspected.

There is guidance from the UK Department for Transport for inspecting post tensioning in bridges as it is acknowledged that there can be problems with this type of construction. The guidelines refer mainly to the difficulties in establishing the condition of post tensioning strand in bridge decks which are of similar structural form and construction as the pre-tensioned strands in the anchorage tunnels. These difficulties are exacerbated in a tunnel. Following much discussion, research and consultation with various specialists, it was concluded that excavation behind the anchorage chambers down to the top of the tunnel to expose, inspect and test the pre-tensioning strands was the preferred method of investigation.

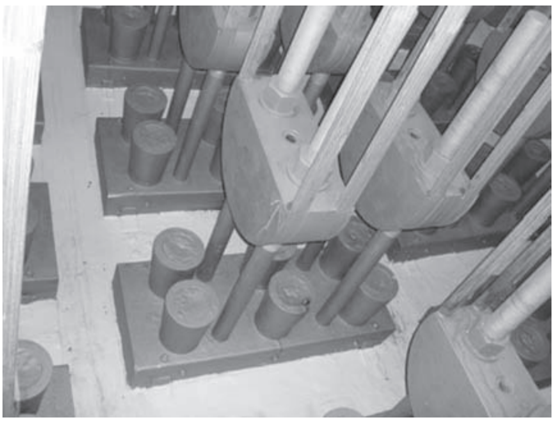

Figure 13. Anchorage strand shoe detail.

Acoustic monitoring and the use of radar have been considered but these methods have not appeared to show much potential in solving the problem alone but may provide some useful data. The use of magnetostriction scanning is also being considered once the strands have been exposed.

It is proposed to commence work at the South Anchorage as it is expected that if corrosion of the strands has occurred it will be worse at this location. The excavation work and visual examination and testing of the strands would be carried out in the first instance as a discrete contract. Any further work would depend on the results of this examination.

The excavation will be difficult as the ground conditions vary and there is methane present within the shale. The work is further complicated by the close proximity of the foundations to the viaduct piers. Any insitu testing of the anchorage sockets will also be challenging as access is very difficult within the anchorage chambers. Safety of the workforce, the bridge and users will govern all aspects of this complex project.

Work on the anchorages is not anticipated to involve major disruption to bridge traffic. However, given the nature of the works there will be significant environmental issues to be dealt with especially with regard to noise, dust and discharge from the excavation and hydrodemolition.

A £6.6 million sum has been set aside for this work in the current capital plan for the period 2010/11 to 2014/15. However, it is difficult to determine the exact cost of the work as the full extent of the investigation will not be known until the strands are exposed.

Due to the complex nature of the work Flint & Neill Ltd were appointed to chair a Peer Review Panel in order to audit and review the work being carried out by Fairhurst. The panel members are David Mackenzie of Flint and Neill assisted by Peter Sluszka of consultants Ammann & Whitney and Bill Valentine, Chief Bridge Engineer, Trunk Roads Network Management, Transport Scotland.

Following comments and recommendations of the Peer Review Panel the project is out to tender with work on site expected to start in August 2011 and be completed on site in September 2012.

To assist in this planning statistical Condition and Strength Models of the anchorages are being developed with the assistance of the University of Strathclyde in Glasgow, prior to the inspection work taking place on site. These models will assist in determining the likelihood of the condition and strength of the anchorages being at a certain level based on the information obtained from the inspection.

From this investigation (Quigley & Walls 2010) an assessment will provide a determination of the likely current strength of the anchorages and the condition survey will assist in the difficult task of estimating future strength.

Depending on the findings of the investigation, challenging engineering decisions may have to be taken which may impact on the load carrying capacity of the bridge. As stated previously, all strand sockets are currently firmly in bearing with the anchorage plates and this is taken to indicate that there has been no complete loss of pre-tensioning in any one tendon. However, as confirmed by the audit, significant corrosion cannot be discounted.

Contingency planning based on a number of possible scenarios will be developed prior to the inspection to ensure that all foreseeable events are taken into account. This work will involve discussion and consultation with other agencies and stakeholders.

5 CONCLUSIONS

These two examples show decisions taken at the design stage mainly for economic reasons have had far reaching effects on the well being of Forth Road Bridge and those that use the crossing. They were taken with the best of intentions but without any thought on future inspection or maintenance. It may well be that we now consider that the knowledge materials available to us now and our bridge management techniques mean that we will make the right choices and avoid such errors in the future. However, it is only my own personal view that unless there are experienced maintenance engineers involved in some capacity in the design process then short term cost gains are more likely to prevail over potentially longer term cost savings as a result of poor design.

The Romans not only insisted that a council of experts certified a bridge before construction, but they also required the builder to provide a deposit as a guarantee for the stability of the structure for 40 years.

Following this, in the middle ages, the Church sponsored the construction of bridges, the Pope himself was referred to as 'Pontifes Maximum', the 'supreme bridge builder'. What the Church did was insist that monies were not only raised and tolls levied for construction but also for continual maintenance. The order raised to build a bridge was to "build and maintain" (Ryall 2010).

So there is nothing new in recognising that bridges need to be maintained and adequate funding needs to be made available so that maintenance can be made possible. There is also nothing new in recognising that if bridges are to last then durability needs to be built in at the design stage. To ensure these things happen it is essential that experienced maintenance engineers have an effective say at the design stage and in the case of large bridges, their maintenance and operation is carried out by a dedicated on site team.

THANKS

I would like to thank Colin Clark and Katarzyna Kukla of WA Fairhurst & Partners; and Anne Muir of Forth Road Bridge for all their help in the preparation of this paper.

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Chapter 6

The New Lake Champlain Bridge design process

T. Potts & J. Billings
HNTB Corporation, New York, NY, USA

J. Bridges & J. Boni
New York State Department of Transportation, Region One, Schenectady, NY, USA

ABSTRACT: The Lake Champlain Bridge, also known as the Crown Point Bridge, was an historic steel truss bridge stretching 2,187 feet across Lake Champlain between Crown Point, New York, and Chimney Point, Vermont. After significant deterioration was discovered in the unreinforced concrete bridge piers in the fall of 2009, the bridge was abruptly closed to all traffic. An emergency declaration was issued by the Vermont Agency of Transportation and by New York State. HNTB prepared a Safety Assessment Report, and together with New York and Vermont, concluded that the existing truss bridge should be demolished. The two regions connected by the bridge are economically linked and share services, including hospitals and fire departments. Given the criticality of the bridge for the regional economies, it was imperative that the design and construction of a replacement bridge be expedited as soon as possible. Together with New York, Vermont, and the FHWA, the HNTB design team developed plans for the demolition of the original truss bridge and the design and construction of a new network tied arch bridge. Utilizing design-bid-build contracting methods, the demolition of the existing deteriorated bridge, and the design and construction a new signature crossing was to be completed within twenty months. This paper discusses reasons for the bridge closure and the innovative design-bid-build approach that enabled an extremely expedited replacement of the Lake Champlain Bridge. The design was completed under a compressed schedule with traditional linear functions done concurrently with bid advertisement, packaging and permitting. The teamwork between the two states and numerous local and federal agencies allowed for a cooperative process that can serve as a model for success in collaboration amongst various agencies and interest groups, thus aiding in significant time and cost savings. Topics of the paper include multi-agency coordination, the importance of the public involvement process, environmental process, design methodology, constructability, and measures taken to expedite bridge construction.

1 PROJECT BACKGROUND

The original Lake Champlain Bridge was an iconic continuous truss bridge designed by Charles M. Spofford, an early pioneer in design methods for continuous trusses. The bridge served as a vital economic link that connected Crown Point, New York and Chimney Point, Vermont, crossing over the southern end of Lake Champlain (Figure 1). Comprised of 14 steel spans totaling approximately 2,187 feet, the two-lane bridge opened to traffic in 1929. Of the 14 spans, five of the spans were deck trusses, one span was a half-through truss, and the remaining were steel girder spans (Figure 2). The combination deck and through trusses at midspan, Spans 6–8, have been noted for historic significance and aesthetic character.

The Lake Champlain Bridge Commission owned and operated the Lake Champlain Bridge until 1987 when the commission was abolished. At that point, the bridge changed ownership to the New York State Department of Transportation (NYSDOT) and the

Figure 1. Location of the Lake Champlain Bridge.

Figure 2. Photo of the original Lake Champlain Bridge.

Vermont Agency of Transportation (VAOT). The bridge served two sparsely populated regions, with average daily traffic of roughly 3,500 vehicles. However, there are very few alternative crossings for Lake Champlain resulting in significant detour lengths and increased crossing times. For these reasons, the regional importance of the crossing could not be underestimated.

It all began in the summer of 2009 when New York and Vermont executed a bi-state agreement for the commencement of project scoping to address the condition of the original Lake Champlain Bridge. The bridge had reached 80 years of service life, and a study was to be undertaken to determine the need for possible major rehabilitation or replacement. Funding of the project would be split evenly between the two states with 80% Federal support. New York State would maintain contractual control of the project. However, project oversight would be provided by the three co-lead agencies: the New York State Department of Transportation (NYSDOT), the Vermont Agency of Transportation (VAOT), and the Federal Highway Administration (FHWA). HNTB Corporation out of New York City, with a team of four sub-consultants, was contracted to carry out project scoping, with the intent of continuing through the environmental impact study, and possible bridge rehabilitation or bridge replacement. Project scoping through final design was estimated to take approximately five years.

2 THE ORIGINAL LAKE CHAMPLAIN BRIDGE CIRCA 1929

2.1 *Historical significance*

The original Lake Champlain Bridge opened in 1929 and was widely known for its iconic form. The bridge form was a particularly elegant application of Spofford's ideas that demonstrate the efficiency of continuous structural systems. This bridge had an important place in the evolution of continuous trusses and the practice of bridge engineering in the United States. In February 2009 the Lake Champlain Bridge was granted approval to be entered in the National Register of Historic Places. While not formally listed by the time of closure, the structure had met the eligibility requirements regarding "age, integrity, and significance" (National Park Service, 2011).

2.2 *Unusual design aspects of the existing bridge*

The Lake Champlain Bridge was an early design of a continuous truss and its chief designer, Charles M. Spofford, was influential and active in the analysis, design and construction of such structures. His book entitled *Theory of Continuous Structures and Arches*, published in 1937, discussed in detail the design aspects of the continuous truss bridge. This structural form was a clear early innovation in the design of continuous trusses, and Spofford's role in its development cannot be argued.

One of the challenges with continuous trusses is that forces in the truss system are dependent upon support geometry and must be prescribed, given the structure's static indeterminacy. For construction, after closure of the main span superstructure and prior to installation of the bearings, the structure must be jacked into its final geometry. This process is described by Francis Griggs (2007) as well as Spofford in his writings on continuous truss bridges, and is a critical aspect of the design and construction of the structure. As will be discussed in more detail below, the superstructure's sensitivity to pier movements was a key concern. Another unusual aspect of the original Lake Champlain Bridge design was the use of plain rather than reinforced concrete for the piers, particularly given the piers' slenderness. In addition, there were no obvious considerations in the pier design for the potential of ice abrasion. Spofford notes "that the piers are in a fresh-water lake with little current and are practically free from danger of abrasion from ice and floating objects" in an ASCE paper (Spofford 1931). Based upon visual observations, concrete cores near the waterline, and past and current diving inspections as will be outlined below, ice abrasion and ice pressure resulted in significant damage to the piers over the 80 year service life of the bridge.

Finally, for concrete placed below water in deep open cofferdams, the use of a patented one yard dump bucket instead of a conventional tremie pour was highly unusual. This construction method results in potential planes of weakness throughout the caisson which reduce the reliability and therefore enhance the risk of failure of this element. This is of particular concern given the difficulties in assessing the condition of the caisson as well as implementing an effective rehabilitation to these massive underwater elements.

2.3 *Historic significance of surrounding land*

Both the New York and Vermont shores contain several historical and archaeological sites, some dating back thousands of years. At the New York approach, remains of two 18th Century forts lie adjacent to the northern side of the bridge: Fort St. Frederic and Fort Crown Point. The historic Toll House is located just south of the bridge approach road, with a 19th Century lighthouse further to the south. At the Vermont approach there are remains of an 18th Century French Fort to the north, and an historic tavern to the south. Additionally, artifacts dating back more than 7500 years have been found at the Chimney Point Historic

Site, directly adjacent to the bridge. The rich historical and archaeological setting added to the value and meaning of the original bridge's iconic form.

3 THE CONDITION OF THE LAKE CHAMPLAIN BRIDGE IN 2009

3.1 *Pier deterioration observed—Fall 2009*

There had been a significant amount of rehabilitation and retrofit work on the original bridge over its 80 year life, with the most extensive work completed in the early 1990's. Rehabilitation work included replacement of the existing concrete deck with a concrete filled steel grid deck, installation of new traffic barriers, incorporating drainage improvements, bearing rehabilitation, post tensioning containment of piers, gusset plate repairs, and other miscellaneous steel repairs. However, in spite of this major rehabilitation, deterioration was progressing rapidly both in the superstructure and the substructure elements. Recent inspections make note of excessive deterioration and structural inadequacy of the bearing pedestals. The superstructure steel was deteriorating rapidly given the localized failure of the painting system, particularly at the truss connections. Many primary load carrying members and connections exhibited heavy section loss and localized perforations, particularly structural steel adjacent to areas with heavy de-icing salt exposure at the roadway level. Following the 2009 inspection, the original bridge was reduced to one lane of traffic and had a posted weight restriction of 40 tons.

Of most concern, however, was the significant cracking in the piers and the freeze-thaw induced damage to the piers, both at the bearing seats and at and below the waterline. Overall, Piers 5, 6, 7, and 8 exhibited severe deterioration of the existing concrete at the water level. A drop in the water level in September 2009 exposed surface deterioration much worse than previously noted (Figure 3). Upon further investigation by NYSDOT Region One, it was discovered that Pier 5 exhibited approximately 30% section loss. NYSDOT took core samples from Pier 5 that verified this 30% section loss. Qualitatively, this equated to 18 inches of deteriorated concrete around the perimeter of the pier. A similar level of severity of section

Figure 3. Pier 5 concrete deterioration in September 2009 *(Photo courtesy of NYSDOT)*.

loss was later observed at Piers 6, 7, and 8. Numerous repair contracts had been let for the piers and bearings starting as early as 1945. This in and of itself is critical in understanding the existing condition of the piers. It is highly unusual for concrete and bearing repairs to take place on a bridge that is only 15 years old, as the Lake Champlain Bridge was in 1945. Repairs performed at this stage in the bridge's life suggest that by 2009 the piers may have been under distress for decades. It is also important to note that these repairs were superficial, as they did not increase the structural integrity of the piers. In fact, the previous concrete repairs masked some of the structural deterioration to the piers and the bearings.

3.2 *Bridge closure*

Upon discovering the extensive pier deterioration, NYSDOT & HNTB determined that an immediate in-depth analysis of the piers was warranted. HNTB analyzed the reduced pier sections, and determined the remaining load carrying capacity against vertical and lateral load demands. On October 16, 2009 a discussion between NYSDOT and HNTB led to bridge closure at 1:30 pm. There was no warning to the local public on that Friday afternoon, and many found themselves with an extended commute home, of two or more hours longer than normal, Friday night. The public's initial response was that of anger and frustration, and concern about their short term ability to travel across the lake for business or medical purposes. A Declaration of Emergency was issued on October 20, 2009 by Vermont Secretary of Transportation David Dill, and a State Disaster Emergency was declared under Executive Order No. 28 by Governor David A. Paterson of New York on October 21, 2009.

The deterioration of the piers represented a significant threat to public safety given the potential that a localized failure could cause catastrophic structural collapse. Given the structure's height above water and the depth of the lake, a catastrophic collapse could result in multiple fatalities. With the information furnished by the HNTB team, NYSDOT concluded it was too dangerous to risk leaving the bridge open to traffic given the level of uncertainty and potential for abrupt failure.

In an effort to address the concerns and questions of the public, two public information meetings were held October 27 and 28, 2009 in both New York and Vermont. Hundreds of concerned citizens attended to voice their opinions. It was clear to NYSDOT and VAOT that a temporary solution was needed in an expeditious manner. On October 30, 2009 a multi-agency meeting attended by representatives from New York and Vermont was held to discuss the possibility of constructing a temporary ferry just south of the bridge location. Due to the historical and archaeological significance of the area, cooperation from all involved agencies was imperative from the beginning of the planning process. The co-lead agencies included all stakeholders and agencies that would be involved throughout the decision making process, so as to avoid delays during the approval and permitting phases. Agencies such as the NY and VT State Historic Preservation Offices, Adirondack Park Agency, Vermont Fish and Wildlife, and numerous others were involved from the early stages in an effort to familiarize those involved with the project background, possible historical and environmental effects, and the need to expedite the approval process. Coordination between involved agencies proved successful, and on November 11, 2009 FHWA, New York and Vermont granted NEPA approval for the proposed temporary ferry project in both states. The following week, the bridge emergency standby contractor began work clearing the site for the temporary access roads.

3.3 *Steps taken after bridge closure*

Following the decision to close and replace the bridge, as discussed below, there were four major projects that would occur: short-term mitigation for the public following the bridge closure; design and construction of the temporary ferry system; bridge demolition; and new bridge design. In normal circumstances a traditional approach would be that of linear functions: once a task is complete, the next begins until all tasks are completed. In the emergency declaration following bridge closure, however, the traditionally linear tasks were performed concurrently to expedite the processes as much as possible. Each of the four tasks was treated as an independent project managed by separate personnel at NYSDOT. Communication was critical to keep all involved parties informed. In terms of short-term mitigation, NYSDOT and VAOT worked to establish bus routes for daily commuters to avoid driving in the harsh winter months. Park and ride lots were established for commuters using these services. Additionally, existing ferry systems were utilized. The Essex County Ferry and Ticonderoga Ferry are within 30 miles of the bridge site, and their operational costs were subsidized by New York and Vermont. The two states "winterized" the Ticonderoga Ferry in an effort to keep it operational until the temporary ferry could open. Their efforts extended the hours of operation to accommodate the increased traffic demand.

While not equal to the ease and efficiency of a bridge crossing, the existing ferry systems provided some relief until the new temporary ferry opened on February 1, 2010. The temporary ferry at the bridge site currently runs 24 hours per day, 7 days a week, and is completely subsidized by New York and Vermont. The new temporary ferry is able to carry more cars than the pre-existing ferries, and is capable of supporting commercial truck traffic. The increased capacity aids to minimize delays in crossing, and winter ice breakers allow the ferry to continue operation throughout the year. The ferry is adjacent to the original bridge site, and will be removed by the bridge contractor once the new bridge opens to traffic.

4 BRIDGE SAFETY ASSESSMENT

4.1 *Diving inspection findings*

The week after bridge closure, a diving inspection was performed to gather additional information on the condition of the piers below water. At Pier 5, a horizontal crack was discovered below lake level that appeared to extend through the full cross section of the unreinforced concrete pier. In the days following, numerous cracks, measuring up to 3/8" wide were found in the inspected piers (Figure 4). Upon completion of underwater inspection of Piers 5, 6, 7,

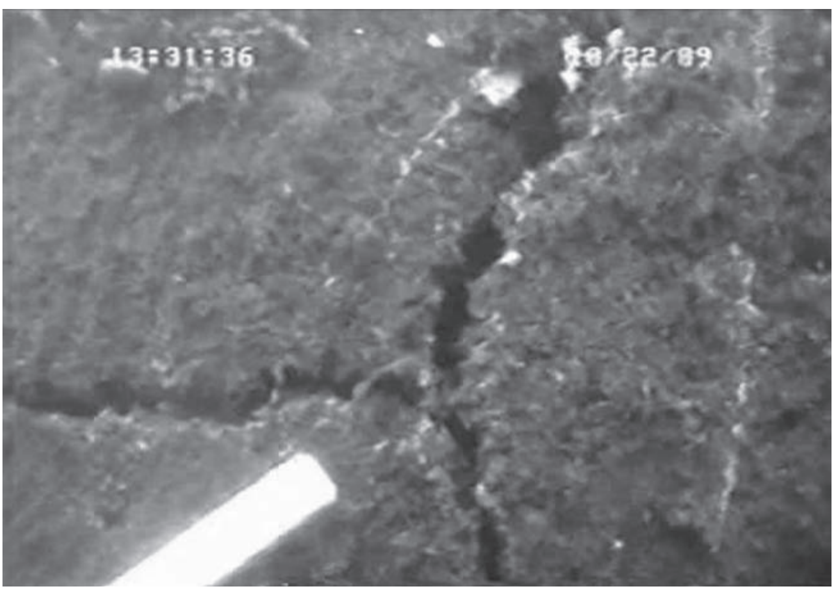

Figure 4. Severe cracking observed during underwater inspection in October 2009 *(Photos courtesy of NYSDOT)*.

and 8 cracks were found in all piers. The underwater inspections are typically completed every five years, with the most recent one occurring in 2005 for this bridge. The 2005 inspection had documented cracks, but not to the extent that the 2009 inspection found.

4.2 *Pier monitoring*

To further assess pier stability, HNTB installed triaxial accelerometers/bi-directional tilt meters at Pier 5 and Pier 6 to assess pier movements (Figure 5). Installation of these remote sensors was completed on November 4th, 2009. As was feared, a striking correlation between temperature and tilt was observed after the first few days of monitoring. From this data, it was clear that the bearings were frozen and inducing longitudinal forces into the piers with changes in temperature. It was believed that lateral movement cracked the unreinforced piers forming a hinge that allowed the piers to 'rock' with fluctuating temperatures.

The monitoring system installed consisted of a network enabled sensor unit capable of measuring three axes of acceleration, two axes of tilt, and ambient temperature. These sensors utilized a microwave link to communicate with a base station computer located in the nearby Lake Champlain Visitors' Center on the New York shore. The base station recorded and locally cached the data which was continuously streaming from the sensor. The base station, in turn, was able to broadcast data over the internet providing near-real time external access to pier movements. The ability to continuously track temperature and tilt of the piers allowed inferences to be drawn regarding the relationship between temperature shift and structural behavior. As can be seen in Figures 6 & 7, the strong correlation between temperature and tilt of Pier 5 in the longitudinal direction indicated that the pier was 'rocking' about a fracture induced pier 'hinge' point, as the steel superstructure underwent thermal expansion and contraction.

A secondary function of the sensors was to monitor the dynamic behavior of the structure through the tri-axial accelerometers. Highly dynamic events such pier cracking release energy into the structure which would be recorded by the accelerometers. Significant events generate a unique and recognizable response spectrum, which allows thresholds to be set to trigger

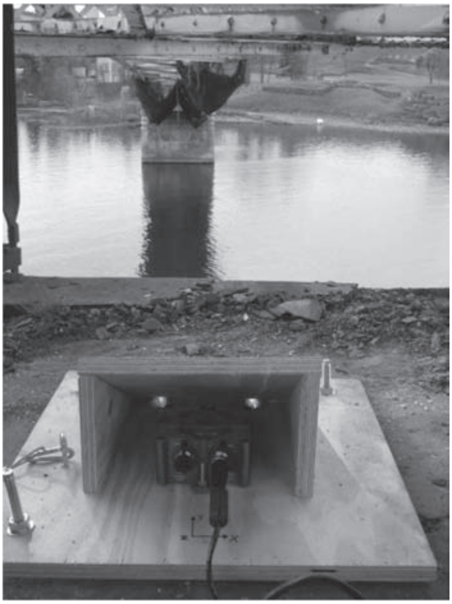

Figure 5. Sensor installed at Pier 5.

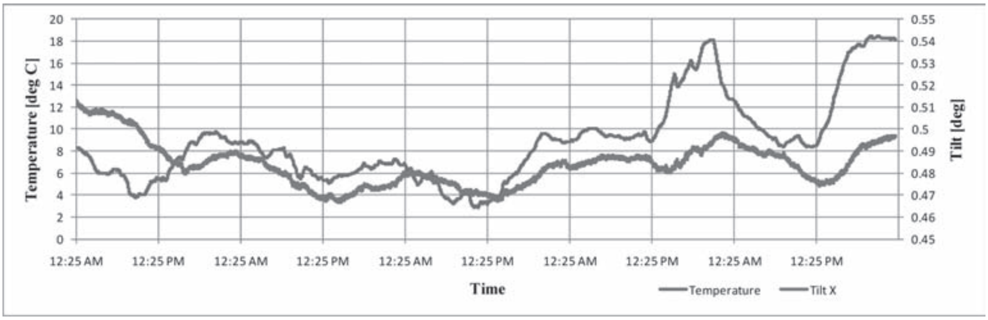

Figure 6. Transverse tilt (X-direction).

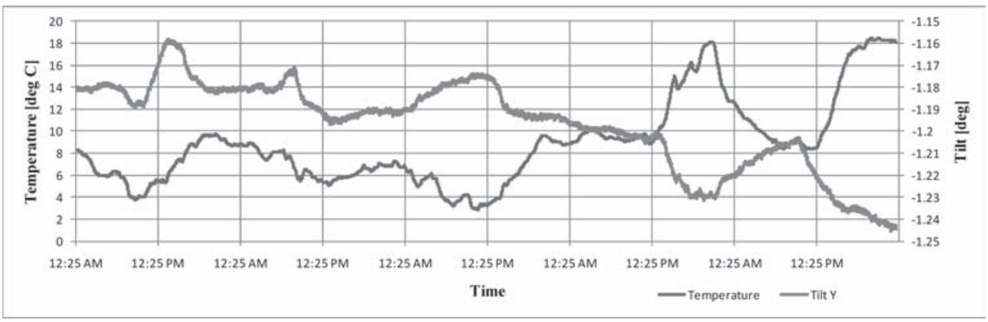

Figure 7. Longitudinal tilt (Y-direction).

emergency notification to all necessary parties. This system was able to provide early warning of changing structural conditions to workers on and around the bridge, as well as engineers monitoring the bridge remotely.

4.3 *Recommendations for bridge demolition*

Based on visual inspections supported by sensor data captured from Pier 5, it was determined that the existing bearings were frozen and not functioning as originally designed. In a situation where a bearing or bearings are frozen and the bridge superstructure expands or contracts under temperature changes, pier flexure occurs to accommodate the bridge movement. In the case of the Lake Champlain Bridge, the additional forces created in the structure, in conjunction with unreinforced concrete piers having low elastic ductility, the behavior of the bridge became unpredictable.

For the unreinforced concrete piers of the Lake Champlain Bridge the potential for sudden, abrupt failure could not be ruled out. Freeze-thaw deterioration would continue to damage the piers at water level. Lake icing and thrust associated with thermal ice sheet movements could produce large horizontal loads in the piers, well beyond the piers' flexural capacity. Wind loads alone could introduce stresses that would exceed computed pier capacities. Additionally, frozen bearings introduce longitudinal forces capable of rupturing the unreinforced piers at their weakest locations. The potential for any of these loads, individually or in combination, to precipitate pier failure and collapse of the structure could not be dismissed.

In particular, Piers 5 and 7 had full width horizontal cracks at two locations below the water line, as well as numerous vertical and inclined cracks that intercepted the horizontal cracks. If any major cracks developed diagonally in the pier shaft or concrete deterioration reduced the contact bearing area between concrete segments, the pier could fail without warning. This mass failure of an unreinforced pier would be sudden and catastrophic.

Figure 8. Demolition of the original Lake Champlain Bridge, December 28, 2009.

HNTB evaluated both short-term and long-term repair strategies that involved pier strengthening and pier replacement. Estimates of construction duration and costs to extend the serviceable life of the piers were discouraging. Of utmost concern was public safety, and given the risks associated with rehabilitation, HNTB recommended against this option. Given the long detour lengths, the importance of the bridge to the regional economy, and the need for a safe and reliable crossing at this location, both temporary and permanent rehabilitation strategies were deemed inadequate. With the severe regional impacts of this bridge closure, replacement of the Lake Champlain Bridge on its existing alignment was recommended to expedite regulatory approvals and minimize impacts to cultural resources in close proximity to the bridge site.

4.4 *Lake Champlain Bridge demolition*

The original Lake Champlain Bridge was demolished by explosives on December 28, 2009 (Figure 8). Removal of the bridge debris from the lake commenced immediately after the implosion. Removal of the unreinforced concrete piers involved hoe-ramming the structures above water and the placing the removed debris on barges. The portion of the water piers remaining below water were removed by detonation. Sections of the piers more than two feet below mudline were left in place. Bridge demolition was completed in time for the new bridge contractor to mobilize for construction.

5 THE PUBLIC INVOLVEMENT PROCESS

5.1 *The Public Advisory Committee (PAC)*

The original Lake Champlain Bridge was an iconic and historic structure that was nominated to become a National Historic Landmark at the time of closure. The bridge played an important role in the lives of the local community, and many citizens expressed interest in the decisions being made as to the future of the bridge. Prior to the start of project scoping a Public Advisory Committee (PAC), comprised of elected officials and representatives from various interest groups and public agencies, was formed. The role of the PAC was to meet with NYSDOT, VAOT, FHWA, and HNTB team representatives periodically to provide input, ideas and concerns regarding the bridge during project development.

5.2 *Public frustration*

Following bridge closure there were common feelings of frustration and distrust toward the state agencies by the public. The immediate response of NYSDOT and VAOT to the public was to hold a series of PAC and public information meetings to explain the short-term mitigation plans. To that end, the states developed a comprehensive public involvement plan utilizing the expertise of HNTB's team. Constant collaboration and decision-making was necessary to meet the bridge replacement schedule. This was only made possible when all involved agencies, parties, and members of the public put aside any differences and moved forward to meet an expedited bridge design and construction schedule. The collaboration between the co-lead agencies, consulting parties, permitting agencies, the design team, and the public was unprecedented, and is perhaps the single-most important factor in successfully expediting the bridge design process.

5.3 *Intensive 6-day public review process*

Once the decision was made to demolish the Lake Champlain Bridge, the co-lead agencies immediately began the Section 106 review process focusing on the requirements of the National Historic Preservation Act. The Section 106 and public involvement process were both started in mid-December 2009. An intensive schedule consisting of a six day review process was established, and included meetings with the historic consulting parties, a PAC meeting, as well as a series of public information meetings to introduce the proposed bridge replacement alternatives.

Prior to the commencement of the official design review process, the design team presented the co-lead agencies with five suggested bridge alternatives: a long span steel girder bridge, a concrete segmental bridge, a steel composite cable stay bridge, a concrete extradosed bridge, and a network tied arch bridge (Figure 9). HNTB believed these five alternatives to be the most feasible and appropriate for the proposed location, to which NYSDOT

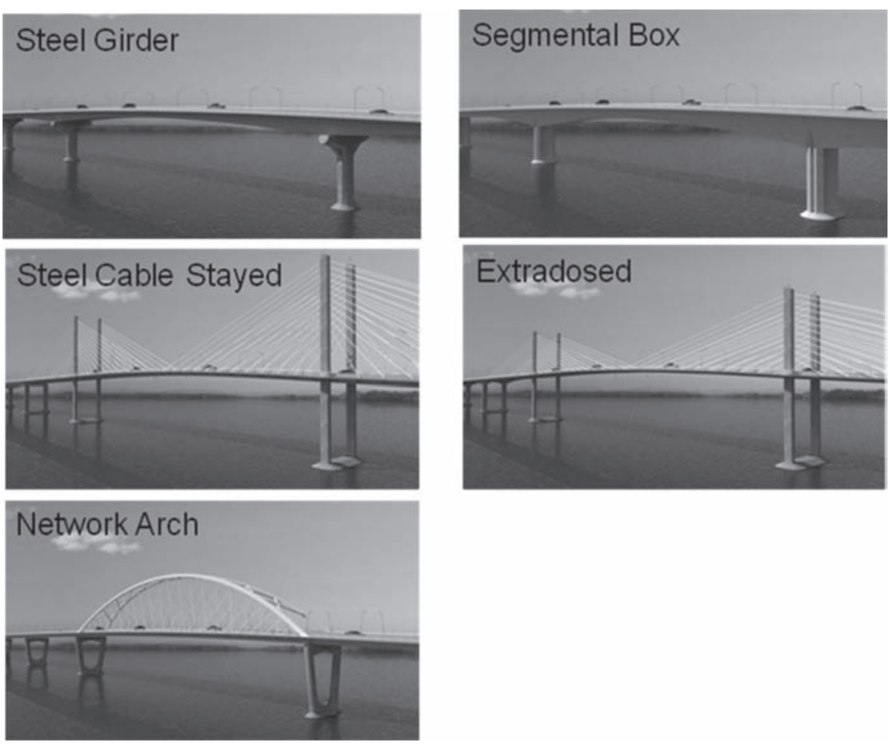

Figure 9. Renderings of the five original bridge replacement alternatives.

and VAOT agreed. The dismissed bridge types included a conventional truss bridge, lattice truss bridge, concrete arch bridge, fin back bridge, and suspension bridge. These bridge types were dismissed based upon high life cycle costs, lack of redundancy in form, vulnerability to progressive collapse, incompatible soil conditions, inefficiency for span length, construction complexity and long construction schedule. Of particular interest were reasons for dismissing a conventional truss bridge as a replacement option. The design team emphasized the issues with lack of redundancy, vulnerability to progressive collapse, high life cycle costs, and the increased costs associated with maintenance.

The design review process officially commenced on December 10, 2009 with a meeting between the co-lead agencies, the design team, and the historic consulting parties. A 30-day review process was required per Section 106 regulations, and the process began with this meeting. The lead design agencies presented each of the five proposed bridge alternatives to the Historic Consulting Parties. During discussions, the benefits and shortcomings of each bridge type were presented for discussion. Issues such as cost, construction duration and complexity, maintainability, navigational clearance, redundancy and safety, and overall aesthetics were considered. Additionally, each of the dismissed bridge types was presented with explanations as to why that bridge type was not considered. Throughout this meeting, discussions were held directly between the consulting party representatives and the structural engineers designing the bridge, in an effort to alleviate possible concerns or issues that could arise during the 30-day review process. NYSDOT and VAOT were present at the meeting to mediate the discussions and provide input when needed. The direct interaction between the design engineers and the consulting parties was valuable in establishing relationships and trust before moving into final design. The bridge closure and demolition left some consulting party representatives discouraged and resistant toward a new bridge type for replacement, since the original bridge had such historic meaning to the area. During this meeting the design team was able to explain the risks and drawbacks associated with replacing the bridge with another truss bridge, as well as discussing the potential benefits of using a more modern bridge type. The meeting was attended by a smaller group, allowing for productive dialogue between the design team and the consulting parties. Additionally, concerned individuals had the opportunity to identify issues to be address during design, such as historical and archeological impacts associated with an increased bridge footprint. It was at this point in the process when the consulting parties began to move away from feelings of anger and frustration, and embraced the concept of constructing a new bridge.

The following day, December 12, 2009, members of FHWA, NYSDOT, VAOT, and the bridge design team met with the PAC to reveal the five bridge types being considered for bridge replacement. For the bridge type selection meetings, the PAC was expanded to include more than two dozen additional members of the public interested in the project, including planners, business representatives, historians, preservationists, transit representatives, as well as local high school students. This expanded group was referred to as the PAC+. The design team presented benefits and drawbacks for each bridge type, and provided animations and renderings for review. The PAC+ provided feedback on the five bridge replacement alternatives. Additionally, PAC+ members voiced concerns regarding lane and shoulder widths, inclusion of sidewalks, aesthetic value to the surrounding area, and other design aspects. The PAC+ showed an overwhelming support for the network tied arch bridge, but many felt the bridge lacked fluidity and the transition from arch span to standard steel girders was too abrupt. Immediately following the meeting, NYSDOT and HNTB agreed that a sixth bridge concept should be developed and presented. That evening, the HNTB design team created a sixth option for the Public Information Meeting to be held the next morning. The sixth option included a network tied arch main span, supported by triangular rigid frame elements at each end of the arch (Figure 10). This design became known as the "modified network tied arch" alternative throughout the public review process.

This revised network tied arch alternative was unveiled the next day, at the Public Information Meeting held in Ticonderoga, New York. This all-day event drew over 600 attendees

Figure 10. Rendering of the modified network tied arch alternative.

from the surrounding area, and the design team gave three presentations introducing the proposed bridge types. The public was then given the opportunity to interact with and ask questions to members of NYSDOT, VAOT, and the design team engineers. Information stations were set up for discussions regarding bridge demolition, the temporary ferry, bridge commemoration, and new bridge design concepts. Members of the public that attended the meeting were given questionnaires to vote for their preferred bridge type, as well as include any feedback or comments for the new bridge. Additionally, an on-line survey was conducted for members of the public unable to attend the meetings. In addition to the 600 people that attended the public information session, over 3,000 online surveys were completed. There was an overwhelming level of support for the sixth alternative, the modified network tied arch. The original network tied arch was second in number of votes, and together, the two network tied arch concepts received the majority of the votes.

The PAC reconvened on Tuesday, December 15, 2009 to make a formal recommendation of the new bridge type to the Commissioner of NYSDOT and the Secretary of VAOT. As expected from the previous public meeting, the PAC recommended the modified network tied arch alternative as their preferred replacement bridge type.

Although condensed, the six day public review process was a critical step to progress the new bridge design. Involving the public directly in the new bridge type decision opened up communication and demonstrated the value of public input to the co-lead agencies and the design team. Additionally, by adding a sixth bridge type alternative based on the PAC+ comments, the co-lead agencies and design team demonstrated their willingness to listen and adapt to the needs and desires of the public. The overall level of trust and cooperation from the public increased as a result of the six day public review process, and enabled the remainder of the design phase to progress as expeditiously as was needed to meet the project schedule.

6 THE ENVIRONMENTAL PROCESS

6.1 *The NEPA process*

The National Environmental Policy Act requires that a new bridge undergo one of three different forms of an environmental review. The three levels range in complexity and include a Categorical Exclusion (CE), an Environmental Assessment (EA), and an Environmental Impact Study (EIS), with the CE being least complex and the EIS being most complex. The original project intended to include a full EIS, with the entire NEPA process estimated to last five years. In an effort to expedite the typical NEPA process for the new bridge, strategic planning and decision-making took place at the beginning of preliminary design. Of particular importance was the decision to construct the replacement structure along the same alignment as the original bridge. This critical decision reduced the NEPA documentation to a Categorical Exclusion with Documentation (DCE), thereby expediting the environmental review process from years to weeks. Similar strategizing efforts were made throughout the design process to minimize delays to the greatest extent practical, while still meeting current standards of practice.

6.2 *Section 106 programmatic agreement*

In addition to the NEPA required documentation, a Programmatic Agreement was executed as part of the National Historic Preservation Act, Section 106 process. The Programmatic Agreement is contracted between FHWA, New York State Historic Preservation Office, New York State Department of Environmental Conservation, Vermont State Historic Preservation Office, Advisory Council on Historic Preservation, NYSDOT and VAOT. Included in the agreement are provisions for environmental and archaeological compliance during the construction phase, as well as measures for mitigation and restoration to the lands affected by bridge construction. Additionally, actions to commemorate the original Lake Champlain Bridge are required to mitigate the loss of the historic structure. The agreement was incorporated into the final design contract documents to emphasize the historical, archaeological, and environmental sensitivities of the project, as well as make the Contractor aware of his responsibilities during construction.

6.3 *Permitting*

The touchdown points at each end of the new bridge are characterized by rich historical and archaeological resources. Limiting the area of impact by maintaining the same bridge alignment reduced the time required for the environmental review process. Permitting was another aspect of the new bridge design that required strategic planning in order to meet the strict design schedule. Due to the environmental, historical, and archaeological sensitivity of the area, numerous permits were required in each state. NYSDOT, VAOT, FHWA, and the design team worked closely throughout the design process to compile permit application forms and backup information as efficiently as possible. Agencies requiring permitting were made aware of the emergency nature of the project, and agreed to expedite their reviews where possible. Agencies such as the US Coast Guard, US Army Corps of Engineers, Department of Environmental Conservation in both states, Adirondack Park Agency, Vermont Fish and Wildlife, and several others, required permits prior to commencing new bridge construction. Because of effective communication between agencies, and the high level of cooperation that resulted, all permits required for the new bridge were in place well before the project was awarded on May 27, 2010.

7 PRELIMINARY DESIGN PERIOD

7.1 *Establishing design criteria*

With the PAC recommending a bridge type on December 15, 2009, NYSDOT initiated Preliminary Engineering on December 16, 2009. The HNTB design team worked closely with NYSDOT, VAOT, and FHWA and held early technical coordination meetings. The overall design concept was presented to the co-lead agencies on January 6, 2010 for feedback and comment. Preliminary drawings were distributed at the meeting that included proposed geometry, typical bridge sections, and conceptual details for primary structural members. The meeting set the necessary design criteria, established the roadway cross section, and identified functional needs of the new structure. Design issues were brought up for discussion and critical decisions were made prior to beginning final design. All required groups were involved in the January 6 meeting, and those not present were able to acquire preliminary drawings. The co-lead agencies took a proactive approach to ironing out major design issues as early as possible. There was no room in the schedule for changing direction or modifying the basic design concepts once final design began, thus engaging all involved agencies was critical in making conceptual design decisions.

One particularly important design decision was setting the bridge cross section. While the bridge alignment remained similar to the original structure, the new bridge cross section is slightly wider, therefore increasing the area of impact. The original bridge had two 11'–0" lanes, two 2'–0" shoulders, and an overall out-to-out dimension of 31'–8". The new bridge geometry consists of two 11'–0" lanes with two 5'–0" shoulders and sidewalks on each side of the bridge. The total bridge width varies from 47'–4" at the bridge approach spans, up to 56'–8" at the arch span. Much input was received from consulting parties and the public before the co-lead agencies agreed upon this final layout. Some groups were of the opinion to minimize the area of impact as much as feasible, while still meeting modern design codes. These groups were concerned with the historically sensitive setting of the structure, and wanted to minimize impact to undisturbed soils at the bridge touchdown points. Others wanted to seize the opportunity to widen the bridge to accommodate cyclists and pedestrians in the region. Additionally, the presence of historic sites at both ends of the bridge sparked interest to include sidewalks for pedestrians. The final resulting bridge section was a unique collaboration between all stakeholders.

7.2 *Engaging the US Coast Guard*

Following the January 6, 2010 meeting, the design team engaged the US Coast Guard. Because the new Lake Champlain Bridge was to traverse a navigable waterway, Coast Guard involvement was required. On January 12, 2010 NYSDOT and HNTB met with the Coast Guard to explain the design concept and verify Coast Guard requirements for navigational clearance. From these discussions a 75 ft vertical and 300 ft lateral navigational clearance was established. The new bridge permit application process began immediately following the meeting. This direct contact with the Coast Guard greatly expedited the permit application process, and the local 'Notice to Mariners' was posted by mid-March. By engaging the Coast Guard early in the process, the new bridge permit was in place well before the project was awarded on May 27, 2010.

7.3 *End of preliminary design*

The mandatory 30-day review period required by the Section 106 process ended on January 11, 2010, and on January 15, 2010 New York and Vermont announced that the modified network tied arch bridge concept would be progressed into final design. The preliminary design phase officially completed at the end of January, once the new bridge Programmatic Agreement was finalized. In terms of the actual bridge design, however, January 15, 2010 marked the beginning of final design.

8 PROJECT PROCESS/SCHEDULE

8.1 *Schedule*

During the preliminary design phase, a strict schedule for design deliverables was established. The design contract was to progress as a design-bid-build contract with some modifications to allow for expedited project delivery. A design-build contract was considered but quickly dismissed as New York State would require legislative action to permit design-build.

Considering the HNTB design team had an existing contract with NYSDOT for project scoping, NYSDOT took advantage of that existing contract to advance final design. The HNTB design team working on an accelerated schedule delivered bidding documents in six weeks and 95% contract plans on April 1, 2010, just 10 weeks after initiating final design. On April 15, 2010 bids were received and publically opened. All things considered, the expedited design-bid-build process progressed faster than a conventional design-build.

Unlike the bridge closure, bridge demolition, and installing temporary ferries, the new bridge design phase did not receive emergency relief from FHWA. With demolition of the existing bridge and activation of a temporary ferry service, public safety was restored and the severity of bridge closure was significantly reduced. Therefore, the new bridge design project was required to follow typical design project requirements, including the previously discussed NEPA environmental review process, all permitting required for a new bridge, and the formal NYSDOT bidding process. In an effort to expedite this process while still following FHWA and NYSDOT guidelines, the FHWA arranged for a federal regulatory agency summit in January 2010. Federal agencies that would have a role in the project participated, and all agreed on permitting requirements, process, and project timeline. Collaboration within the agencies carried through the permitting phase, including approvals needed from the US Coast Guard, the US Army Corps of Engineers, endangered species consultation, various agencies for clean water act provisions, as well as coordination with the St. Regis Mohawk Tribe. The collaborative efforts by FHWA, NYSDOT, VAOT, and their respective agencies were critical in acquiring all required permits and documentation to progress bridge construction.

Establishing a detailed schedule of deliverables early in the design process greatly contributed to the success of the Lake Champlain Bridge project. In an unprecedented move, the co-lead agencies agreed to make the 75% contract documents available for bid to interested contractors. Four weeks after 75% delivery, the HNTB design team delivered a major addendum that contained 95% contract documents for the contractors to finalize their bids. Details not critical to bid, including camber and haunch tables, bar lists, and load rating tables, would be delivered to the winning contractor at contract award. The proposed schedule, although expedited, still followed required time regulations per NYSDOT's standard bid process, and maximized the amount of time that contractors had to prepare their bids.

Once finalized, all involved parties were made aware of the design schedule, and that there could be no modification thereto. The scheduled deliverables included:

February 1, 2010	30% contract drawings available to contractors
March 1, 2010	75% "unofficial" contract documents available on NYSDOT website
March 17, 2010	75% contract documents available as official bid documents
March 29, 2010	95% "unofficial" contract documents available on NYSDOT website
April 1, 2010	95% contract documents issued by amendment

8.2 *Contractor involvement during design*

A primary strategy in the expedited design-bid-build process was to deliver as much information to interested contractors as early as possible. Because the bidding process was to be expedited in addition to the final design, it was important to deliver as much information to potential bidders as possible; not only for contractors to understand the type of construction and intended construction sequence, but also for contractors to begin reaching out to subcontractors and fabricators. Additionally, the environmental and historical sensitivity of the project site required strategic planning by the contractors in site access and construction sequencing. Included in NYSDOT contracts are Special Notes that offer additional project-specific information and clarification to the contractor. Special Notes in the Lake Champlain Bridge contract included information regarding issues such as construction access at the site, a full copy of the Section 106 Programmatic Agreement, an explanation of the schedule of deliverables under the modified design-bid-build contract, and also structural clarifications such as guidelines for mass concrete placement, metalizing, and arch erection. Additionally, Special Notes were included to clarify permitting and environmental monitoring required during construction. Vibration monitoring was also required for the historic structures adjacent to the construction site.

Contractor participation and input was a critical component during the design process, for not only was the design expedited, but the construction schedule was accelerated as well. On February 1, 2010, roughly two weeks following the start of final design, a contractor review meeting was held at NYSDOT offices. Interested contractors and subcontractors were invited to attend a presentation by the design team. A question and answer session followed the presentation in an attempt to have contractors provide feedback to the design team. A set of 30% plans was distributed to all attendees for additional information. Following distribution of the plans, an email address was made available to interested bidders on NYSDOT's website to address contractor questions and concerns. The website was continually monitored and updated with additional information for potential bidders. A webinar of the contractor review meeting was also made available to interested contractors unable to attend the meeting at NYSDOT's offices.

A pre-bid meeting was held on March 29, 2010 to provide additional information to interested contractors. Similar to the February 1, 2010 meeting, members of the design team presented the advanced bridge design, specifically pointing out changes or additions to the 75% bidding documents. Contractors also had the opportunity to ask the design team questions for clarification regarding the bid documents and constructability issues. Bids were then received on April 15, 2010.

9 QUALITY CONTROL

9.1 *Constant communication*

A critical component to the successful design-bid-build process was the involvement of all stakeholders and their ability to meet the demanding deliverable deadlines. In order to streamline the design process, conference calls were held twice a week between NYSDOT, VAOT, and the design team. Design questions, issues, and review comments were addressed. All parties maintained awareness of recent changes, design decisions, and any other developments that could alter the design documents.

In addition to collaboration with the design team, effective and constant communication between the involved agencies was imperative to the success of the expedited bridge design process. In order to keep lines of communication open, NYSDOT, VAOT, and FHWA participated in inter-agency conference calls every Monday, Wednesday, and Friday during the design phase.

9.2 *Review of drawings and specifications*

In addition to effective communication, review of drawings and specifications was a crucial component to quality control. Separate review teams were established within NYSDOT for the approach roadway and structural drawings. The members of these review teams at NYSDOT, VAOT, and FHWA understood that HNTB would submit updated or completed drawings by noon each Monday and Thursday and the review teams were given 24 hours to provide comments to NYSDOT's Deputy Project Manager. The Deputy Project Manager would then compile comments and deliver them to HNTB by close of business that same day. This process permitted all drawings in the contract plan set to be reviewed by each of the co-lead agencies prior to PS&E delivery. Any comments requiring further discussion would be addressed on the semiweekly conference calls mentioned above.

9.3 *Working sessions with the design team*

Major design and constructability issues were addressed in several all-day working sessions held in HNTB's office in New York City. The key players in design from NYSDOT, VAOT, and FHWA came to HNTB's office for three consecutive days in January, and again for two days in February prior to the 75% PS&E package submittal. Additionally the key players met once more for two days in March to address comments on the 75% submittal. These all-day working sessions allowed the co-lead agencies to effectively address any design issues directly with the design team. The direct interaction between the design engineers and the co-lead agencies during these working sessions was essential to avoid misunderstanding the comments received. The design team was able to effectively communicate thoughts and ideas through sketches, props, and other visual methods that typical conference calls or periodic meetings cannot employ effectively. These working sessions allowed for efficient and effective decision-making that greatly streamlined the overall design process.

Also included in the first three day working session in January was a value engineering review by an outside consultant. HNTB presented the 30% plans on January 11, 2010 and the value engineering team had 48 hours to complete their review of the documents and present their findings to NYSDOT, VAOT, and FHWA representatives on January 13, 2010.

Part of all three working sessions at HNTB's office was a constructability review. A construction representative from NYSDOT attended each of the working sessions to provide input and help modify design elements where appropriate to expedite construction. Minimizing construction duration was a driving factor in the design of the replacement bridge. Furthermore, the construction representative would serve as an important link within NYSDOT as the project progressed from the design phase to construction.

10 THE NEW LAKE CHAMPLAIN BRIDGE DESIGN

10.1 *General design parameters*

Throughout January, the design team developed the Final Design Report for the new bridge, summarizing the bridge selection process, as well as general design parameters for the new bridge. Within days of submitting the report to the co-lead agencies, FHWA granted design approval on February 5, 2010.

Early in the preliminary design phase, the new bridge geometry was set. The new bridge would be composed of eight spans, with one network tied arch signature span over the navigation channel. The arch span would be supported by triangular rigid 'delta' frames cantilevering from the multi-steel approach span girders. The total bridge length was set at 2200 ft with seven approach spans each measuring approximately 250 ft, and a single 402 ft tied arch span (Figure 11). Two traffic lanes were set at 11 ft each to match the connecting Vermont Route 17 pavement configuration. In an effort to accommodate cyclists, 5 ft wide shoulders flank the two lanes. Additionally, the public expressed a need to accommodate wide farming equipment that frequently traverses the lake, and the combination of 11 ft wide lanes and 5 ft wide shoulders satisfies this requirement. Since the arch structure extends above the roadway with lateral cross bracing, vertical clearance was also of concern. A minimum vertical clearance of 16'–6" is provided at the bridge curb with vertical clearance increasing toward the center of the roadway. Figure 12 provides a rendering of the arch span cross section.

An additional feature added as a result of public input is the sidewalks on both sides of the bridge. The sidewalks range in width from 5 ft at the approach spans to 9'–8" at the arch span. Navigational clearance was also discussed at the public meetings. Information gathered from local boaters and marinas, and approved by the Coast Guard, led to a 75 ft high navigational clearance over a 300 ft wide channel. This satisfied local boating needs while also complying with the maximum desired grade of 5% on the structure.

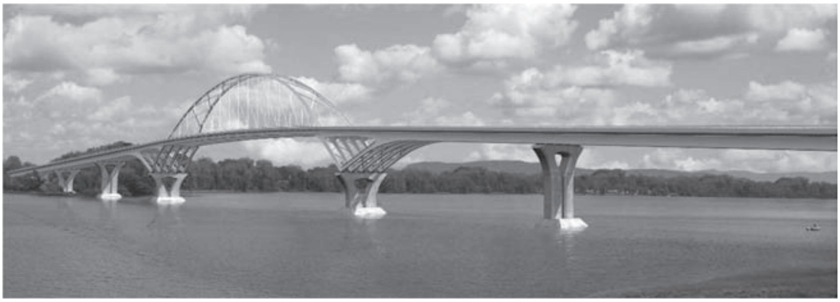

Figure 11. Rendering of the proposed replacement bridge.

Figure 12. Rendering of the arch span cross section.

10.2 *Bridge substructure*

In an effort to reduce permitting and environmental documentation, consideration was given to proposed pier locations that minimized impact to undisturbed soils. As a result, the new pier locations were placed in close proximity to the original bridge piers while still avoiding existing caissons left from the original structure. Drilled shaft foundations were used at Piers 1 through 6 to minimize excavations in the lake mud. Subsurface investigations confirmed that the lake bottom soil was of poor quality for foundation support, having a consistency similar to that of toothpaste. The drilled shaft foundations are anchored into bedrock with the longest shafts at Piers 4 and 5, measuring roughly 110 ft in length. The remaining piers had shafts ranging from 35 ft to approximately 80 ft in length. Two of the seven piers are supported by four 6 ft diameter shafts, and four of the seven piers are supported by six 6 ft diameter shafts. The pier adjacent to the Vermont abutment was designed as a spread footing on rock. Both proposed abutment locations are within a few feet from historic soils, thus minimizing soil disturbance was critical. As a result, both the Vermont and New York abutments were designed with micropile-supported foundations. Additionally, remains of Fort St. Frederic and Fort Crown Point lie adjacent to the New York abutment location and the Vermont abutment is directly adjacent to the site of an 18th Century French Fort and the Chimney Point Historic Site. Concerns with excessive vibration eliminated several foundation types from consideration. Use of micropiles reduced the risks associated with both disturbing virgin soils and producing potentially damaging levels of vibration. As an additional precaution, vibration monitoring of historic structures was included in the bid package.

The new bridge piers are reinforced, cast-in-place concrete Y-shaped piers with triangular openings carved out of the upper portion of each pier (Figure 13). Tapered, rectangular footings with rounded corners support the piers. The approach span piers have identical pier columns and pier caps that sit on top of a varying height pier stem. The same forms can be used to construct the top portion of the piers with the opening. The two piers flanking the arch span are similar, but slightly wider to support the rigid 'delta' frames. The pier shape was primarily chosen for aesthetics to complement the superstructure they support. In order to reduce the potential for ice floe damage, granite facing will be installed on the new pier pile caps. Additionally, the faces of the pier footings are inclined to aid in ice breaking.

10.3 *Bridge superstructure: Approach spans*

In an effort to lengthen the serviceable life of the steel superstructure, a double corrosion protection method was included in the design. All of the superstructure steel is designated as weathering steel to increase durability. Additionally, most steel members will have a metalized coating comprised of 85% zinc and 15% aluminum. Members not metalized will be galvanized. The design life of the superstructure steelwork is 100 years.

The approach spans are standard continuous long-span steel plate girders with a cast-in-place reinforced concrete deck. Epoxy-coated reinforcement is used in the deck which has an estimated design life of 50 years. There are five girders each measuring 8 ft deep, and all five girders increase in depth at the transition to the rigid 'delta' frames. The rigid frame members extend roughly 180 ft between the approach girders and the supported arch span.

Because the arch span is 9'–4" wider than the approach spans, a transition is needed in the bridge width. As a result, a flare is introduced into the girder spacing approximately 250 feet before reaching the arch on both the New York and Vermont approaches. The outer two girders on each side of the bridge are angled outward at a field splice, and a gradual increase in bridge width is achieved up to the cross beam located at the end of the rigid frame members. The cross beam is a closed box member that spans transversely between each of the rigid frame members, and upon which the arch span rests in its final position.

Figure 13. Rendering of new piers with granite facing at the footings.

10.4 *Bridge superstructure: Arch span*

The arch span is supported by two open arch ribs situated between the roadway and sidewalks, and inclined toward the roadway center at 10°—commonly known as a 'basket handle' arch. (Figure 14). In an effort to expedite arch detailing and fabrication, the arch is circular in shape. This allows for repetition in the arch rib segments. The two arch ribs are connected with I-shaped beams as lateral bracing. The lateral bracing members are detailed as straight pieces with bent gusset plates to compensate for the arch curve and inward inclination of the two arch ribs. Horizontal arch thrust is resisted by the longitudinal tension tie girders that connect the ends of each arch rib at the knuckle joints. The tie girder is a closed box, measuring roughly 4 feet high by 3 feet wide, with webs angled at 10° to follow the incline of the arch ribs (Figure 15). Both the arch and the tie girder are Grade 50 W, metalized steel.

Cable hangers that support the arch span are made of a single seven-wire strand. Each cable hanger crosses at least two other cable hangers, which is the defining characteristic of a network tied arch bridge. Cable hangers are greased and sheathed for advanced corrosion protection. For greater redundancy, it is possible to replace any cable without requiring temporary supports; a redundant feature of significant importance to the co-lead agencies.

Spanning between the two tie girders are steel floorbeam members spaced 12 ft on-center. Arch span sidewalks are supported on cantilever brackets connected to the tie girder. The arch span deck is composed of longitudinally post-tensioned precast concrete panels supported by the floorbeams and the sidewalk brackets. Galvanized steel reinforcement is included in the precast concrete panels to aid in extending the design life of the deck to 75 years. Trans-

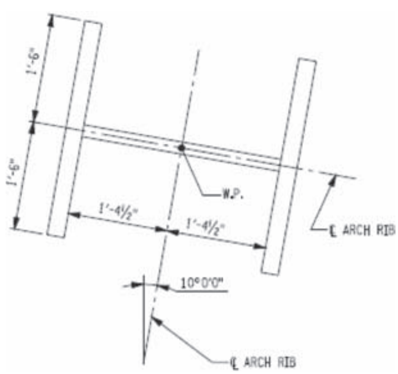

Figure 14. Arch rib section.

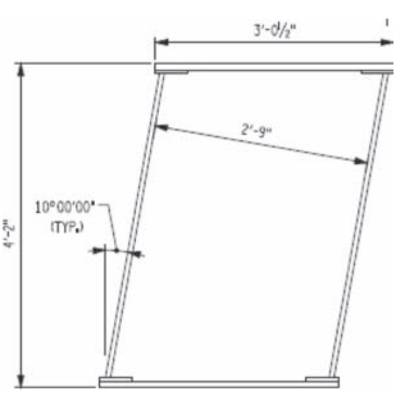

Figure 15. Tie girder section.

verse closure pours are located above floorbeam and sidewalk bracket flanges. There are only two longitudinal closure pours, each running along the top flange of the tie girders, which separate the deck precast panels from the sidewalk panels. Concrete blisters are included in the longitudinal closure pours that protect the bottom of the cable hangers. Both the approach span cast-in-place deck and the arch span precast concrete deck are designed to be replaceable. The new bridge does not include a wearing surface, but measures were taken in the design to allow for future wearing surface installation.

11 CONSTRUCTABILITY IN THE DESIGN PROCESS

11.1 *Incentive/Disincentive clause*

In addition to the expedited design schedule, the co-lead agencies agreed to an expedited construction schedule of roughly 16 months. The temporary ferry operation is costly and the co-lead agencies agreed to include an Incentive/Disincentive (I/D) Clause in the contract documents based on the ferries operating costs. Upon award of the contract, the contractor would be given 500 calendar days to reach substantial completion and have the bridge open to traffic. The incentives were set at $30,000 per day, with a maximum of 50 days ($1.5 million). Disincentives were also set at $30,000 per day for work extending beyond the deadline. There was no cap on disincentives included in the contract documents.

11.2 *Constructability in design*

In an effort to accelerate construction of the new Lake Champlain Bridge, constructability was a driving factor in the design. Perhaps the most important design decision was to have the arch span steel constructed off-site, floated into position and erected onto the rigid ‘delta’ frames using a heavy lift (Figure 16). By erecting off-site, the Contractor is able to construct the arch span and approach spans simultaneously, greatly reducing the overall construction schedule. Once the approaches are completed, the arch span can be floated in by barge and lifted into place using strand jacks.

Originally, the rigid ‘delta’ frames were included in the design to placate the public’s desire for a more aesthetically pleasing structure. Once the decision to include the rigid frames was made, however, the design team utilized them to aid in the proposed construction sequence. In order to lift the arch span, the arch rib and tie girder cannot align with the rigid frame girders. By splaying the outer two approach girders/rigid frames, the arch can be lifted as shown in Figure 16. In general the arch will be lifted above the rigid frame members, the end diaphragms connecting the rigid frames installed, and the arch span lowered onto bearings positioned atop the end diaphragms at the four corners of the arch span. This heavy lift operation is possible due to the lightweight nature of the network tied arch.

Once the arch is in place, the precast deck panels can be installed. It was decided to use precast panels in order to reduce the weight of the lift and to eliminate the need for extensive formwork and cure time associated with a conventional cast-in-place deck. In addition, prior to pouring the closure pours the weight of the precast panels allows the arch to achieve its deflected shape under dead load without inducing tensile stresses into the deck. Both the heavy lift operation and precast concrete deck installation require little to no interruption for vessels navigating the channel. A 200 ft navigation channel is to remain open for the duration of construction, with a possible one day temporary closure for the heavy lift operation.

In addition to design details and methodologies, other means for expediting construction were written into the bid documents. Explicit turnaround times for shop drawing reviews and answering contractor’s Requests for Information (RFIs) are to be adhered to by the design team and the co-lead agencies. Additionally, engineers from the design team are on-site to facilitate improved coordination and communication between NYSDOT’s Engineer-In-Charge (EIC), the Contractor, and the design team.

Figure 16. Rendering of the arch span heavy lift.

12 CONCLUSIONS

The original Lake Champlain Bridge was constructed in 1929. An historic and iconic structure, the bridge was closed and demolished in 2009 due to severe deterioration of the unreinforced concrete substructure. Following bridge closure and demolition, a signature replacement structure was designed within an accelerated time frame that was unprecedented for a standard design-bid-build contract. Together with New York, Vermont, and the FHWA; the HNTB design team developed a plan for the demolition of the original truss bridge, the design of a new signature crossing and the construction of that bridge within twenty months.

The success of this project is that three months after final design was initiated, bids were opened for the new Lake Champlain Bridge. Due to the expedited design-bid-build process, the design was completed on a compressed schedule with traditional linear functions done concurrently with bid advertisement, contract packaging and permitting. The teamwork between the two states and numerous local and federal agencies allowed for a collaborative and cooperative process. Open and constant lines of communication, as well as a dedicated design team, were critical components to the success of the project. This Lake Champlain Bridge project serves as a model for success in communication and cooperation amongst various agencies and interest groups, resulting in significant time and cost savings for the public.

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Chapter 7

A signature footbridge for Xinjin, China

Q. Ye, J. Zhang, C. Clark & G. Fanjiang
Weidlinger Associates, Inc., New York, NY, USA

ABSTRACT: A signature footbridge has been designed for Xinjin, China. The pedestrian bridge has 5 spans with a total length of 229.0 meters, and features two intertwining steel-box girders in a 3-D space, creating a symbol of the five rivers that converge at the city. There were many design challenges: 1. finalizing a bridge geometry that satisfied the architects, owners, engineers and review committees; 2. detailing the critical connections between the columns and box girders 3. providing special design consideration for pedestrian-induced vibrations. The design team overcame these difficulties, came up creative solutions, and completed the preliminary design in 3 months.

1 INTRODUCTION

Xinjin is a historical town located 12 km south of Chengdu, China. It has attracted many residents and visitors over the years, and is geographically centered on the convergence of five rivers. Over the years poets have exalted its natural beauty. To elevate the image of Xinjin and attract business and investment, the City of Xinjin decided to construct a signature pedestrian bridge, the Nanhe Landscape Bridge, in the center of its city.

An international competition was held in April, 2010, and five teams submitted their design concepts. After presentations of concept, the selection committee chose the 3D double-helix bridge developed jointly by WXY architecture of New York City and Weidlinger Associates as the winning concept. The five piers of the selected concept symbolize the five rivers that have nurtured the city for centuries, and the intertwining curves in the plan represent their convergence. The city of Xinjin awarded the preliminary and final design to our team, together with our local partner, Southwest Municipal Engineering Design and Research Institute. The schedule for finalizing the concept and preliminary design was extremely tight, only 3 months.

The five spans of the bridge have a total length of 229.0 m (24.5 m + 4 × 45.0 m + 24.5 m), as shown in Figure 1. There are five concrete piers and each pier supports four slender steel columns. Each pier foundation consists of four 1.8 m-diameter concrete caissons. The superstructure consists of two 6.5 m-wide steel boxes (Bridges 1 and 2), and in plan their maximum out-to-out distance is 35.0 m.

Even though many aspects of the bridge design are worth discussion, this paper focuses on the following three subjects:

1. Bridge geometry;
2. Column-to-box connections;
3. Mitigation of pedestrian-induced vibration.

There have been many changes during the course of the design. The details and analysis results presented in this paper are related to the bridge geometry for the preliminary design.

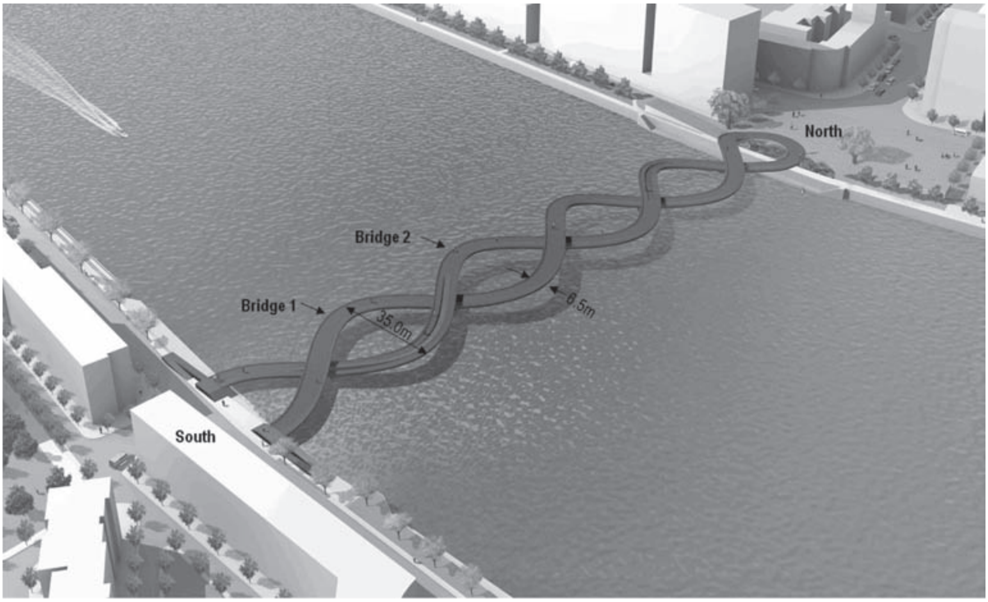


Figure 1. Bird's eye view of bridge.

2 BRIDGE GEOMETRY AND CONFIGURATION

2.1 *Elevation and bird's eye view*

Finalizing the elevation turned out to be the most difficult task of the project. Figure 2 shows the change of elevation at different design phases. The factors that affected the vertical profile are:

1. Aesthetics and bridge access;
2. Maximum slope;
3. Navigation clearances;
4. Elevation of the design flood.

2.1.1 *Aesthetics and bridge access*

In elevation, the winning concept (Figure 2a) is visually pleasing due to its elegant vertical curves and slender structural elements. Also, the absence of ramps on both ends of the bridge not only makes the bridge more appealing, but also provides a smooth transition between the bridge and the plaza on the North end and the street on the South end.

Even though it was recognized that, in order to satisfy code requirements and provide sound engineering design, it would be necessary to modify the geometry, the aesthetics of the structure have always been an important consideration in the design process.

2.1.2 *Maximum slope*

In accordance with the Chinese codes for footbridges, the maximum slope is 1: 4 for ramps; 1:2 for stairs; and normally 1:12 for wheelchair access, but may be 1:10 under very difficult conditions. The owner of the bridge initially directed the design team to provide wheelchair access on this bridge, but decided against it after realizing the extreme difficulty of implementing this requirement on a bridge with such complex curves without significantly altering the vertical profile. The design effort to accommodate wheelchair ramps is elaborated on Section 2.3.

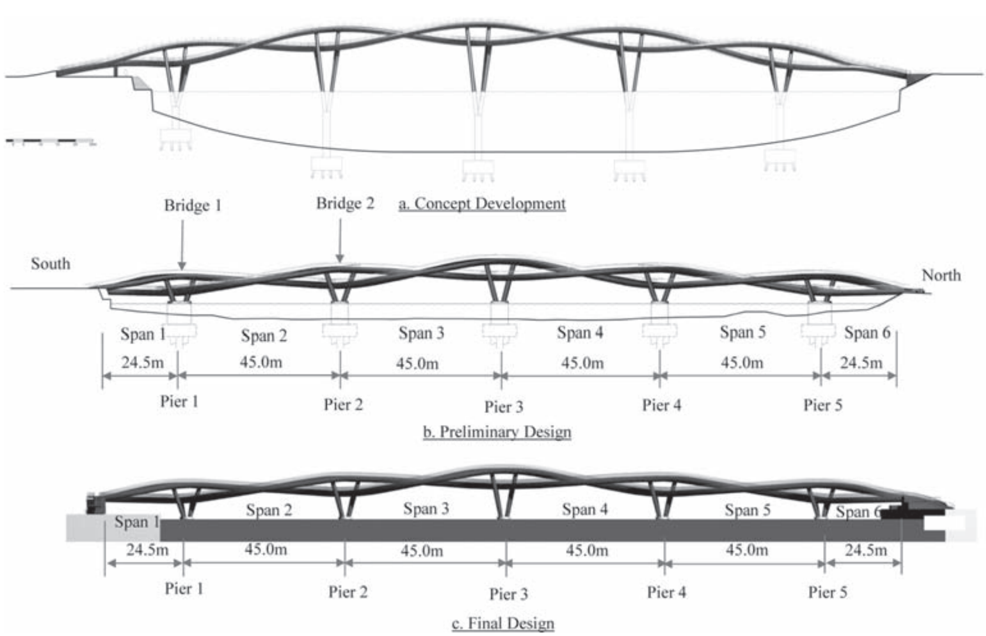


Figure 2. Bridge elevation.

Without the wheelchair ramps, the maximum slope is approximately 1:5 for Bridge 1 and 1:7 for Bridge 2—a rather steep walk. To make the walk more comfortable and safe, ramp steps are introduced at locations where the slope exceeds 1:8.

2.1.3 *Navigational clearance*

The river that the bridge crosses, the Nan River, is not a navigational channel, and there are dams downstream to regulate the water depth. However, according to the city plan, there will be recreational boats passing under the bridge, and the owner set the navigational clearance at 8 meters from the mean high water, it was subsequently reduced to 6 meters since large boats are not expected to cross the bridge. The two middle spans are designated as navigational channels and satisfy this requirement.

2.1.4 *Elevation of design flood*

The Nanhe Landscape Bridge is a footbridge and a 50-year design flood would suffice. However, since this bridge will be a signature bridge, it was decided to use the 100-year flood for design. Based on the Chinese code, the lowest point of the box girders in all the river spans should be at least 0.5 meters higher than the elevation of 100-year flood. As a result, the box girders would be very high in elevation and long ramps would be required to accommodate the height. This would conflict with the easy-access objective of the architectural design, therefore it was decided to keep the bridge low and design the two end spans to be submerged when the 100-year flood occurs (Figure 2b). However, after the preliminary design, a severe flood occurred at the bridge site, and the owner requested that the bridge elevation be raised to meet at least the 50-year flood design requirement (Figure 2c), which is about 2.8 m higher than that of the preliminary design. The final design was based on the raised elevation.

2.2 *Cross section and height of box girder*

Each box girder has three cells for resisting torsional moments, as shown in Figure 3. The height of box girders also plays a role in the bridge geometry. The clearance between two

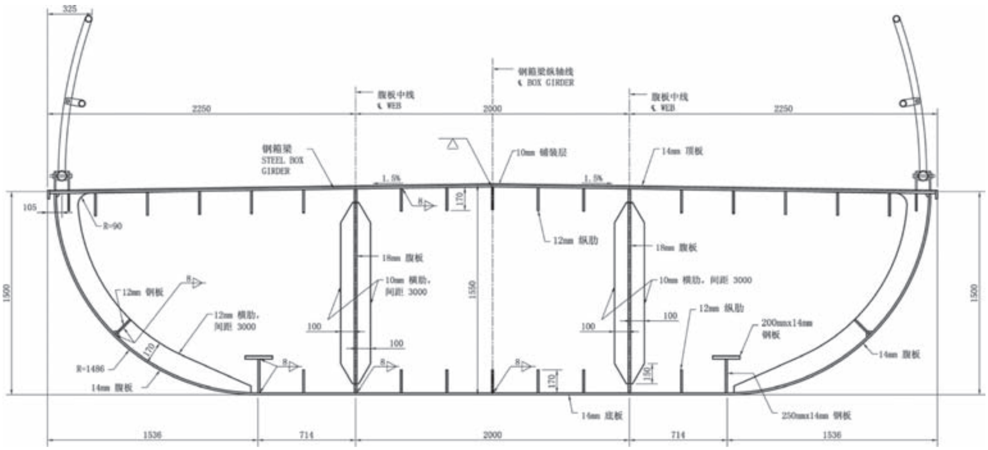


Figure 3. Cross section of steel box girder.

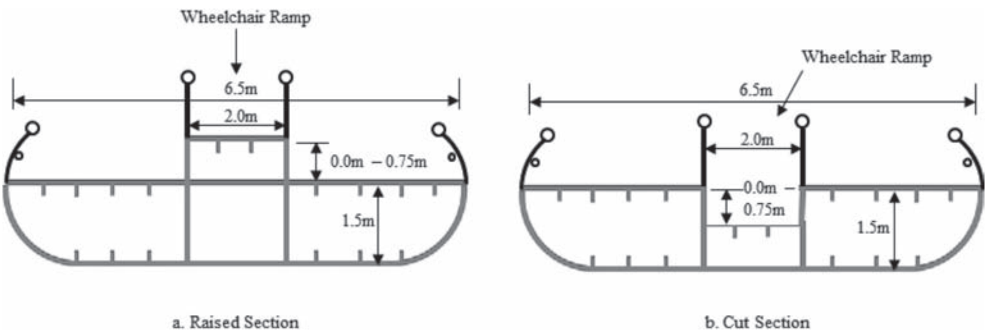


Figure 4. Wheelchair ramp and ramp steps.

intersecting box girders shall be at least 2.5 meters according to code and architecturally it is desirable to increase this clearance so that the openings will be larger and assume an aesthetically pleasing lens shape. Shallower box girders allow for larger openings, giving the lens a more appealing aspect ratio.

However, a shallower section is less efficient than a deeper section, and therefore is heavier and more costly than a deeper one. In addition, for construction and future maintenance, deeper box girders offer easier access. After a few iterations, the height of the box girder was chosen to be 1.50 m at the outside edges; and the thickness of top, bottom and curved web plates is 12 mm; the thickness of two web plates in the middle is 20 mm. The clearance between two boxes is 3.0 m at Pier 3, and 2.75 m at other piers.

2.3 *Wheelchair ramps*

Although the owner eventually decided not to provide wheelchair ramps on the bridge, a great deal of engineering effort was put into a design that would accommodate ramps and is worth discussing here.

To keep the maximum slope under 1:12 and maintain the architectural vertical profile of the bridge at the same time was very difficult. The solution developed by our team was to raise the portion of box girder designated for the wheelchair ramp at the low points of the bridge (Figure 4a) and cut it at the high points (Figure 4b), therefore limiting the maximum slope to 1:10.

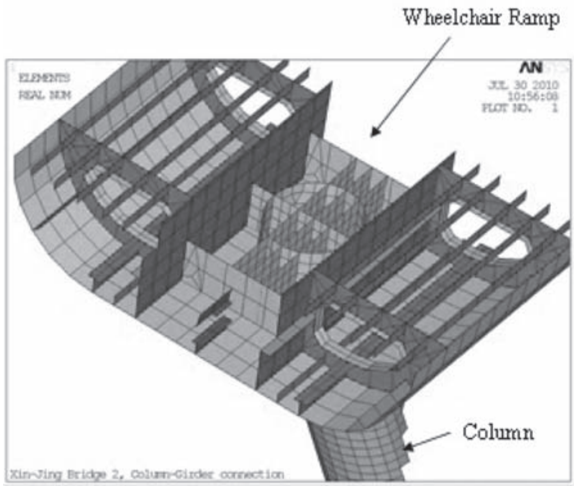


Figure 5. Internal details of model at intersection of cut section and wheelchair ramp.

Extreme effort was put into the modeling, analysis, design and detailing of these cut and raised box sections. Creating a 3D model to analyze the stresses at these sections was very difficult and time consuming. Figure 5 shows the intricacies of the finite element model details at the intersection of cut section and the column. After seeing the complex design and realizing the challenges of providing wheelchair access on a bridge of 3D curved shapes, the review committee and owner decided to waive the requirement for this bridge and use another bridge downstream for the wheelchair access instead.

3 COLUMN-TO-BOX GIRDER CONNECTIONS

At each pier, there are four steel columns, which are hollow and 1.3 meters in diameter. The typical shell thickness is 30 mm. There are large forces and moments that need to be transmitted between the box girders and the columns. Therefore, the connections at these locations must be carefully designed.

3.1 *Column-to-box connections—Piers 2, 3 and 4*

Figure 6 shows the typical connection details between columns and box girders. In this detail, the column intersects with the bottom flange of the box girder, where it is butt welded to an internal column segment, which has an oval-shaped cross section that matches the shape of the column section at the interface. A 24 mm thick, vertical web plate in the center of the box girder runs longitudinally and is connected to two transverse diaphragms located 1.5 m away, one on each side of the center diaphragm.

3.2 *Column-to-box connections—Pier 1 and 5*

The connections on Piers 1 and 5 are different from those on other piers since the thermal expansion or contraction of the bridge would cause excessive forces in these columns if they were rigidly connected to the box girders. Therefore the boxes shall be allowed to expand or contract freely in the longitudinal direction on these piers.

In addition, when the 100-year design flood is applied transverse stream pressure and vertical buoyancy forces act on the box girders in the end spans. To resist these loads, transverse

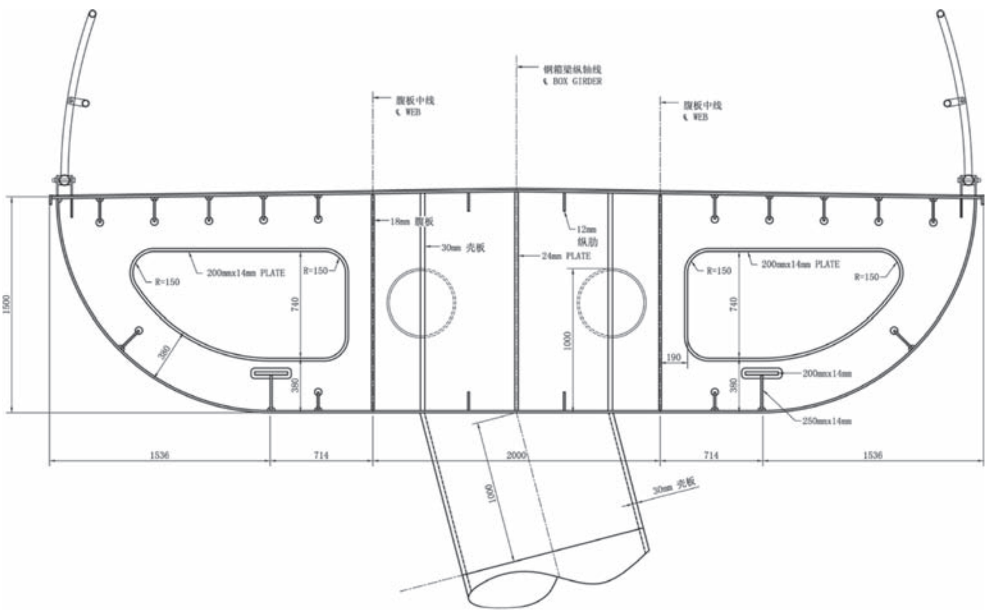


Figure 6. Typical column-to-box connection.

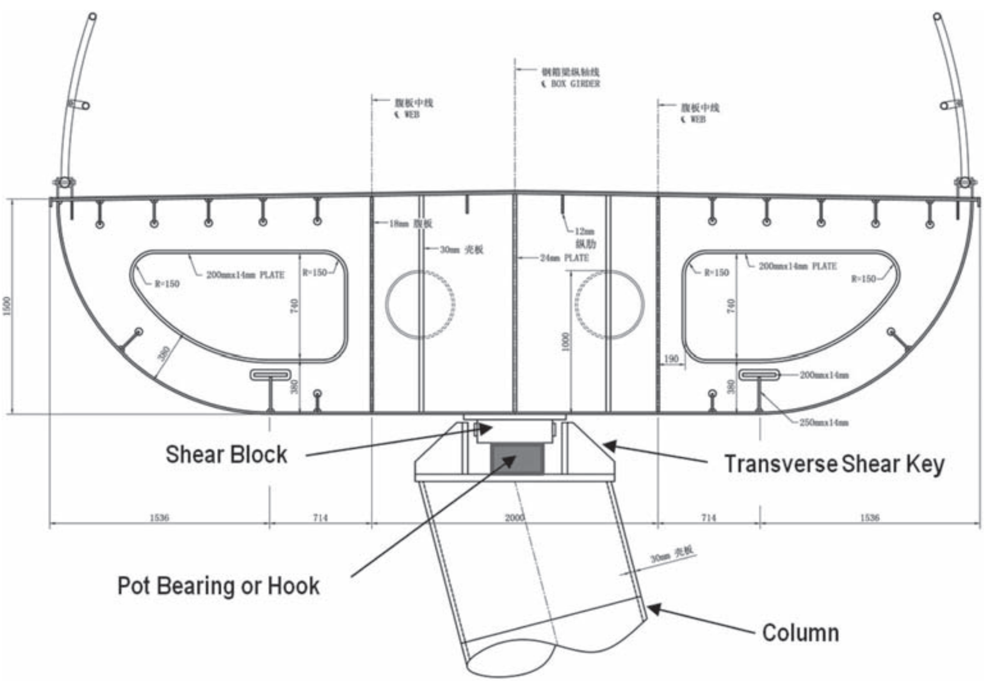


Figure 7. Typical column-to-box connection.

shear keys and pot bearing (or hooks) are provided on Piers 1 and 5, as shown in Figure 7. On top of one of the short columns supporting Bridge 2 at Piers 1 or 5, a hook was provided to resist uplift forces during a flood event; three other columns on the same pier have pot bearings that would resist only compression forces.

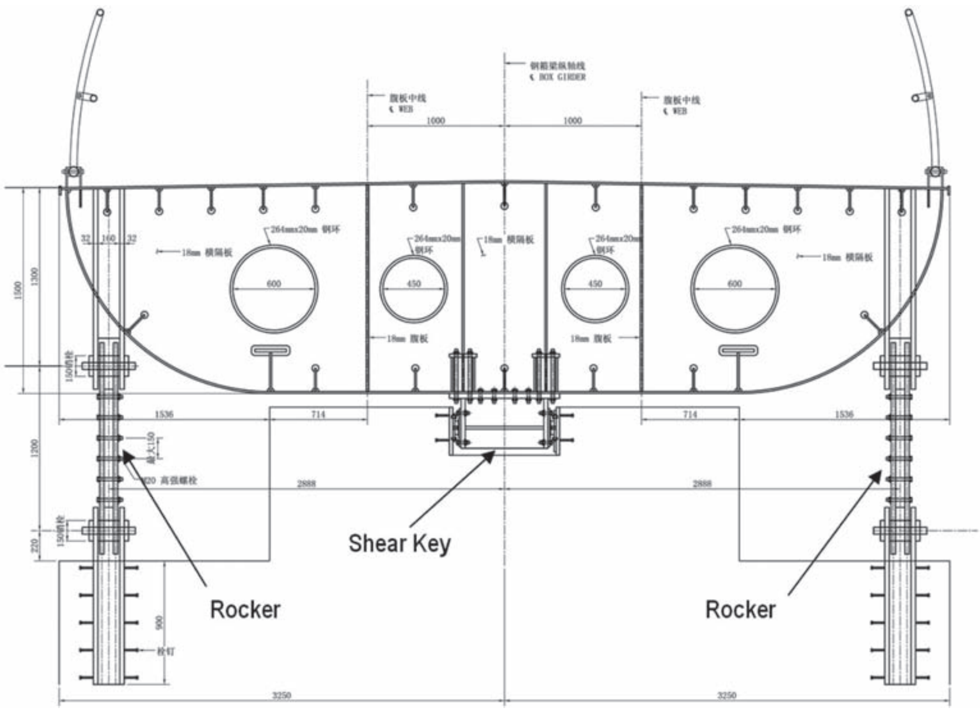


Figure 8. Supports of box girder at abutments.

3.3 *Supports at abutments*

The supports for box girders at abutments are shown in Figure 8. There are two rockers at each side, providing vertical and torsional resistance. The shear key at the center of the pier prevents transverse movements of the girder. In the longitudinal direction, there are no restrains and the box girder can move freely. The inventive detail was necessitated by the torsion induced by the curving of the box girder in plan.

4 FINITE ELEMENT MODELLING AND ANALYSIS

Two 3-dimensional (3D) models have been created for analyzing the bridge: one frame model in SAP2000 (Computers and Structures, Inc.) for global static and dynamic analysis; one finite element model for Bridge 1 in ANSYS (Ansys, Inc.) for local stress analysis.

4.1 *3D Frame model in SAP2000*

Figure 9 shows the 3D frame model in SAP2000. All the structural elements are modeled by beam elements. The caissons are also included in the model, with soil springs applied along their height. The shear keys, pot bearings and hooks at Piers 1 and 5 are modeled with non-linear spring elements.

Static analyses of the structure under dead, live, wind, buoyance, and thermal loads were performed, and the box girders, columns, and foundation were designed for many different combinations of these loads.

Dynamic analyses included earthquake and pedestrian-induced vibration analyses. The spectrum analysis method was used in the earthquake analysis. The analysis of

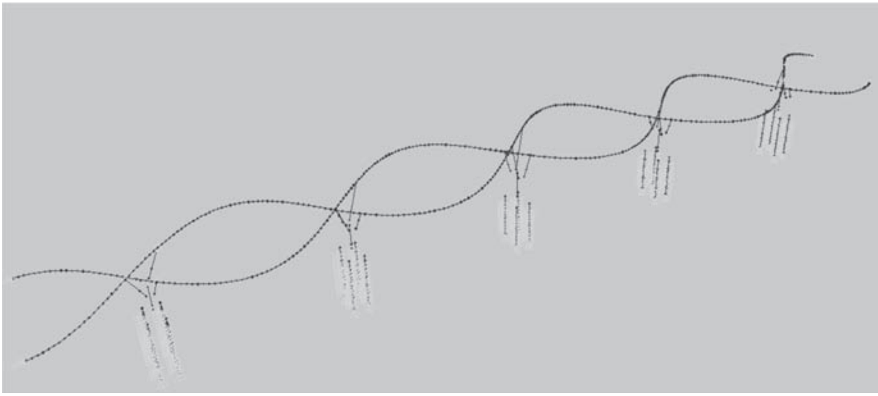
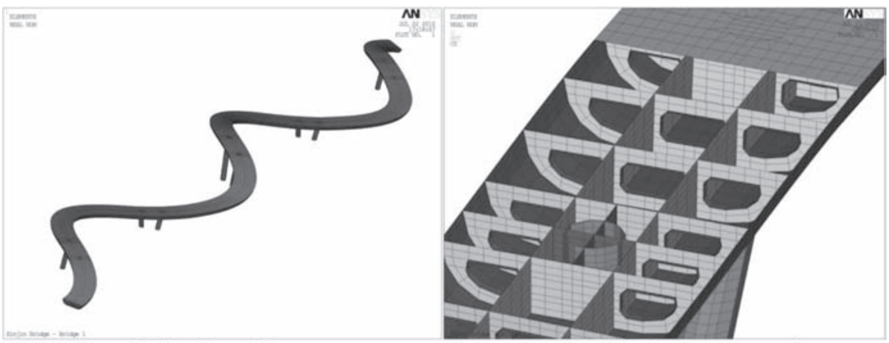


Figure 9. Global model in SAP2000.



a. Overview of bridge 1 model b. Details of model of column connections

Figure 10. Global model of bridge 1 in ANSYS.

pedestrian-induced vibrations used a steady-state resonant vibration analysis to find the peak acceleration of each mode under the design dynamic force.

4.2 *3D finite element model in ANSYS*

The main purpose of finite element analysis was to investigate the local stresses at the column-to-box connections. From the SAP2000 model it was observed that the two halves of bridge, Bridge 1 and Bridge 2, have essentially the same forces and moment diagrams, therefore it was decided to model only Bridge 1 in ANSYS, as shown in Figure 9. All the main elements of the box girder, such as top plate, bottom plate, diaphragms, web plate, etc., were included in the model as shell elements. Again, due to the complex geometry, tremendous amount of effort was devoted to the modeling.

The load case that creates the maximum vertical bending in the SAP2000 model was applied to the ANSYS model, and the maximum Von-Mises stress is 93.0 MPa, as shown in Figure 10, well below the allowable value of 200.0 MPa.

5 VIBRATION MITIGATION

Due to the large horizontal curve there were concerns about vertical vibrations caused by pedestrians, therefore a detailed vibration study was conducted. The SAP2000 model was used to perform a steady-state analysis using the dynamic forces calculated based on the guidelines of German code EN03–2007, as referenced in Chen & Hua (2009).

It was found that the first three modes, whose frequencies are around 2.0 Hz (Figure 12), are susceptible to pedestrian-induced vibrations, and the maximum acceleration could reach 15% g (Table 1), exceeding the upper-bound comfort limit, 10% g, recommended by many experts and guidelines, such as British Standard BD 49/93 and ISO 10137. Therefore, vibration mitigation measures were necessary.

The first option considered was to use toggle-braced dampers (TBD), a patented product by Taylor Devices, installed at the abutments (Figure 13). Due to the geometry of the toggle

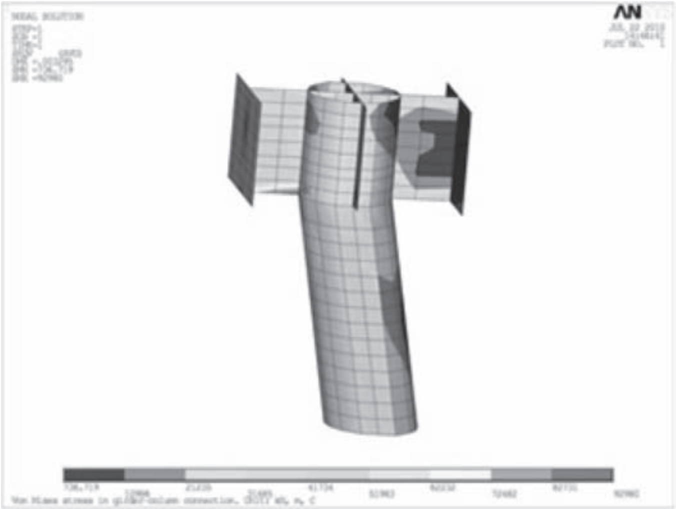


Figure 11. Von Mises stress at column-to-box connection (kPa).

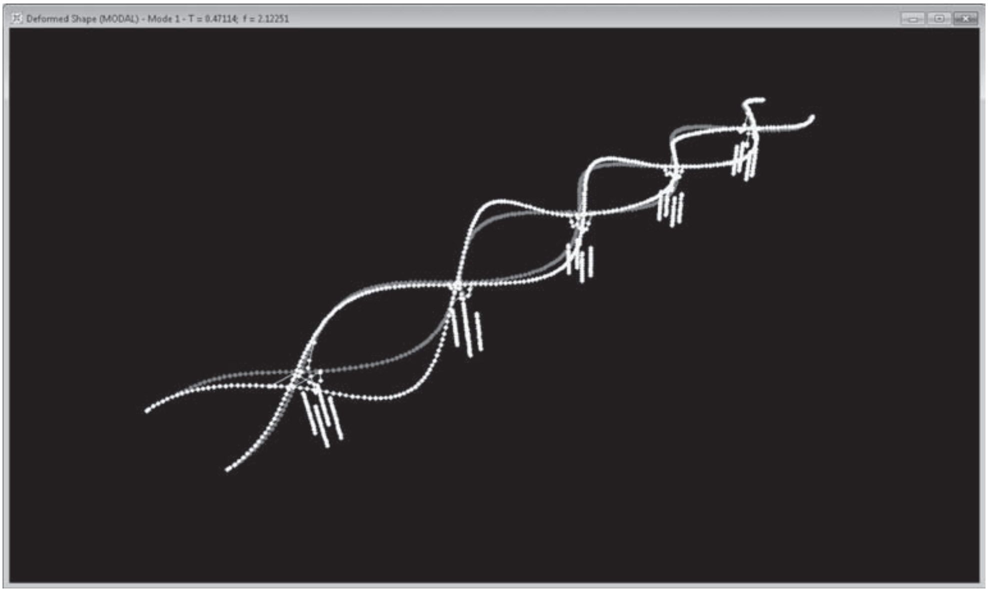


Figure 12. 1st vibration mode, f = 2.123 Hz.

Table 1. Vertical bridge vibrations without dampers.

			Entire bridge loaded without TMD		
Mode	f (Hz)	ξ_{ext}	Δ_{Rn} (mm)	a_n (% g)	a_n (m/s^2)
1	2.1225	0.40%	8.2	14.8	1.45
2	2.131	0.40%	8.2	15.0	1.47
3	2.1813	0.40%	5.1	9.8	9.8

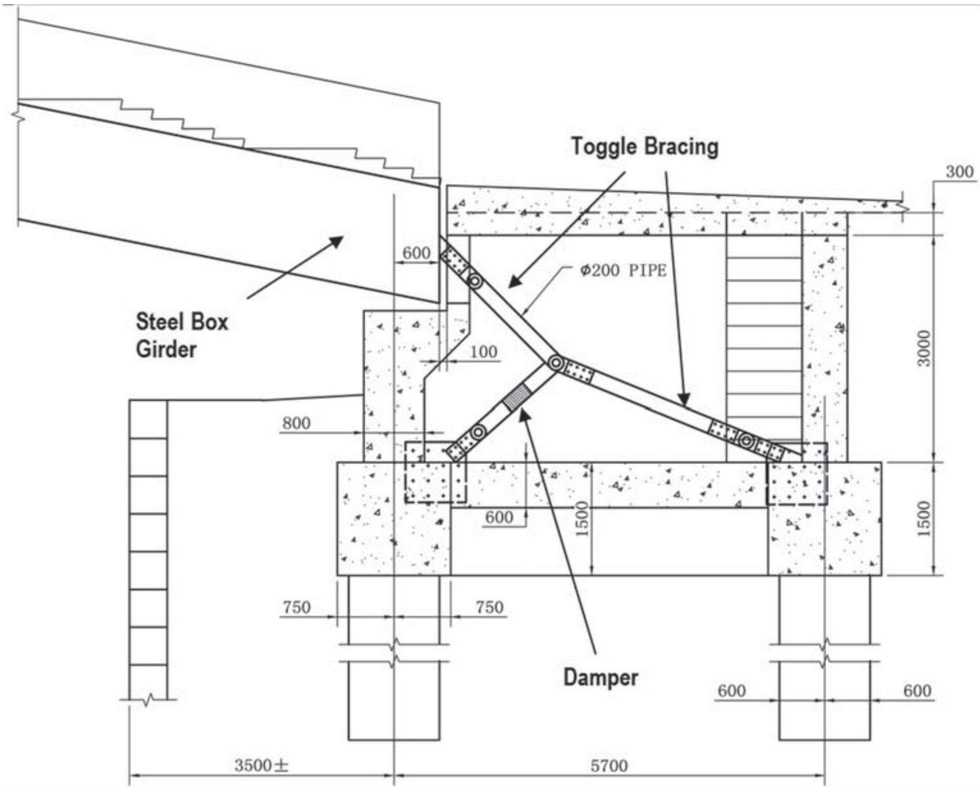


Figure 13. Toggle-braced dampers (TBD) at abutment.

bracing, the strokes in the dampers would be about 3 to 4 times the longitudinal movements at the end of the box girders. However, the increase in effective damping was determined to be only about 1.5% for this bridge due to the low magnitudes of the longitudinal displacements. In addition, only Spans 2 and 5 have relatively large reductions in vibrations since other spans are further away from the TBD.

In addition, since the design flood elevation is higher than the location of dampers, these dampers would be required to be waterproof and the only product available is the metal-bellow damper, which is also product by Taylor Devices. These dampers have great performance and long service life, but are more expensive than other types.

After study, it was decided to install one tuned mass damper (TMD) inside each box girder in the middle of each span, similar to the ones shown in Figure 14. The mass of each TMD equals to 1% of the total mass of one girder in each span, and the cost is lower than TBD. Steady-state analysis was performed again after the TMDs were included in the model,

Figure 14. Vertical TMD inside steel box girder.

Table 2. Vertical bridge vibrations with TMD.

			Entire bridge loaded, TMD = 1.0% bridge mass			
Mode	f (Hz)	ξ_{ext}	Δ_{Rn} (mm)	a_n (% g)	a_n (m/s^2)	Total damping, ξ_{tot}
1	2.1225	0.40%	1.1	2.0	0.19	3.1%
2	2.131	0.40%	0.9	1.6	0.16	3.8%
3	2.1813	0.40%	0.6	1.1	0.11	3.6%

and the peak acceleration was reduced to 2% g, as shown in Table 2, and the total effective damping (ξ_{tot}) is above 3%, based on an assumed 0.4% (ξ_{ext}) available structural damping.

6 CONSTRUCTION SEQUENCE

Construction sequence is extremely important to ensure the Nanhe Landscape Bridge is built per design specifications. The construction of the bridge can be executed through the following five stages:

Stage 1: Construct substructures such as concrete piers, steel columns, abutments and concrete ramps. Temporary caissons at strategic locations will be constructed to facilitate the box girder installation.

Stage 2: Bearings at the abutments and at piers 1 and 5 will be installed. Shop manufactured box girder segments will be delivered to the site. There are 25 segments per bridge and 50 segments total for the entire bridge. The segments at the piers, each about 20 m long, will be erected and temporarily fastened to the steel columns. The temporary support shall be capable of supporting all temporary loading during construction.

Stage 3: The rest of the box girder segments with lengths varying from 10 m to 15 m will then be erected and supported on the temporary caissons constructed in Stage 1. Each segment will be temporarily connected to adjacent segments via bolting.

Stage 4: Final connections between the box girder segments with a bolted splice and between the box girder and steel columns at piers 2, 3 and 4 will proceed after all segments are installed and adjusted within the tolerance.

Stage 5: After final inspection remove temporary connections and caissons.

7 CONCLUSIONS AND ACKNOWLEDGEMENTS

Designing a signature bridge in a foreign country and under such a tight schedule created many challenges, many of which were not technical. Finalizing the bridge geometry, minimizing the risks and finding a balance between maintaining the high quality of the project and delivering it on time were some of the chief concerns aside from technical issues.

To design a signature bridge and realize the architectural vision, many innovative solutions were needed to solve unique technical issues, such as the column connections, as well as hooks and sliding bearings over Piers 1 and 5. The tuned mass dampers are very effective in vibration mitigation for this bridge, since the frequencies of the vibration modes susceptible to pedestrian-induced vibrations are in a narrow band of frequency range.

WXY architecture of New York City is the architect for this project, and developed ingenuous design to suit the history and future of the city, Xinjin. The City of Xinjin, Chengdu, China is the owner of this bridge.

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Chapter 8

Comparative of single pendulum and triple pendulum seismic isolation bearings on the St. Laurent Bridge, Quebec, Canada

J. Barbas, P. Matusewitch & M. Williams
Arup, USA

ABSTRACT: The St. Laurent Bridge, part of the new A30 highway in Quebec, is a low-level crossing over the St. Laurent River. This concrete structure supported on short stocky piers has been designed with seismic isolation to reduce the otherwise high seismic demand due to its support configuration. The seismic design compared the use of friction pendulum bearings in two of their modalities, single and triple pendulum, to determine which was most effective. In this particular case the single pendulum bearings performed slightly better, although the triple pendulum bearings were selected based on supplier price.

1 INTRODUCTION

1.1 *The A-30 project*

The A30 project will be built in two sections and will be a two-lane divided highway totaling over 54 km. The concession to finance, build, maintain and operate the western section for 35 years was awarded to Nouvelle Autoroute 30 S.E.N.C (a partnership between Acciona and Iridium) on October 7th 2008 and detailed design commenced thereafter. The construction itself is being carried out on a design-build basis by the Nouvelle Autoroute 30 Construction Joint Venture with lead design services provided by Arup. The public-private partnership procurement route should provide the Government of Quebec an estimated CAD $750 M savings compared to a traditional procurement route (KPMG, 2008).

The western section extends from Vaudreuil-Dorion to Châteauguay, a distance of about 35 kilometers. An additional seven-kilometer section will link up with Route 201 in Salaberry-de-Valleyfield. In total there will be 32 bridges as well as a tunnel under the Soulanges Canal. Two of these structures are major river crossings required to bridge the St. Laurent River and the Beauharnois Canal. The total length of these two bridges is about 4.4 km and the estimated construction cost is one third of the total construction cost of approximately CAD$ 1.5 B for the project.

1.2 *The St. Laurent Bridge*

As this reach of the St. Laurent River is not navigable, the bridge is a low-level crossing between Les Cedres on the north shore and St. Timothee on the south shore. It is located approximately 1 km downstream (east) of the Les Cedres hydroelectric generating station and approximately 4.5 km upstream (west) of the Pointe de Buisson dam. The river is almost 1.5 km wide at this site, requiring a total length of structure of 1.862 km (Figure 1).

Twin concrete structures will carry separate carriageways of two lanes each. The superstructures each comprise five NEBT 2000 precast, prestressed concrete beams supporting a 230 mm thick concrete deck. The beams will be continuous except at expansion joints.

Figure 1. Saint Laurent Bridge rendering.

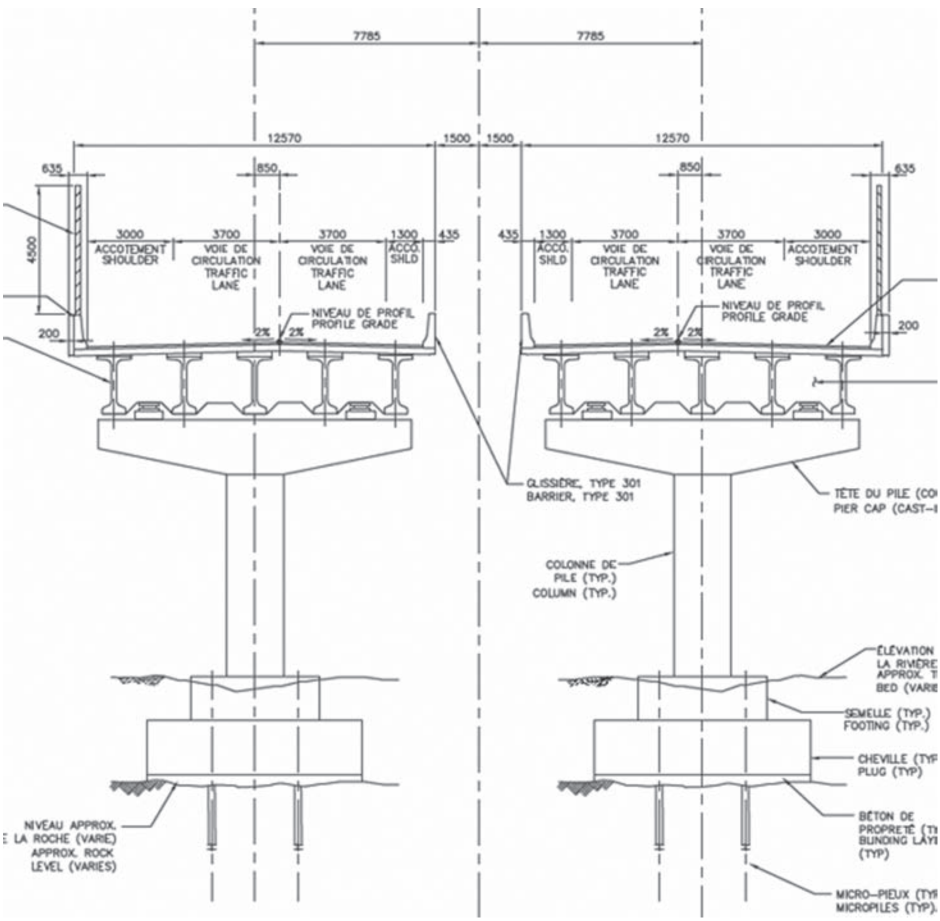

Figure 2. Typical section of St. Laurent River Bridge.

The deck spans transversely and comprises permanent precast concrete form panels working compositely with a cast-in-place concrete deck.

Typical spans are 45 m long. Expansion joints are provided at each abutment and at intermediate piers to divide the 42 spans into 6 interior units of 5 spans each, and two exterior units of 6 spans each.

Figure 2 shows the typical cross section at the piers. Each carriageway is supported on separate piers. All of the piers are reinforced concrete hammerhead types with solid circular columns. The columns are 2.0 m in diameter at Piers 2 through 34, and 2.5 m in diameter at Piers 35 through 42. The larger diameter columns are required to resist higher wind loads due to the presence of noise barriers on those spans.

The bridge was designed in accordance with the Canadian Highway Bridge Design Code (CHBDC) CAN/CSA-S6-06.

2 SEISMIC DESIGN

As is common with many rivers, the St. Laurent River follows the path of an ancient fault line which the site investigation determined to be inactive. The seismic risk of the area has been determined based on the published data from the Geological Survey of Canada (GSC) and CAN/CSA-S6-06.

The twin parallel structures are fairly narrow, so hammerhead piers were selected as an economical configuration. The bridge profile is relatively low to the water, so the columns are mostly short. The piers in the water are subject to large ice forces that must be resisted elastically, and the piers of the southern spans carrying noise barriers, which are also the tallest piers, are subject to large wind forces. The combination of these circumstances dictates very stocky pier columns. The non-seismic forces require pier columns either 2.0 m or 2.5 m in diameter; the profile results in pier columns that range approximately from a maximum height of 11 m down to less than 2 m.

A conventional bridge with these dimensions would have a very short fundamental period, experiencing large accelerations under earthquake shaking. The corresponding forces on the piers and foundations would be significantly larger than the non-seismic forces. This would require strengthening the piers beyond that required for non-seismic forces, increasing the stiffness of the bridge and in turn further increasing the seismic forces. The size of the foundations also would have to increase correspondingly.

Seismic isolation was thus pursued as a means to reduce the seismic demand on the short stocky piers.

3 SEISMIC ISOLATION

One alternative strategy to the yielding of ductile substructure elements to achieve both safety and efficiency is to reduce the seismic design forces by means of seismic isolation and then design the bridge to respond elastically to all forces, including the seismic forces. Buckle (2006) offers an authoritative review of the theory and practice of seismic isolation. To further reduce the seismic design forces, a more accurate non-linear time history analysis can be done as well, without the use of Response Modification Factors or other approximations. Arup selected this alternative strategy for the St. Laurent River Bridge.

Seismic isolation physically isolates a bridge superstructure from horizontal earthquake ground motions, uncoupling its motion from that of the earth. This greatly reduces the inertial forces caused by earthquake shaking. Mechanical devices with low horizontal stiffness and appreciable damping are installed between the superstructure and substructure to implement this strategy, lengthening the period of the bridge and increasing the effective damping.

Deformations associated with earthquake ground motions mostly occur in the isolators, not in the substructure.

Conventional bearings with elastomeric and/or sliding elements can lengthen the period of a bridge and increase its damping. The period is lengthened because the more flexible connection permits greater displacement of the superstructure compared to fixed connections. Damping is provided by the internal shear resistance of the elastomer or the friction of the sliding elements. However, these effects typically are limited. Additionally, positive connections between the superstructure and substructure (such as shear keys or guide bars on the bearings) are provided at some locations to limit relative superstructure displacements under service loads and earthquake shaking. These positive connections transfer much of the horizontal seismic force to the substructure, so substantial distortion of the substructure still occurs. To be effective, an isolation system must substantially isolate the superstructure from the substructure.

Specialized isolation bearings, fuses and damping devices have been developed to provide greater benefit than conventional bearings. Seismic isolators require the following characteristics:

– Horizontal flexibility to lengthen the fundamental period of the bridge
– Horizontal stiffness to transmit service loads such as wind, braking and centrifugal forces from the superstructure to the substructure
– Ability to accommodate slowly applied strains from thermal expansion/contraction, creep and shrinkage
– Energy dissipation (damping) to limit the relative displacements between the superstructure and substructure
– Capacity to carry all vertical loads.

Figure 3 illustrates a typical hysteretic force-displacement curve for a seismic isolator with an energy dissipating mechanism. The loading curve (the top line of the parallelogram) and the unloading curve (the bottom line of the parallelogram) are parallel but not coincident. The shaded area within the parallelogram represents the energy dissipated during each cycle of motion of the isolator.

Lengthening the period of a bridge with isolators reduces the corresponding acceleration of the structure. Figure 4 presents the CAN/CSA-S6-06 475-year return period response

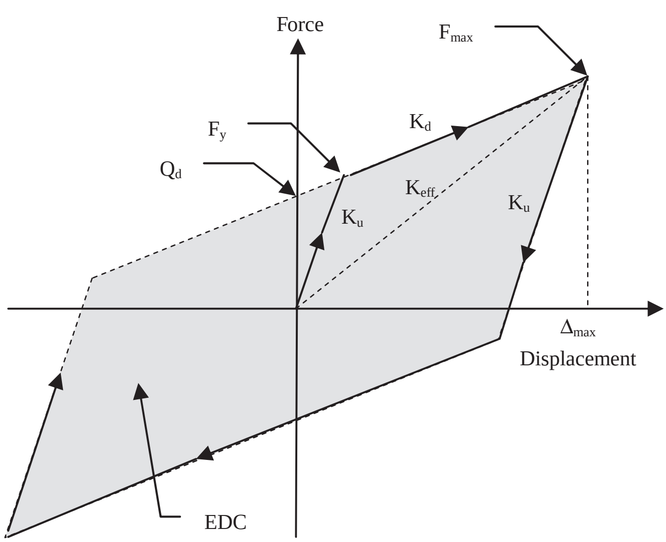

Figure 3. Typical force displacement curve for a seismic isolator.

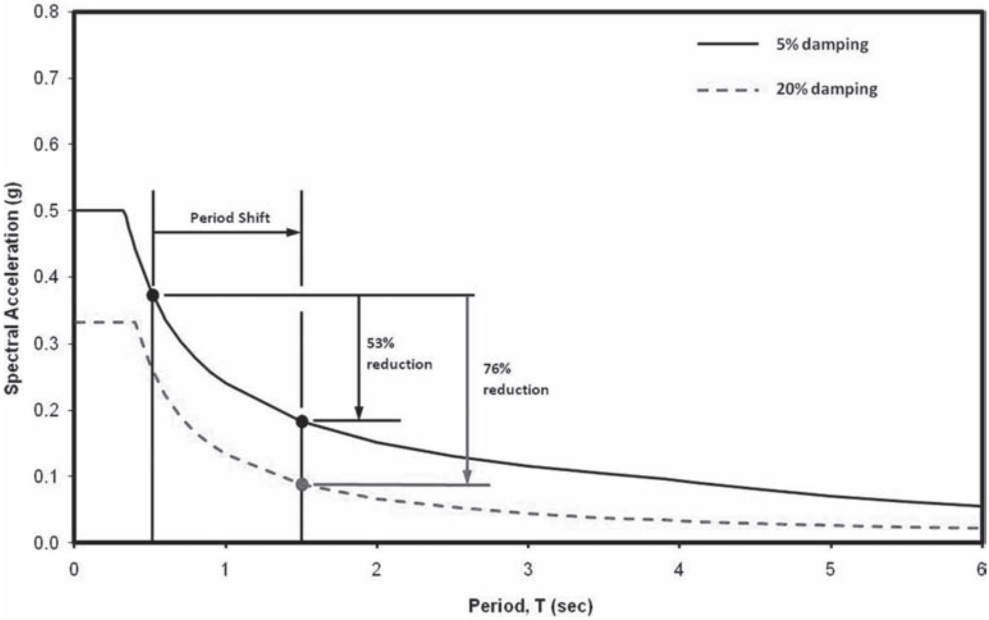

Figure 4. Period shift and increased damping.

spectrum with both 5% and 20% damping. Lengthening the period of a stiff bridge from 0.5 sec to 1.5 sec would realize a 53% reduction in the design accelerations. With 20% damping the total reduction in accelerations would be 76%. This illustrates the considerable advantages of seismic isolation. The ideal outcome would be to reduce seismic forces to less than or equal to non-seismic forces.

4 TYPES OF SEISMIC ISOLATORS

Some isolators combine all of the functions listed above in a single device, while others provide only some of these functions and are used in combination with supplemental devices, such as active or passive dampers, shear keys or fuses. Arup considered only seismic isolators that do not require supplemental devices for the St. Laurent River Bridge.

Arup considered two types of isolators: Lead-Rubber Bearings and Friction Pendulum Bearings.

4.1 *Lead rubber bearings*

Lead rubber bearings are laminated elastomeric bearings with a lead core to increase energy dissipation during lateral displacements. The lead core provides lateral stiffness against service forces such as wind, centrifugal and braking forces to minimize service level displacements, but yields under seismic lateral displacements, dissipating energy. Creep in the lead core accommodates slowly applied movements such as thermal expansion with minimal effect on the substructure.

Though cost effective, the stiffness of lead rubber bearings can increase significantly at low temperatures, possibly altering the response of the bridge appreciably. A change in product behavior at low temperatures can be accounted for in design if adequate data is available. The change in rubber material properties at low temperatures is well documented, but performance of specific bearings is not. Manufacturers of lead rubber bearings contacted during the design phase were unable to provide adequate data on performance of their specific bearings at low temperatures typically encountered at the bridge site.

Additionally, CAN/CSA-S6-06 cl. 4.10.1 states that isolation systems without self-centering capabilities shall not be used. Lead rubber bearings are not self-centering. The bridge may come to rest significantly out of position after an earthquake unless supplemental restoring device(s) are provided.

Although lead rubber bearings are the most commonly used isolator in the U.S., it was dropped from consideration due to the lack of self-centering capability and low temperature performance data.

4.2 *Friction pendulum bearings*

The principles and technical properties of the bearings have been published by the inventor and main manufacturer, EPS (2003). The following is a brief explanation of their behavior.

Friction pendulum bearings are spherical steel bearings with spherical sliding surfaces, as opposed to the flat sliding surfaces on conventional expansion bearings. The spherical shape of the sliding surfaces provides lateral stiffness and friction provides energy dissipation. Friction pendulum bearings are self-centering, meeting the requirements of CAN/CSA-S6-06 cl. 4.10.1.

Developed by Earthquake Protection Systems (EPS) of Vallejo, California, these bearings come in single, double and triple pendulum variations. The patent for the single pendulum bearing has expired, and a variation of it is now offered by various bearing suppliers.

The name friction pendulum bearing derives from the idea that sliding motion along a curved surface is similar to the motion of a conventional pendulum. The same equation of motion applies to both. The period of motion is controlled entirely by the radius of curvature of the sliding surface. The lateral stiffness of the bearing results largely from the curvature of the sliding surface, with a small contribution from friction. Friction largely accounts for effective damping. The bearings can provide very long periods of motion and substantial horizontal stiffness, and can accommodate large displacements and large vertical loads.

The sliding surfaces are lined with polished stainless steel and a low-friction composite material described as a non-metallic self-sacrificing lubricant. The composite material is a proprietary formulation of multiple layers of fibers and resins. The desired coefficient of friction across the sliding surface can be obtained by altering the make-up of the composite liner.

The curvature of the sliding surface gives rise to a restoring force equal to the tangential component of the vertical load on the bearing. Varying the radius of curvature increases or decreases the restoring force.

4.2.1 *Single pendulum bearings*

Single pendulum bearings maintain constant friction, lateral stiffness and sliding period for all motions and displacements.

Single pendulum bearings have been used in Canada on the Mississippi River Crossing in Ontario and on the White River Bridge in Yukon. Other cold weather locations where they have been used include the Sakhalin II Offshore Gas Platform in Russia and the Kodiak-Near Island Bridge in Alaska. To date Arup is not aware of any reports of problems with these bearings. Arup was involved in the design and testing of the Sakhalin II bearings, which included extensive verification of bearing properties at very low temperatures.

Single pendulum types were initially selected for the St. Laurent River Bridge. Upon the recommendation of EPS, triple pendulum bearings with equivalent characteristics were ultimately selected for the piers.

4.2.2 *Double pendulum bearings*

The double pendulum was developed to improve performance under smaller earthquake motions by adding a second pendulum mechanism with different period and friction. It has both upper and lower concave plates with an articulated slider between them. Stainless steel and composite liners are used in the same way as the single pendulum bearing, and the rotational surface is within the two-component slider. The upper and lower sliding surfaces have different friction coefficients and different radii of curvature.

4.2.3 *Triple pendulum bearings*

Since single pendulum bearings maintain constant friction, lateral stiffness and sliding period for all motions and displacements, their characteristics are selected to optimize bridge response to the design level of earthquake shaking. Therefore, bridge response is not ideal for lower or higher levels of earthquake shaking.

The triple pendulum bearing was introduced to improve performance under higher levels of earthquake shaking and to reduce the plan size of the bearings, by adding two pendulum mechanisms.

The triple pendulum bearing has a lower and upper concave plate with different radii and friction coefficients. Between the two concave plates there is a slider with internal concave plates and another slider within it. Thus each sliding surface can have a radii and friction coefficient for a particular performance.

5 DETERMINING OPTIMAL ISOLATION AND DAMPING CHARACTERISTICS

5.1 *Time history analysis*

A non-linear seismic time history analysis was done using the LS-DYNA software package.

A modified "hybrid" seismic response spectrum was proposed by Arup, and five time history records were matched to that spectrum for the seismic analysis, refer to Carter & Barbas (2008) for a description of the definition of the spectrum and matching of the time history records. Each of the five spectrally-matched time histories was run with dead load factors of 0.80 and 1.25 to account for varying vertical components of earthquake shaking, in accordance with CAN/CSA-S6-06 Table 3.2, for a total of 10 analysis runs.

After review and comment by the Independent Engineer, it was agreed that all bridge components also would be checked against time histories matched to a second response spectrum equal to the Geological Survey of Canada (GSC) 2500-year return period site-specific uniform hazard spectrum scaled up by a factor of 1.35 (1.35×2500-yr spectrum). Again, a total of 10 analysis runs were done with this second set of time history records.

5.2 *Parametric study*

A parametric study of single pendulum bearing characteristics was performed to find the most favorable balance between forces and displacements. In general, longer sliding periods make the bearings "softer", resulting in smaller forces but larger displacements. Higher friction offers greater resistance against service loads and increases damping (the bridge will return to equilibrium in fewer oscillations).

Four pairs of bearing characteristics were tested in the parametric study using the time histories matched to the "hybrid" spectrum:

- 1.8 sec period with 3% friction
- 2.0 sec period with 4% friction
- 2.2 sec period with 4% friction
- 3.0 sec period with 5% friction

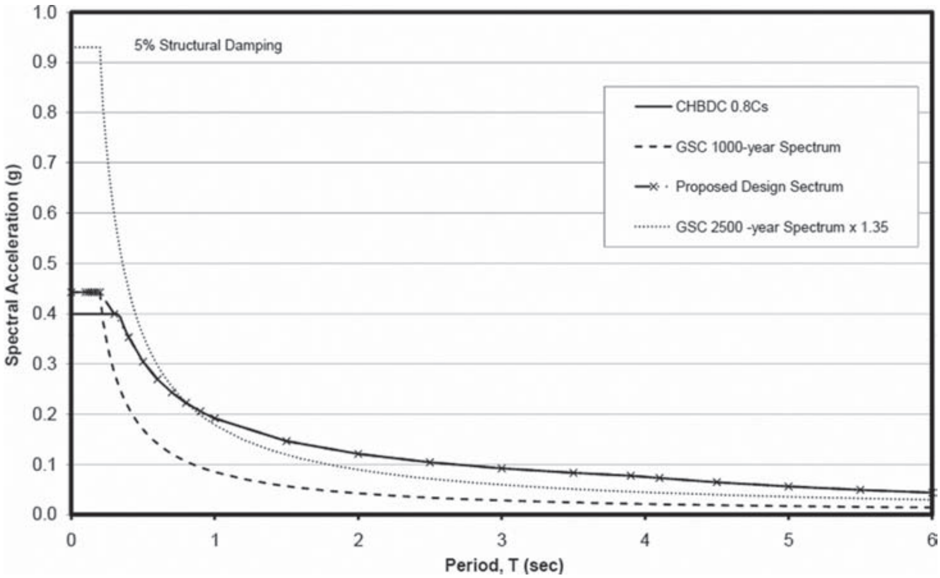

Figure 5. Seismic design spectrums for the St. Laurent Bridge.

Figure 6 presents the linear force-displacement curves used for each of the four pairs of characteristics. Note that the initial slopes are arbitrary to provide a ramp-up for the dynamic analysis, since zero displacement at a finite force would represent an infinite stiffness which cannot be modeled in a stiffness analysis.

Each set of bearing characteristics was used as input in two different models:

- An LS-DYNA model, to evaluate performance under earthquake shaking
- A static structural analysis model to evaluate performance under wind forces

The following structural responses were used to compare force effects:

- Longitudinal and transverse moments at the base of the columns.
- Longitudinal and transverse displacement of the tops of the columns.

The following movement responses were used to compare displacement effects of the superstructure:

- Longitudinal and transverse displacement of the bearings.
- Positive (opening) and negative (closing) longitudinal displacement at the expansion joints.
- Relative transverse displacement at the two halves of the expansion joints.

Additionally, all were checked for the restoring force requirement of CAN/CSA-S6-06 cl. 4.10.10.2.

5.3 *Seismic modeling results*

5.3.1 *Force effects on columns*

The moments at the bases of the columns and the displacements of the tops of the columns were compared to assess the force effects. Figures 7 and 8 present the transverse results graphically for each of the piers. The longitudinal effects have been omitted for brevity, although their magnitude is similar to the transverse effects. Although no one bearing gives the best results in all of these figures, the bearings with 2.2 sec period and 4% friction overall presents the most consistent favorable results.

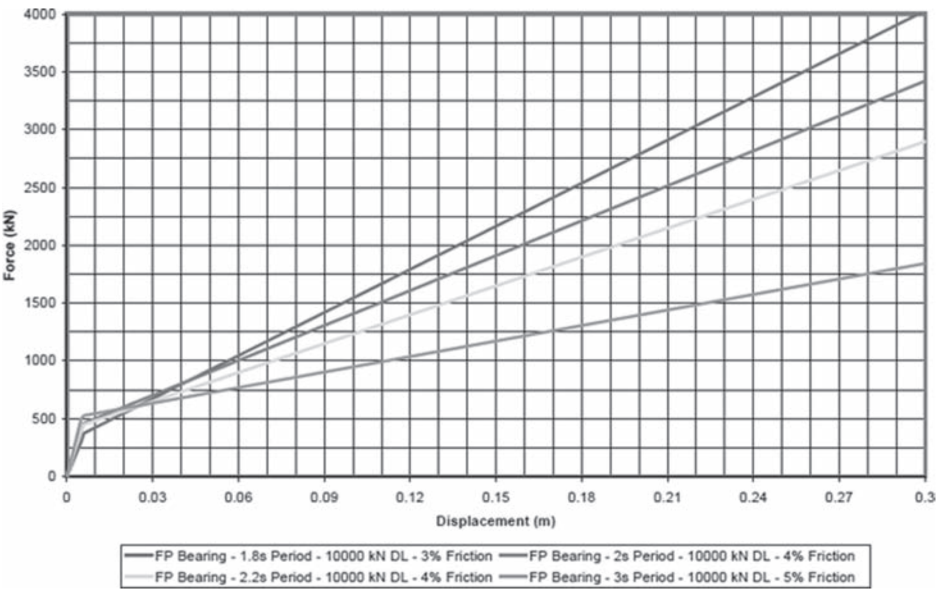

Figure 6. Single pendulum force-displacement curves for parametric study.

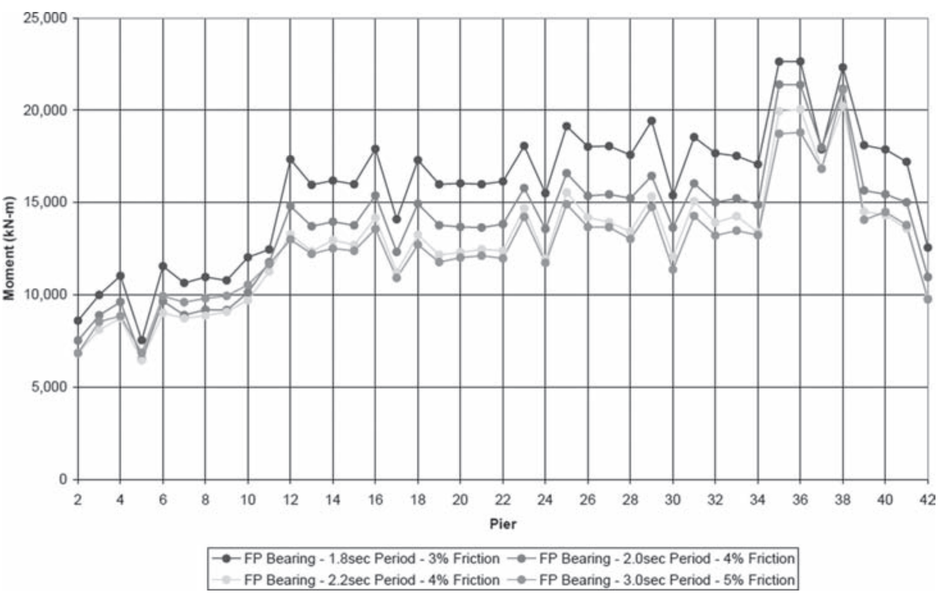

Figure 7. Column transverse base moments.

5.3.2 *Displacements*

The displacements of the bearings themselves and at the deck expansion joints were compared to assess the movement of the superstructure. Figure 9 presents the bearing transverse displacement results graphically for each of the piers. Again, although no one bearing gives the best results in all of these figures, the bearings with 2.0 sec period and 4% friction presents the most favorable results overall.

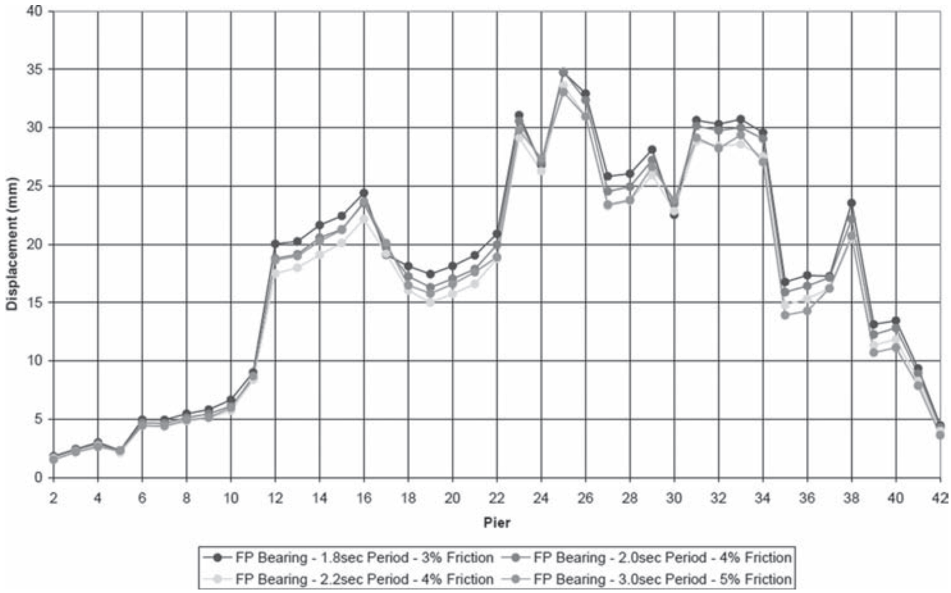

Figure 8. Top-of-column transverse displacements.

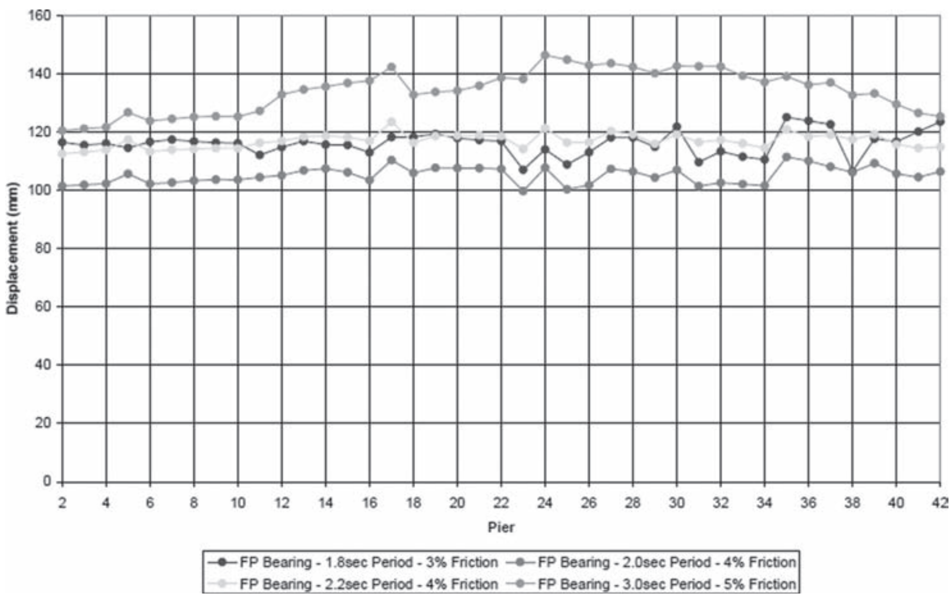

Figure 9. Bearing transverse displacements.

5.3.3 *Restoring force*

CAN/CSA-S6-06 cl. 4.10.10.2 requires that the bearings produce a restoring force such that the lateral force at the design displacement is at least 0.025 W greater than the lateral force at 50% of the design displacement. This was checked for all four pairs of bearing characteristics. The design displacement was taken as the largest resultant displacement occurring under any of the 10 time history runs for each set of bearing characteristics. The restoring forces for all four met this criterion.

5.4 *Conclusion for optimal isolation and damping characteristics*

None of the four sets of bearing characteristics performed best in all the comparisons. With no outstanding winner, the bearing with 2.2 sec period and 4% friction were chosen as the design criteria for the friction pendulum bearings.

5.5 *Revisions to the design*

Due to design development progressing in parallel to the seismic evaluation, several changes to the bridge design were made:

1. The number of expansion joints at piers was increased from six to seven to reduce creep and shrinkage effects in the superstructure.
2. The width of the diaphragms was increased, adding substantial mass.
3. Changes in the lengths of some columns and depth of some footings and plugs were made based on new data from the geotechnical investigation.
4. Shear keys were added between the adjacent span superstructures at the expansion joint diaphragms to restrain relative transverse displacements between spans. The west approach of the Beauharnois Canal Bridge, which uses the same type of swivel joints, but does not use seismic isolation, has shear keys between the superstructure and substructure, which also limits relative transverse displacements between spans. The shear keys were added to maintain a level of transverse protection for the joints consistent with that provided on the Beauharnois Canal Bridge.
5. The GSC 1.35 $\times$ 2500-yr seismic response spectrum criterion was added (see figure 5).

The time history analyses were repeated using time histories matched to the GSC spectrum, with all of the four structural changes above, for the selected 2.2 sec period, 4% friction, single pendulum bearing characteristics. The results before and after all of these design revisions were compared to assess the effects of these changes on seismic performance.

5.6 *Revised seismic modeling results*

5.6.1 *Force effects on columns*

The moments at the bases of the columns and the displacements of the tops of the columns before and after the design revisions were compared to assess the changes in the force effects. Figures 10 and 11 present these results graphically for each of the piers (longitudinal effects have been omitted for brevity). There are only small changes in the column transverse base moments.

There is a general moderate reduction in top-of-column transverse displacements.

5.6.2 *Displacements*

The displacements of the bearings themselves before and after the design revisions were compared to assess the changes in movement demands on the superstructure. Figure 12 presents the bearing transverse displacement results graphically for each of the piers. There are very significant reductions isolator displacements.

The displacements at the joints cannot be compared directly, since the number of joints changed.

5.7 *Conclusions*

The changes to the structure and the introduction of the GSC 1.35 $\times$ 2500-yr spectrum generally reduced both seismic force effects and seismic displacements, but for some piers some of the effects increased. This is not surprising since the two response spectra have different shapes.

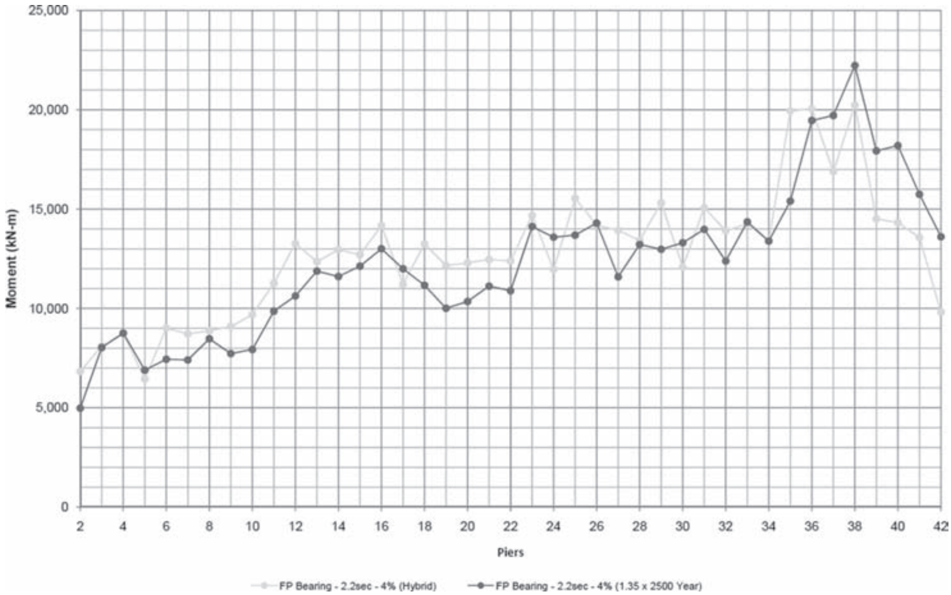

Figure 10. Column transverse base moments.

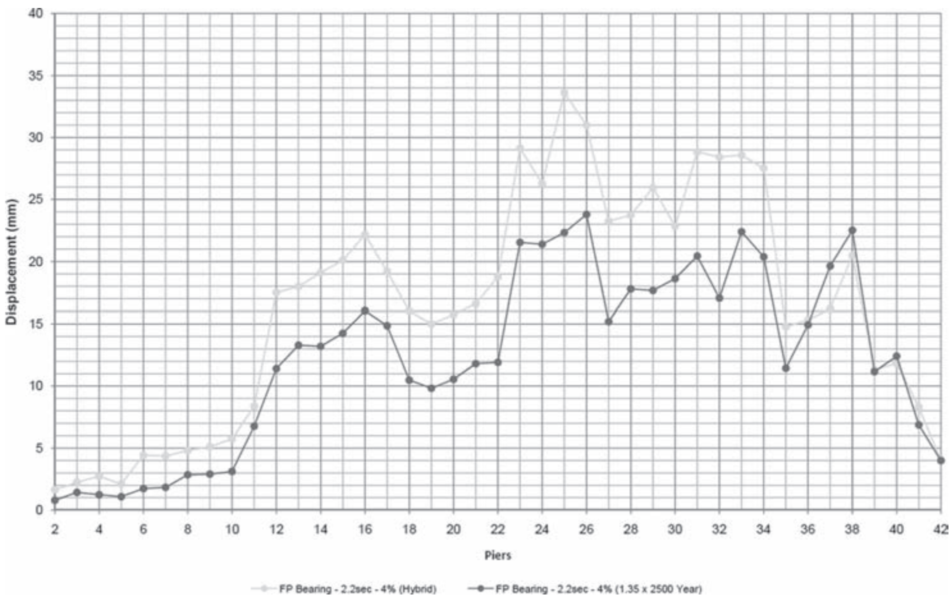

Figure 11. Top-of-column transverse displacements.

6 ALTERNATIVE BEARINGS PROPOSED

6.1 *EPS proposal*

Upon receipt of Arup's preferred single pendulum bearing characteristics, EPS conducted their own modeling and recommended switching to triple pendulum bearings at the piers and internally guided double pendulum bearings at the abutments.

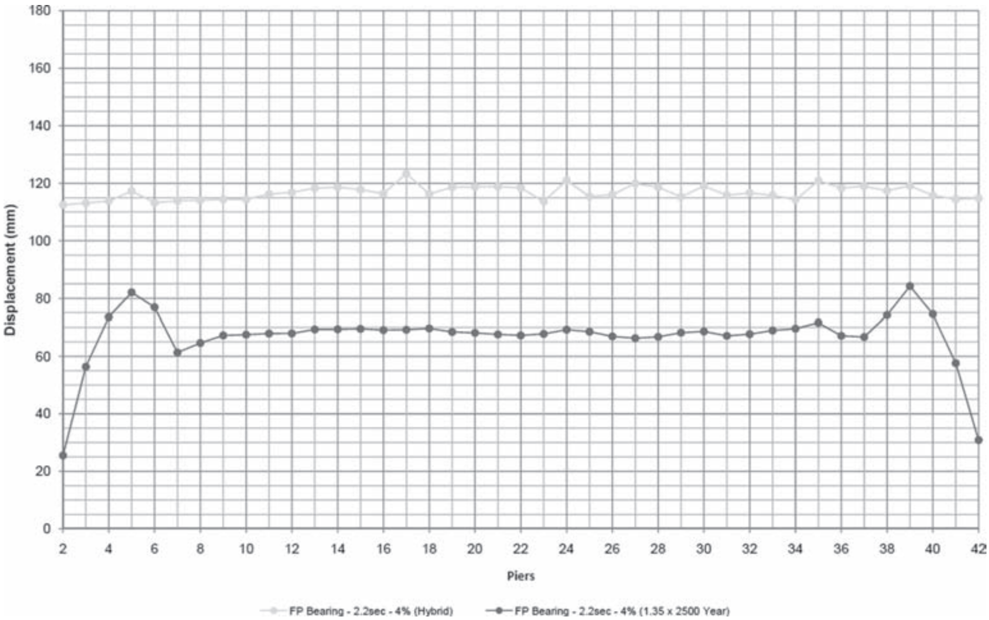

Figure 12. Bearing transverse displacements.

The recommended bearings use EPS's now standardized composite liners, with friction coefficients of 2% and 5%, which is more economical than custom composite liners to achieve a 4% friction coefficient. The recommended bearings also would have smaller plan areas than the single pendulum bearings. EPS was also of the opinion that the recommended bearings would have superior response over a wider range of earthquakes.

The proposed bearings were also attractive in price and delivery time, leading the Contractor to request their consideration.

Figure 13 presents the hysteretic force-displacement diagram for this bearing provided by EPS, which defines its characteristics.

This bearing has the following characteristics:

- Inner sliding friction: 2%
- Inner radius: 559 mm
- Inner period: 1.50 sec
- Intermediate sliding friction: 5%
- Intermediate radius: 1753 mm
- Intermediate period: 2.66 sec
- MCE displacement capacity: 218 mm
- Maximum displacement: 254 mm
- Maximum vertical dead load + live load: 8 451kN
- Maximum vertical dead load + live load + earthquake: 12 455kN

The bearing remains static up to an initial horizontal force equal to 2% of the total vertical load on the bearing. After that, the slider moves on the inner pendulum surfaces up to a displacement of 18 mm, controlled by the inner period of 1.50 sec and the inner friction of 2%. Beyond that point the sliding takes place along the intermediate pendulum surface, up to a displacement of 218 mm, controlled by the intermediate period of 2.66 sec and intermediate friction of 5%. The bearings are sized such that 218 mm maximum displacement of the intermediate pendulum interface is greater than or equal to the maximum displacement from Arup's seismic time history analysis. The 218 mm displacement capacity

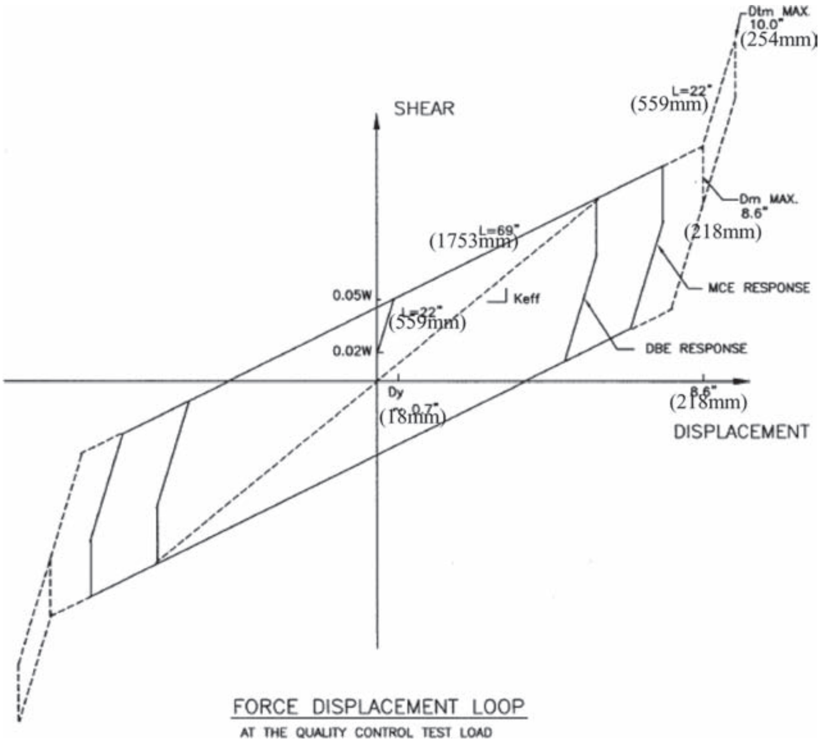

Figure 13. Hysteretic force displacement diagram.

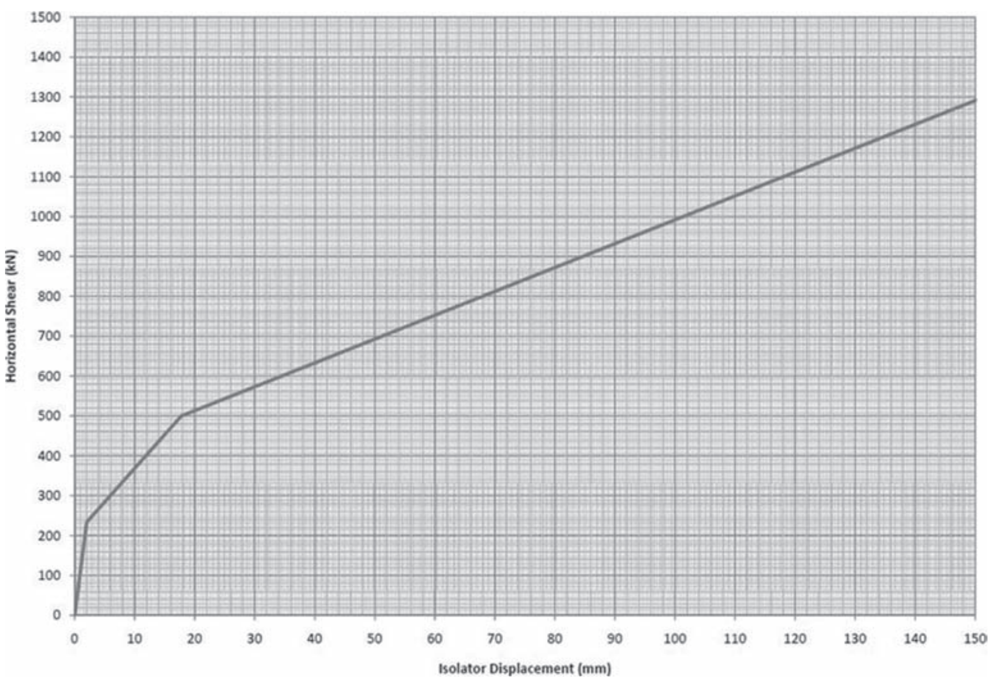

Figure 14. Force-displacement curve used for modeling.

is checked against the maximum non-seismic displacements as well. Earthquake shaking from still larger events that caused displacements beyond 218 mm would mobilize the third pendulum interface, with higher friction. Maximum displacements up to 254 mm can be accommodated.

The plan area of the bearing is governed by limiting the bearing pressure on the concrete pedestal below the bearing, so the bearing is larger than would be required for displacement capacity only.

Figure 14 presents the bi-linear force-displacement curve used to model this bearing in LS-DYNA and the static structural analysis. The initial slope is arbitrary to provide a ramp-up for the dynamic response, since zero displacement at a finite force level would represent infinite stiffness, which cannot be modeled in a stiffness analysis. The other two slopes represent the inner and intermediate pendulums, defined by their sliding periods. Since the bearing is sized to remain within the intermediate sliding capacity under seismic and non-seismic loads, the third sliding period is neglected for the analysis. The response of the bearing under the design earthquake accelerations is controlled almost entirely by the intermediate period, or the third slope in the diagram below.

7 COMPARISON OF SEISMIC PERFORMANCE OF SINGLE AND TRIPLE PENDULUM BEARINGS

The seismic time history analyses were repeated again for the new triple pendulum bearings under both response spectra. To assess the relative performance of the single and triple pendulum bearing characteristics, the results under the 1.35×2500-yr spectrum for each type of bearing were compared.

7.1 *Force effects on columns*

The moments at the bases of the columns and the displacements of the tops of the columns were compared. Figures 15 and 16 present these results graphically for each of the piers.

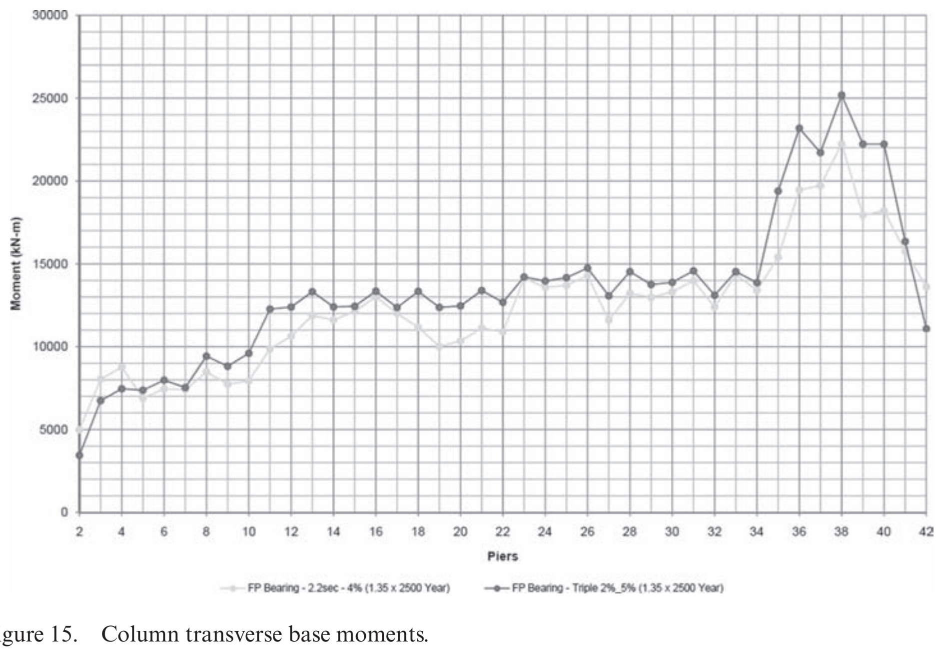

Figure 15. Column transverse base moments.

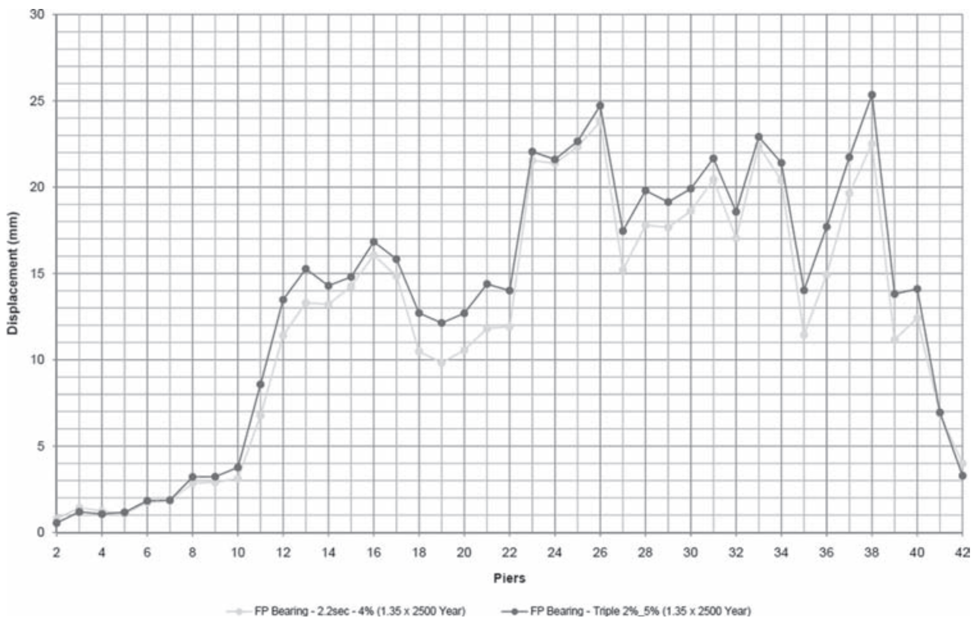

Figure 16. Top-of-column transverse displacements.

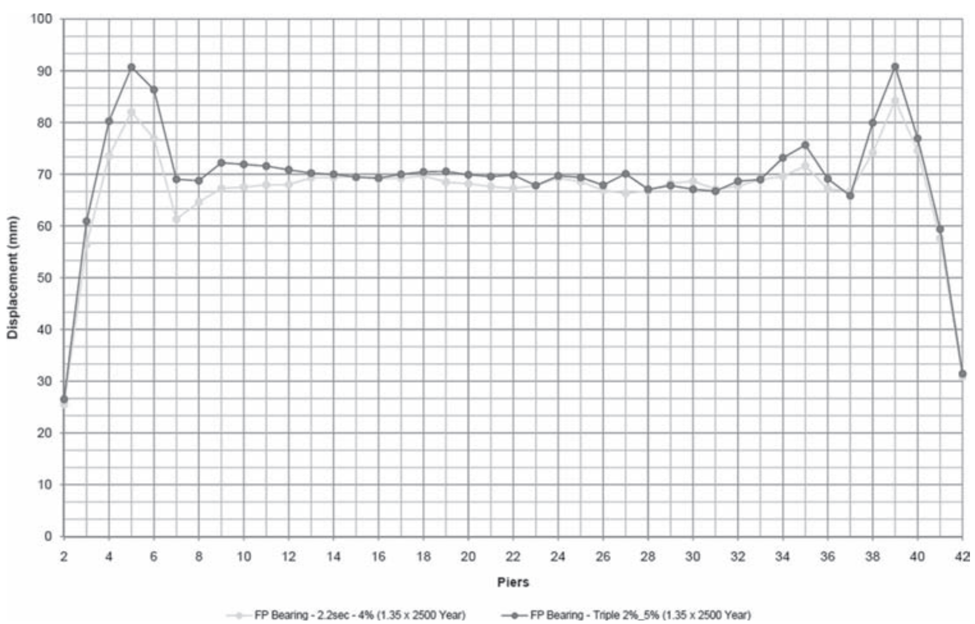

Figure 17. Bearing transverse displacements.

The column base moments are higher with the triple pendulum characteristics at some piers and lower at others. The differences are under 20%, with most under 10%. There is a similar pattern for the top-of-column displacements, which is consistent. The performance of the two bearings with respect to seismic forces is similar, though overall somewhat better with the single pendulum bearing characteristics.

7.2 *Displacements*

The displacements of the isolators and the expansion joints were compared. Figure 17 presents the bearing displacement results graphically for each of the piers.

The triple pendulum bearing displacements are up to 10% greater in the transverse direction.

7.3 *Conclusions*

Overall, the performance of the single pendulum bearing is more favorable than the performance of the triple pendulum bearings, although the difference is not very significant.

8 FINAL RESULTS

The following sections present the final results for force and displacement effects with the triple pendulum bearing characteristics under both design response spectra.

8.1 *Seismic results*

8.1.1 *Force effects on columns*

The shear forces at the tops of the columns, the moments at the bases of the columns and the displacements of the tops of the columns were compared to assess the force effects. Figures 18 through 20 present the transverse results graphically for each of the piers. The longitudinal effects were similar and have been omitted for brevity.

The results with the hybrid spectrum are greater for some piers while the results with the 1.35×2500-yr spectrum are greater for other piers in all comparisons. Differences are generally small, in the 10% to 15% range. Since the two spectral shapes are different, it is not surprising that neither spectrum produces consistently higher or lower results. The greater result at each pier was used for checking the columns.

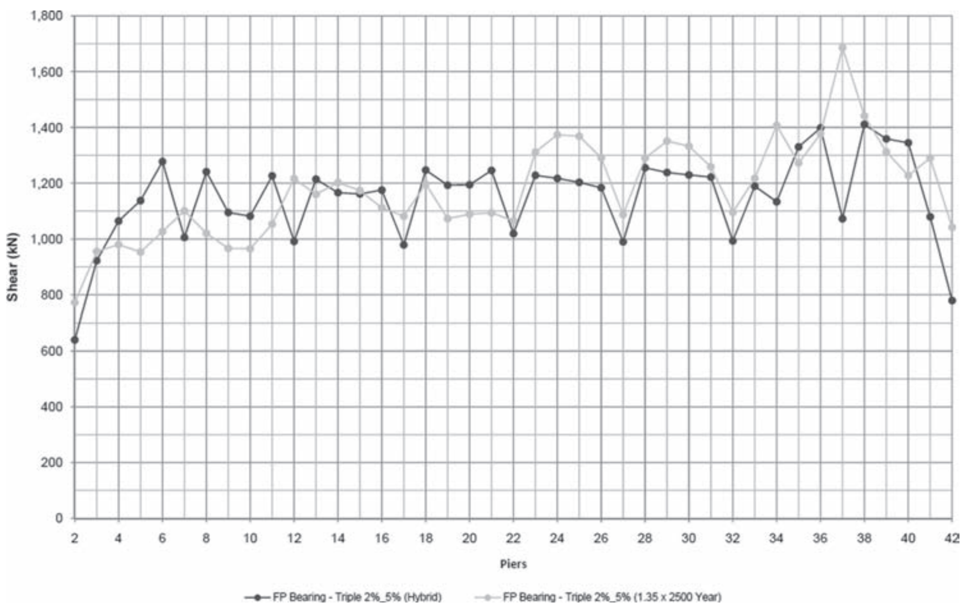

Figure 18. Column transverse shear force.

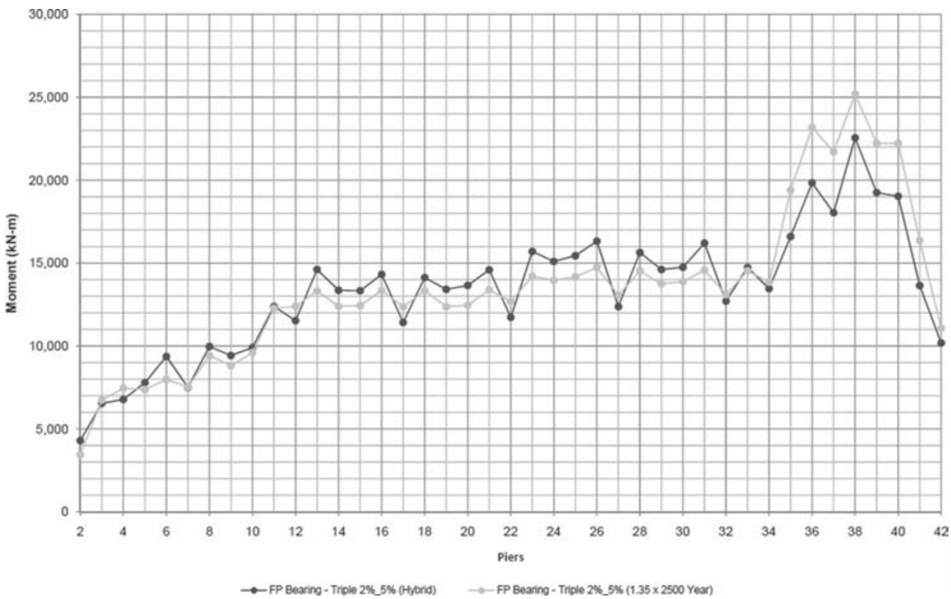

Figure 19. Column base transverse moments.

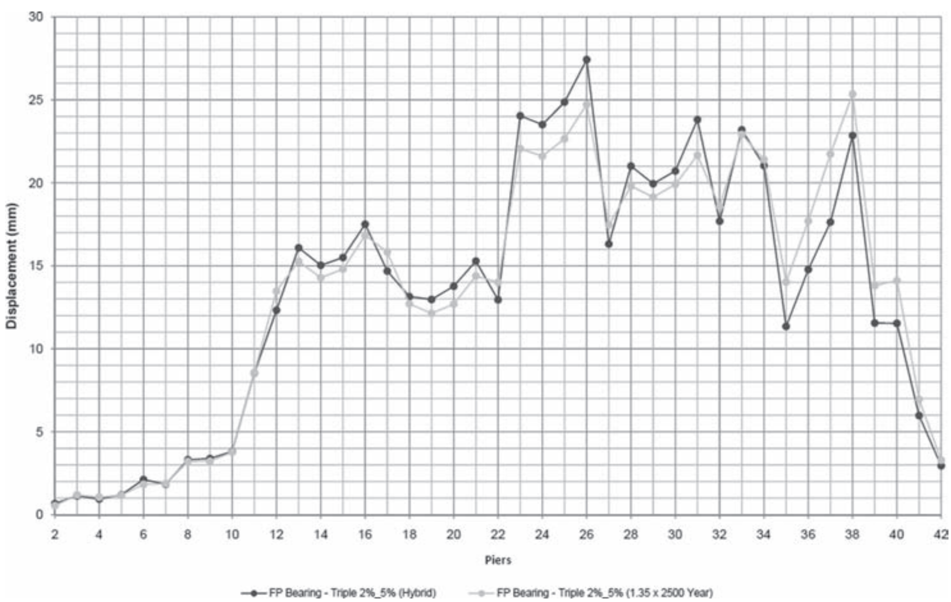

Figure 20. Top-of-column transverse displacements.

8.1.2 *Displacements*

The displacements of the bearings themselves and at the deck expansion joints were compared. Figure 21 presents the bearing transverse displacement results graphically for each of the piers. Relative transverse displacements will be restrained by shear keys.

The bearing and joint displacements are consistently larger with the hybrid spectrum. The larger results were used for checking the movement capacity of the bearings and joints.

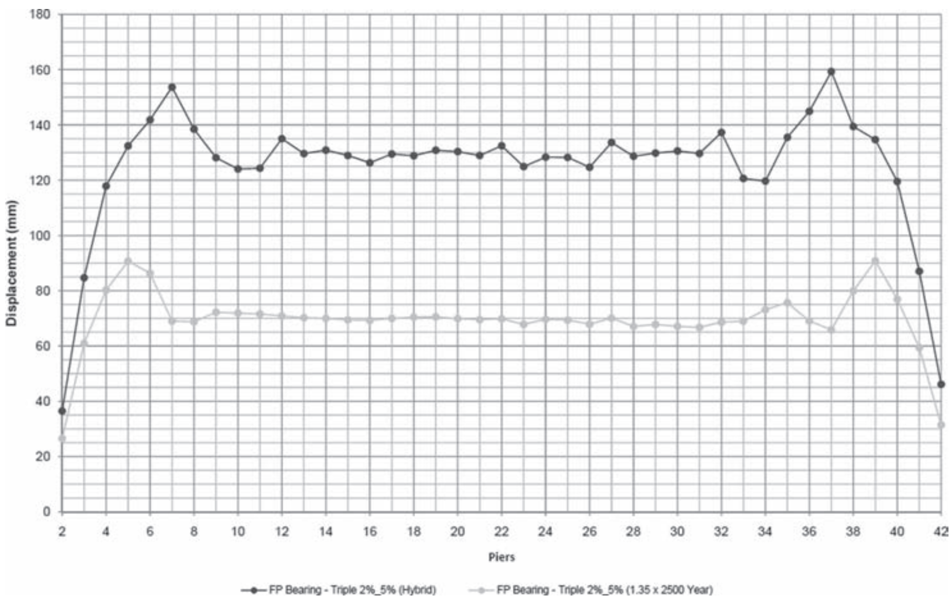

Figure 21. Bearing transverse displacements.

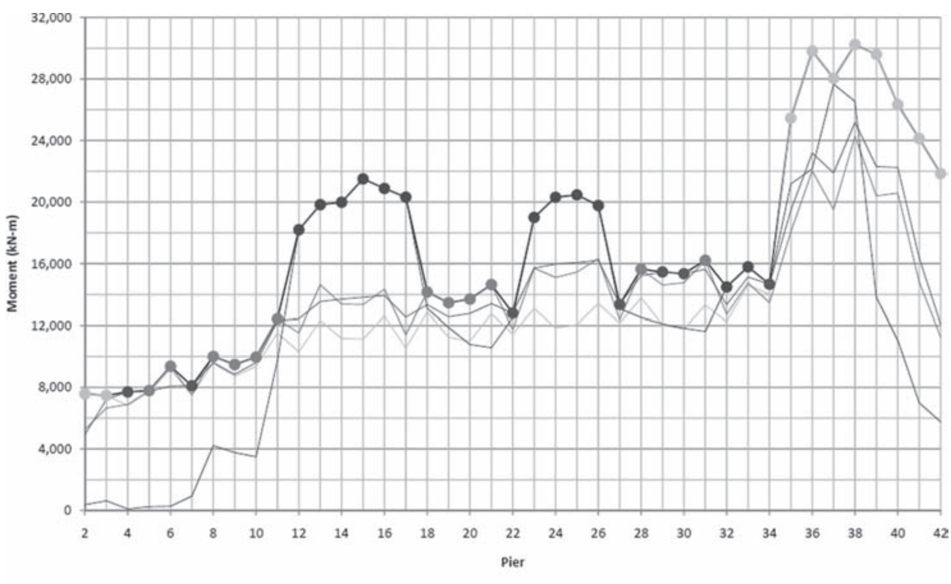

Figure 22. Comparison of governing column base moments.

8.2 *Comparison with non-seismic results*

Figure 22 presents a comparison of column base moments for the 42 piers graphically. None of the load combinations including live load govern, so they are not included. Load combinations including transitory and exceptional loads govern. The following load combinations are compared:

1. ULS load combination 4: creep, shrinkage, thermal and wind
2. ULS load combination 5: earthquake (hybrid spectrum)
3. ULS load combination 5: earthquake (1.35 × 2500-yr spectrum)
4. ULS load combination 6: ice.

The strategy of seismic isolation has produced good results overall. This conclusion is based on the following observations of Figure 22:

– The largest column base moment for the 2 m diameter columns (Piers 2 through 34) is caused by ULS load combination 6 (ice) at Pier 15.
– The largest column base moment for the 2.438 m diameter columns (Piers 35 through 42) is caused by ULS load combination 4 (creep, shrinkage, thermal, wind) at Pier 38.
– ULS 4 is the governing load combination at 10 piers.
– ULS 6 is the governing load combination at 12 piers.
– At 19 piers the seismic loads (ULS 5) govern, but the difference between the seismic and non-seismic base moments is less than 10% at 13 of those piers and on the order of 20% to 25% at the other 6 piers.
– At 35 out of 41 piers (85%) the seismic moments are either less than non-seismic moments or greater by not more than 10%.
– Of the 19 piers governed by seismic moments, 12 are associated with the hybrid spectrum and 7 are associated with the GSC 1.35 × 2500-year spectrum. The seismic moments at these piers are all within 15% of each other, and most are within 5% of each other.

8.3 *Design displacements*

Table 1 summarizes the design displacements and bearing capacities.

The non-seismic displacements are governed by ULS load combination 4, including creep, shrinkage, thermal effects and self-centering tolerance. ULS load combination 5 is the greater of the seismic displacements under either spectrum (hybrid and 1.35 × 2500-yr).

The displacement capacities were established by EPS based on fabrication standardization, limiting bearing pressures on the concrete pedestals below the bearings, and the displacement requirements from Arup in Table 1. The design displacements are checked against the Maximum Considered Earthquake (MCE) capacity of the bearings, since all of the modeling assumes that displacements will not exceed this capacity. This leaves extra displacement capacity for beyond-MCE events.

An additional case (not required in CAN/CSA-S6-06) comprising seismic displacement + ½ of the thermal displacement was also checked. The bearings cannot be offset to account for the ambient temperature at the time of installation, so it is advisable to provide additional displacement capacity, assuming that the bearings may be installed at a temperature that corresponds to ½ of the maximum thermal displacement. This combination governs for the bearings at the piers. ULS load combination 4 governs for the bearings at the abutments.

The displacement capacity-to-demand ratios are between 1.28 and 1.67.

Table 1. Bearing design displacements and capacities (mm).

	Design displacements					
Bearing	ULS 4 (Non-seismic)	ULS 5 (Seismic)	Seismic + ½ thermal	MCE capacity	Capacity to demand ratio	Max capacity
Continuity piers	120	146	170	218	1.28	254
Expansion piers	169	162	198	241	1.22	279
Abutments	152	87	120	254	1.67	254

8.4 *Damping*

Using the average of the seismic displacements for the piers above (155 mm), the damping calculated from the LS-DYNA modeling is 24% of critical.

9 CONCLUSIONS

The seismic design criteria have been a key factor defining the design of the structure. The short, stocky, hammerhead piers were selected as an economical configuration; however, they attracted large seismic forces. Seismic isolation was applied to reduce these forces.

Comparing the alternatives for seismic isolation, lead-core elastomeric bearings were discounted due to their lack of self-centering and their unproven performance at extremely low temperatures, such as those to which the St. Laurent Bridge is exposed in the Canadian winter.

Analytically, friction pendulum bearings proved to be effective in reducing the seismic demands, making them comparable to the non-seismic loads the bridge was to be designed for.

The initial evaluation and sensitivity analysis found that all four different types of single pendulum bearings considered provided effective isolation for the design earthquakes. The bearing that provided the best results overall was one with a 2.2 sec period and 4% friction.

The proposal by EPS to adopt their triple pendulum bearings was also evaluated and compared against the chosen single pendulum bearing. Overall, the performance of the single pendulum bearing is more favorable than the performance of the triple pendulum bearings, although the difference is small. In addition, the resulting forces for the triple pendulum bearing were comparable to those of the non-seismic loads.

The adoption of the triple pendulum bearing reduced the footprint of the bearing saving some material and making both comparable in cost.

Fine-tuning of the triple pendulum bearing may have allowed for better performance compared to the single pendulum bearing considered, but lead-in times and project schedule constraints did not allow for a more thorough design of the triple pendulum bearings.

It should be noted that the comparison between the single and triple pendulum bearings was performed under seismic time-histories made to match the highest expected earthquake. The multiple periods and frictions included within a triple pendulum bearing make the bearings more versatile for a wider range of seismic motions below and above the design values. This would require an additional assessment not performed as part of the design of the bridge.

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Chapter 9

Incorporating climate change predictions in the hydraulic analysis of bridges

G.M. Shields
The City University of New York, NY, USA

ABSTRACT: With the reality of climate change now widely accepted by recognized scientific organizations and governments, bridge owners are beginning to consider how climate change predictions may affect the safety of their bridges. In a study of bridge failure causes in the United States, over 60% were due to hydraulic factors. Changes in sea-level rise, precipitation, and storm frequency may have important implications in the hydraulic design, analysis, inspection, operation and maintenance of bridge structures. Climate change predictions for the northeastern United States are particularly alarming as many coastal cities would be affected. The incorporation of climate change predictions with respect to bridge hydraulics and scour will be discussed using the New York State coastal region as a case study.

1 INTRODUCTION

1.1 *The New York State coastal region as a case study*

New York State is comprised of an extensive network of highways, with a significant number of bridges along the coastlines of New York City and Long Island. According to the National Bridge Inventory, there are currently over 300 bridges within the New York State (NYS) coastal region (FHWA 2009). For the purposes of this study, the coastal region of New York State consists of the following counties: Bronx, Kings, Nassau, New York, Queens, Richmond, and Suffolk (see Figure 1). These counties were selected since they contain the majority of the tidal waterways within the state and the average elevation of the land is less than 3 m above sea level.

Infrastructure in the New York State, including bridges, is governed by a comprehensive set of city, state and federal regulations and standards. As the region's environmental baseline changes, these regulations—which are based on historic climate trends—may be compromised and fail to provide the level of protection and service they were designed to ensure. Changes in sea-level rise, precipitation, and storm activity may have important implications for bridge design, inspection, operation and maintenance of the state's bridges.

Figure 1. NYS coastal region study area.

1.2 *Observations of climate change*

Scientific consensus on climate change has grown rapidly in recent years as advances in analysis have been achieved. As evidenced by the Fourth Assessment Report (AR4) of the Intergovernmental Panel on Climate Change (IPCC), the reality of climate change is now broadly accepted by both national and internationally recognized scientific organizations and governments alike. The IPCC is the leading body for the assessment of climate change, established by the United Nations Environment Programme (UNEP) and the World Meteorological Organization (WMO). The organization is charged with providing "a clear scientific view on the current state of climate change and its potential environmental and socio-economic consequences" (IPCC 2010). The AR4 states that the average global surface temperature has increased over the past century. The 100-year linear trend of global surface temperatures, from 1906 to 2005, indicates an increase of 0.74°C. The temperature increase is widespread over the globe and is greater at higher northern latitudes such as the northeastern United States. Increases in sea level have also been observed with a global average sea level rise rate of 1.8 mm/year over the period from 1961 to 2003. Additionally, trends from 1900 to 2005 have been observed in precipitation amount and intensity. A significant increase in precipitation has been observed in the eastern parts of North and South America, and the intensity of tropical cyclone activity in the North Atlantic has increased since about 1970 (IPCC 2007). While trends have yet to be established, sea level rise and changes in land subsidence are resulting in increased damage from coastal flooding in many areas.

On a smaller scale, the New York State coastal area has also seen these climate changes but to different extents. The coastal area has experienced a significantly higher rate of relative sea level rise than the global average. The National Oceanic and Atmospheric Administration (NOAA) has maintained tidal gages in the United States through the Center for Operational Oceanographic Products and Services. In the region, tidal gages have been recording data for over 150 years in some locations. For example, on the western boundary of the study area, the tidal gage located at the Battery in New York City has observed a mean sea level trend of 2.77 mm/year with a 95% confidence interval of +/–0.09 mm/year based on monthly mean sea level data from 1856 to 2006 (NOAA 2009). This is equivalent to a change of 0.28 m in 100 years. Relative seal level rise trends at various tidal gages within the New York State coastal region are shown below (see Table 1).

1.3 *Predictions of climate change*

Climate change responses and effects depend on both natural and anthropogenic drivers. One such anthropogenic driver is greenhouse gas (GHG) emissions. Global GHG emissions due to human activities have grown since pre-industrial times, with an increase of 70% between 1970 and 2004 (IPCC 2007). Even with current climate change mitigation policies and sustainable development practices, global GHG emissions will continue to grow over the next few decades. Since climate change predictions are dependent on global GHG emissions rates, various scenarios were developed to "predict" future GHG rates. In 2000, the IPCC issued the Special Report on Emissions Scenarios which described the scenarios. Four scenario

Table 1. Sea level rise trends in the NYS coastal region.

Station name	Data collected since (Year)	Relative seal level rise (mm/year)
Montauk	1947	2.78
The Battery	1856	2.77
Port Jefferson	1957	2.44
Kings Point	1931	2.35

Table 2. IPCC emissions scenarios.

Scenario	Description
A1	A1 supposes a world of very rapid economic growth, a global population that peaks in mid-century and rapid introduction of new and more efficient technologies. A1 is divided into three groups that describe alternative directions of technological change: 1. fossil intensive (A1FI) 2. non-fossil energy resources (A1T) 3. balance across all sources (A1B).
A2	A2 assumes a very heterogeneous world with high population growth, slow economic development and slow technological change.
B1	B1 considers a convergent world, with the same global population as A1, but with more rapid changes in economic structures toward a service and information economy.
B2	B2 describes a world with intermediate population and economic growth, emphasizing local solutions to economic, social, and environmental sustainability.

categories, named A1, A2, B1 and B2, were considered (see Table 2). These scenarios take into account a wide range of demographic, economic and technological driving forces and resulting GHG emissions and are based on current climate polices. No likelihood has been attached to any of the scenarios as it is difficult to predict future climate change policies and practices. The scenarios were ultimately used to create 16 global climate models (GCMs). The GCMs were developed by various institutions around the world.

Beginning August 2008, the New York City Panel on Climate Change (NPCC) began looking at GCM predictions from a regional perspective. The NPCC used "IPCC-based methods to generate model-based probabilities for temperature, precipitation and sea level rise from GCM simulations based on three GHG emission scenarios" (NPCC 2009).

The emissions scenarios selected were A2 ("worst case"), A1B ("business as usual") and B1 ("best case") because most of the GCMs contain these scenarios in their simulations. Predictions were averaged across the 16 GCMs for both temperature and precipitation, and these results were superimposed onto historical climate information for the New York City region. The same method was employed for sea level rise predictions, but these predictions were based on seven GCMs that included sea level rise as part of their analysis. Extreme events, such as hurricanes and nor'easters, tend to occur on daily, rather than monthly, time scales. Since monthly output from climate models is considered more reliable than daily output, the NPCC employed a "hybrid projection technique." This technique utilized the monthly precipitation output in the same manner as the annual output. The monthly projections were applied to the observed precipitation values of the Central Park rain gage data collected from 1971–2000. While this is a simplified approach to the projection, this method does help transportation decision makers to prepare for extreme events. The results of the regional climate models for the NYS coastal region predict sea level to rise as much as 30–140 cm and possible precipitation increases of 5 to 10%. In addition, another study suggests that the frequency of events storm will double by the end of the 21st century (Bender et al., 2010).

2 NATIONAL AND STATE RESPONSE TO PREDICTIONS

2.1 *United States*

With the reality of climate change now broadly accepted by both national and internationally-recognized scientific organizations and governments, bridge owners are beginning to consider how climate change may affect the safety of their bridges and are looking to reduce the risk of failure. This sentiment was recently echoed by the national Transportation Research Board (TRB) 2008 Report. The report notes that "historical regional climate patterns commonly used by transportation planners to guide their operations and investments may no longer be a reliable guide for future plans" (TRB 2008).

In response to this finding, the TRB recommends that state and local governments should incorporate climate change into their long-term capital improvement plans, facility designs, maintenance practices, operations, and emergency response plans. Taking measures now to evaluate and protect the most vulnerable infrastructure should pay off by diminishing near-term maintenance expenditures and reducing the risk of catastrophic failure which will directly affect human life and economic activity.

Additionally, it has been determined that transportation professionals often lack sufficiently detailed information about expected climate changes and their timing to take appropriate action. TRB recommends that research that provides climate data and decision support tools should include the needs of transportation decision makers such as bridge owners.

2.2 *New York*

In August 2009, Governor David A. Paterson signed Executive Order No. 24 (State of New York 2009). One goal of the Executive Order is to identify and assess short-term and long-term actions to adapt to climate change across all economic sectors, including transportation. The Executive Order also created the New York Climate Action Council with a directive to prepare a draft Climate Action Plan. The Climate Action Plan will assess how all economic sectors can reduce greenhouse gas emissions and adapt to climate change. The Plan will incorporate developments and findings by ongoing projects addressing climate mitigation options and adaptation strategies. Ongoing projects include: the ClimAID adaptation project, the NYS Sea Level Rise Task Force, and PlaNYC.

Part of the PlaNYC initiative is to protect critical infrastructure from the effects of climate change. To this end, Mayor Michael Bloomberg established the New York City Panel on Climate Change (NPCC) in August 2008. The NPCC was charged with advising the Mayor and the New York City Climate Change Adaptation Task Force on issues related to climate change and adaptation strategies as it relates to infrastructure. The NPCC published a series of three workbooks. The first workbook, "Climate Risk Information," discusses observed and predicted climate changes in the New York City region which was previously discussed. The second workbook, "Adaptation Assessment Guidebook," outlines a general, eight-step process to help stakeholders inventory their vulnerable infrastructure and develop adaptation strategies to address these risks. The eight adaptation assessment steps are shown below (see Table 3).

The final workbook, "Climate Protection Levels," discussed the need to develop design standards that incorporate climate change predictions so that existing and future infrastructure can operate safely. The workbook specifically highlights the need to update design standards and regulations governing bridges, particularly in regards to scour.

Table 3. NPCC adaptation assessment guidelines (NPCC 2009).

Phase	Guideline
1	Identify current and future climate hazards
2	Conduct inventory of infrastructure and assets
3	Characterize risk of climate change on infrastructure
4	Develop initial adaptation strategies
5	Identify opportunities for coordination
6	Link strategies to capital and rehabilitation cycles
7	Prepare and implement adaptation plans
8	Monitor and reassess

3 INCORPORATING CLIMATE CHANGE PREDICTIONS

3.1 *Develop decision making tools and criteria*

Utilizing the TRB recommendation to create decision support tools that address climate change predictions, this study is in the process of developing a geographic information systems (GIS) platform that maps the locations of coastal NYS bridges overlaid with climate change predictions such as sea level rise and storm surge elevations. The geographic location and attributes of each bridge for any state, not just New York, is available through the National Bridge Inventory. Climate change predictions for the region were provided by the Institute for Sustainable Cities within the City University of New York. It is hoped that such a tool would enable bridge owners within the region to manage their current inventory more effectively. State agencies such as the New York State Department of Transportation currently use the GIS platform to monitor their infrastructure and could readily incorporate new information.

Before developing a decision making tool, criteria must be developed. This study is attempting to employ a holistic approach to assessing the vulnerability of a bridge to hydraulic failure. Criteria is being developed in three main categories: a) hydrologic and hydraulic, b) structural and geotechnical and c) social importance (see Figure 2). Suggested areas for criteria development within each of the main categories are briefly discussed in this paper.

3.1.1 *Hydrologic and hydraulic criteria*

Hydrologic characteristics are those that deal with the movement and quantity of water on Earth. The hydrologic factors considered in this research are sea level rise, hurricane activity and precipitation. Sea level rise and hurricane activity are on a more regional scale and will be used as criterion to determine hydrologic vulnerability. Precipitation will be incorporated during the analysis phase using data local to each bridge site. While current design standards do consider historic maximum flooding events, they currently do not incorporate sea level rise due to climate change projections. The bridge sites most vulnerable to sea level rise are situated at the lowest elevations, typically closest to the shoreline. The rapid ice melt scenario projections predicting sea level rise developed by the NPCC are considered in this study because the predictions are more conservative than other scenarios. The bridge sites that would be affected most by these predictions would be the sites within the current 100-year flood zone which are at the lowest elevations. The current 100-year flood can produce approximately 2.6 m surge for much of New York City. These sites would only further be inundated by 2080 with the projected increase of 22 cm in sea level. These sites would be identified as having the highest priority.

Hydraulic characteristics are those that deal with the mechanical properties of water such as pressure, flow and velocity. These characteristics are needed for flooding and scour analysis. Scour is defined as the "erosion of the streambed or bank material due to flowing water" (Richardson & Davis 2001). Characteristics such as the number of spans, cross sectional area under the bridge, embankment health, navigability and bridge skew all affect the ability of water to be conveyed.

The number of spans indicates the degree to which the bridge constricts the flow of the waterway in the immediate area. Multiple, short spans indicate that there are multiple piers in the water which reduces the cross sectional flow area under the bridge. A reduced cross

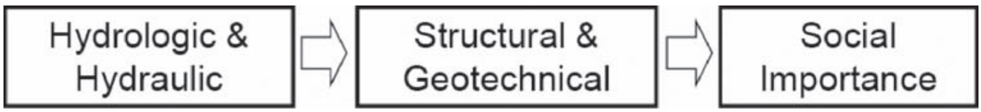

Figure 2. Main categories for decision making criteria.

sectional area increases the flow velocity which increases erosion due to scouring of the streambed. During a flood event such as the 100-year storm, scouring may occur rapidly and the bridge may be compromised within a short period of time.

The reduced cross sectional area of the waterway also makes the bridge more prone to flooding and overtopping of the roadway. If the roadway is flooded, motorists will not be able to use the bridge. Bridges are currently rated by the National Bridge Inspection Standards (NBIS) for their ability to pass the 100-year storm (or less) under the bridge (NBIS 1995). However, this is based on historic or observed flood events, not predicted flooding events due to climate change.

Waterway embankment with evidence of slumping, erosion, or local failures are an indication that the channel is vulnerable to failure, particularly during a flood event. Embankments with well established vegetation or man-made protection such as steel sheeting or riprap are less vulnerable.

Ideally, the length of a bridge pier should run parallel to the flow of water under the bridge so that turbulence of flow is at a minimum. If a bridge pier is not parallel to the flow, this is known as a skew. The skew angle is the angle between the centerline of a pier and a line normal to the roadway.

3.1.2 *Structural and geotechnical criteria*

The type of structure is an important factor. If the main structure of the bridge is continuous, this is more structurally redundant than a simply supported bridge. A redundant structure has alternate paths for the loads to travel which can allow for more time to remove motorists from the bridge. The NBIS Item 43 gives a general classification to the main superstructure of the bridge. Simply supported concrete slabs or steel floor systems are classified from 1 through 7. Truss, arch, suspension, and moveable bridges are classified from 8 through 19.

The year the bridge was built may be an indicator of its vulnerability. For example, bridges built prior to 1940 along the south shore of the study region tend to be characterized by simply supported spans, short span lengths and piers comprised of multiple, short timber piles. Short span lengths may mean that multiple piers are in the waterway which increases obstruction to flow. The use of timber piles in the early 20th century was common, but the timber piles are often not embedded enough to resist scour or the depth of embedment is unknown. All of these factors make a bridge more vulnerable to failure.

The overall rating of the superstructure (deck, roadway, truss members, etc.) and substructure (piers, piles and abutments) of the bridge should be considered. Bridges in excellent condition will be expected to perform as designed while bridges in poor condition are likely to be more vulnerable to flooding and scour.

3.1.3 *Social importance criteria*

Not all bridges are created equal, so to speak. The social importance of a bridge depends on how great an effect it would have on the population if it were to fail, be taken out of service completely or have a reduced level of service. Bridges carrying interstate highway traffic are typically of higher importance than a local road and should be given a higher priority.

Another means of identifying the importance of a bridge is the average daily traffic (ADT). The ADT is a record of the average number of vehicles that traverse the bridge each day. Bridges that serve a large number of motorists are of greater importance than bridges serving a small number of motorists.

The social importance of a bridge is also reflected in the bypass or detour length a motorist would have to travel should the bridge become out of service. A detour length is defined as the amount of additional kilometers that a motorist would have to travel in order to reach his or her destination. The greater the detour length, the greater the travel time and economic cost (ex: price of gasoline) the motorist will experience. Since the average commuter in the New York City Region experiencing a commute of 30 minutes, a detour of 30 km would essentially double the commute time (Pisarski 2006).

The preservation of a bridge with historical significance is often important to a community. The historical significance of a bridge may involve a variety of characteristics. The bridge may be a unique example of the history of engineering, architecture or art. The crossing itself might be significant because it is associated with a significant event or circumstance. Within the United States, bridges are designated as historic by the National Register of Historic Places.

3.2 *Prioritize and analyze the most vulnerable bridges*

Once the criterion has been defined, the bridge inventory needs to be prioritized. Suggested characterizations from most vulnerable to least are: critical, high, medium and low priority as is being used in this case study. Those bridges defined as "critical" require immediate attention while bridges characterized as "low" priority should be discussed as part of long term fiscal plans. Analysis should begin with the bridges characterized as "critical" followed by "high" priority, and so on.

4 CONCLUSIONS

Due to the wide acceptance of climate change, it is prudent for all agencies—including bridge owners—to consider the impact of these predictions on their infrastructure. Obtaining regional climate change prediction data will likely be the greatest challenge for bridge owners. Large metropolitan areas such as New York City are at the forefront of developing regional climate models due to the city's desire to protect the tremendous investment made in their infrastructure. Secondly, the development of prioritization criteria should take on a holistic, multidisciplinary approach in order to fully assess the vulnerability of a bridge to hydraulic failure.

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3 Innovative bridge technology

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Chapter 10

Concrete filled tubular flange girder bridge

R. Bondi
Michael Baker Jr., Inc., Moon Township, PA, USA

ABSTRACT: A Concrete Filled Tubular Flange Girder (CFTFG) is an I-shaped girder that uses a concrete filled hollow structural section (HSS) as the top flange. A key advantage of the CFTFG is the increased torsional stability that reduces the need to brace the girders under construction loading conditions. Time and cost savings in fabricating and erecting the bridge girder system can be realized by a reduced number of diaphragms along with the locating the field splices over the piers. Combined with span-by-span construction, CFTFGs provide a robust steel structure that can erected much more efficiently than conventional I-girder bridges. A first of its kind bridge in the world, using of Concrete Filled Tubular Flange Girders (CFTFGs), has been designed and constructed in 2010. This demonstration project is a two span structure over Tionesta Creek, located in western Pennsylvania. This demonstration project marks the culmination of research funded by FHWA and PennDOT from concept, to laboratory prototype, to implementation.

1 INTRODUCTION

A demonstration bridge, using Concrete Filled Tubular Flange Girder (CFTFGs), was designed and constructed for the Pennsylvania Department of Transportation (PennDOT) District 1-0. The two-span demonstration bridge, carrying SR 1003, spans the Tionesta Creek in Forest County, Western Pennsylvania near the small town of Lynch Village. The bridge utilizes simple spans for dead load made continuous for live and superimposed dead load effects. A CFTFG is an I-shaped girder that uses a concrete filled hollow structural section (HSS) as the top flange.

The design and construction of this bridge is the culmination of past research and development sponsored by the Federal Highway Administration (FHWA) and PennDOT. The potential advantages of CFTFGs for bridges, including large local buckling resistance, large torsional stiffness, and reduced web slenderness, were outlined by Wassef (Wassef et al., 1997). Wimer & Sause (2004), Kim & Sause (2005a, b, 2008a) and Kim et al. (2008) conducted extensive experimental and analytical studies of bridge girders with concrete-filled tubes as flanges (CFTFGs). A design study of CFTFGs for a four girder prototype bridge was performed (Wimer & Sause 2004). The CFTFGs had a rectangular concrete-filled tube as the compression flange and a flat plate as the tension flange. The design study showed that the large torsional stiffness of the tubular flange enables the use of large unbraced lengths in the bridge framing system. These results were verified by large-scale experiments on two 60 ft long CFTFGs. Kim & Sause (2005a, b, 2008a) studied the behavior of CFTFGs with a round concrete-filled tube as the compression flange and a flat plate as the tension flange. FE analyses and large-scale experiments (Kim et al., 2008) were conducted to investigate the flexural strength and stability of CFTFGs. Kim & Sause (2005a, b) summarized several advantages of CFTFGs relative to conventional I-shaped plate girders for bridges: (1) the concrete-filled tubular flange provides more strength, stiffness, and stability than a plate flange with same amount of steel, and (2) fewer cross frames are needed for CFTFGs to maintain lateral-torsional stability compared to similar plate girders, which reduces the

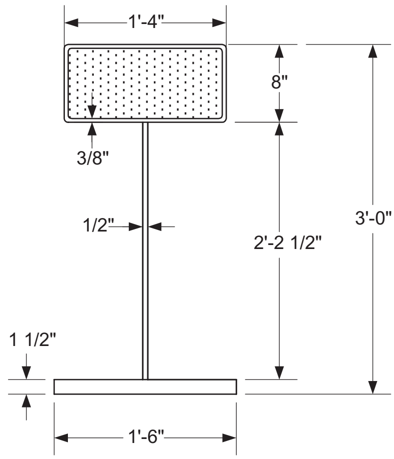

Figure 1. CFTFG girder.

fabrication and erection time and effort needed to construct a bridge. They also provided formulas for determining the lateral-torsional buckling strength of CFTFGs.

The demonstration bridge has two 100-ft spans and an overall length of 200 ft from centerline of abutment bearing to bearing on a tangent alignment and straight grade. The bridge has two 11'-0" lanes, two 3'-0" shoulders and utilizes standard PennDOT barriers 1'-8 ¼" wide and 2'-8' high along each overhang for an overall structure width of 31'-4 ½" and a curb to curb width of 28'-0". The bridge cross section has four CFTFG's spaced at 8'-5 ½" centers and 3'-0" overhangs. The physical dimensions of the CFTFG's are given in Figure 1. The CFTFGs are made composite with the deck using stud shear connectors. The stud shear connector weld to the relatively thin steel tube filled with concrete was a concern. The concerns for the weld was satisfied because for buildings, with cast-in-place concrete floors, some beam flanges are relatively thin, 3/8 inch thick, similar to the tube thickness, and have performed quite well. Also, the butt weld from the flux filled studs happens quite quickly and was therefore viewed as not having detrimental effects to the concrete in the tube. The concrete deck slab is designed to be 8" thick and reinforced with grade 60 mild reinforcing steel placed in two layers.

Each simple span has intermediate diaphragms at approximately 50-ft spacing. Additional diaphragms are placed 3-ft from the centerline of the pier in each span to accommodate construction of temporary 93-ft simple spans during erection. Temporary erection bearings are placed under these diaphragms during construction, supported upon bracket attached to the pier faces. Another line of diaphragms is placed 4-ft from the centerline of bearings at each abutment to facilitate future bearing replacement and to reset the bearings at Abutment 2 during construction. All diaphragms are wide flange W21X57 beams bolted to full height connection plates welded to the CFTFG's. The girders rest on laminated elastomeric bearings at each support.

2 CONCRETE FILLED TUBULAR FLANGE GIRDER (CFTFG)

A CFTFG is a steel girder that uses a concrete filled HSS top flange. The HSS flange is filled with unreinforced concrete in the fabrication shop after girder fabrication. The concrete in the tube strengthens the compression flange of the girder. The torsional stiffness and strength of the girder is significantly increased by the tube, thus increasing the lateral-torsional buckling (LTB) resistance of the girder. Both of these advantages are important during steel erection and concrete deck placement, before the composite deck hardens. The concrete filled tube also has the effect of reducing the depth of web in compression, thus decreasing the web slenderness.

This increased stability and strength of a CFTFG permits the lateral bracing to be reduced compared to conventional plate girders and for CFTFGs to span greater distances with the same structure depth. The increase in torsional stiffness and strength also eliminates the need for fabricating exterior girders with intermediate constructability stiffeners needed on normal plate girders to resist overhang forces. Diaphragms (or cross-frames) and stiffeners are among the most labor intensive and expensive components (per pound of steel) to fabricate and erect, therefore, fewer diaphragms and stiffeners reduces cost and increases speed of construction. A few transverse stiffeners along the girder length are necessary to control cross section distortion, allowing the girder to fully realize its LTB strength (Kim & Sause 2005a, Kim & Sause 2008a).

In addition to weight savings over conventional plate girders, CFTFG's are able to provide greater under-clearance than typical plate girders or prestressed concrete options, resulting in further overall project savings by minimizing the need to raise approaches, add spans and costly substructure units or add additional girder lines. Span-by-span construction further increases the speed of the steel erection, aided by implementing a continuity splice at the pier.

The following discussion focuses on design assessment of CFTFGs for Constructability, where their advantages are most significant. The flexural design criterion for CFTFGs for Constructability is $M_u \leq \phi_f M_n$. Here, M_u is the largest value of the major-axis bending moment in the girder due to factored loads specified in Chapter 3 of the 2004 AASHTO LRFD Specifications (AASHTO 2004, Kim & Sause 2008b). For the construction loading, the following load combination is used in accordance with AASHTO 3.4.2, 1.25*DC +1.5*LL, where the live load (LL) results from the deck finishing machine and other construction live loads. This design criterion is used in place of AASHTO (AASHTO 2004) Equations 6.10.3.2.1-1 and 6.10.3.2.1-2 from Article 6.10.3.2.1 and Equation 6.10.3.2.2-1 from Article 6.10.3.2.2. The calculation of M_n replaces the calculation of F_{yc}, F_{nc}, and F_{yt} for a non-composite section from Article 6.10.3.2.1 and Article 6.10.3.2.2, which refer to Article 6.10.8 of the 2004 AASHTO LRFD Specifications (AASHTO 2004, Kim & Sause 2008b).

The nominal flexural strength, M_n, is taken as: (Kim & Sause 2008b):

$$M_n = M_d^{br} \leq (M_s \text{ and } M_d) \tag{1}$$

The flexural strengths M_d^{br} and M_d are LTB strengths, calculated as described below. M_s is the cross-section flexural strength, which is usually the calculated yield moment M_y, although in unusual cases (described below) the calculated non-composite compact section moment capacity, M_{ncc}^{sc}, may be less than M_y, so M_s is the minimum of M_y and M_{ncc}^{sc}. M_y for a CFTFG non-composite with the concrete deck is taken as the smaller of M_y^{tr}, and M_y^{sc}. M_y^{tr} based on a transformed section analysis, is the smaller of M_{yc}, and M_{yt}. In calculating M_y^{tr}, the concrete in the steel tube is transformed to an equivalent area of steel using the modular ratio. M_y^{sc} is calculated based on an equivalent rectangular stress block for the concrete in the steel tube and a linear elastic stress-strain curve for the steel with the yield strain, ε_y reached in the steel at either the top or the bottom fiber (Kim & Sause 2008b).

Note that if the tube yield stress is large (e.g., 100 ksi) and the compressive strength of the concrete infill is small (e.g., 4 ksi), then M_{ncc}^{sc} may be less than M_y. In this case, M_{ncc}^{sc} should be used for M_s (Kim & Sause 2005a). M_{ncc}^{sc} is determined using an equivalent rectangular stress block for the concrete and an elastic perfectly plastic stress-strain curve for the steel. The maximum usable strain ε_{cu} is assumed to be 0.003 at the top of the concrete in the steel tube (Kim & Sause 2008b).

The ideal design flexural strength, M_d, is based on the assumption that each diaphragm provides perfect lateral and torisonal bracing at the brace point, and is given by Kim & Sause (2008b):

$$M_d = C_b \alpha_s M_s \le M_s \tag{2}$$

The moment gradient correction factor, C_b, is given by AASHTO (2004) Equation 6.10.8.2.3-7. The strength reduction factor, α_s, is given by Kim & Sause (2008b):

$$\alpha_s = 0.8\left(\sqrt{\left(\frac{M_s}{M_{cr}}\right)^2 + 2.2} - \left(\frac{M_s}{M_{cr}}\right)\right) \le 1.0 \tag{3}$$

where, M_{cr} is the critical moment, given by

$$M_{cr} = \frac{\pi E}{L_b/r_y}\sqrt{0.385 K_T A_{tr} + 2.467\frac{d^2 A_{tr}^2}{\left(L_b/r_y\right)^2}} \tag{4}$$

The radius of gyration is given by

$$r_y = \sqrt{\frac{I_{tf} + I_{bf}}{A_{tr}}} \tag{5}$$

Note that I_{tf} is based on a transformed section for the concrete-filled steel tube using the short-term modular ratio to account for the concrete in the tube.

The design flexural strength for torsionally braced CFTFGs, M_d^{br}, is calculated because research (Kim & Sause 2005a) showed that the effective bracing of CFTFGs provided by a system of interior diaphragms coupled with the inherent flexural stiffness of the CFTFGs may not be sufficiently stiff so that lateral buckling occurs only between the brace points. This condition can be attributed to the large torsional stiffness of the CFTFGs as well as the reduced number of interior diaphragms enabled by using CFTFGs. The method for calculating M_d^{br}, is given by Kim & Sause (2005a) and is based on the approach described by Yura et al. (1992). M_d^{br} is given by:

$$M_d^{br} = C_{bu} \alpha_s^{br} M_s \tag{6}$$

The strength reduction factor for the torsionally braced girder is given by Kim & Sause (2008b):

$$\alpha_s^{br} = 0.8\left(\sqrt{\left(\frac{M_s}{M_{cr}^{br}}\right)^2 + 2.2} - \left(\frac{M_s}{M_{cr}^{br}}\right)\right) \le 1.0 \tag{7}$$

M_{cr}^{br} based on the approach described by Yura et al. (1992) is given by:

$$M_{cr}^{br} = \sqrt{\left(M_{cr}^{ubr}\right)^2 + \frac{C_{bb}^2}{C_{bu}^2} M_{br}^2} \tag{8}$$

M_{cr}^{ubr} is given by Kim & Sause (2008b):

$$M_{cr}^{ubr} = \frac{\pi E}{L/r_y}\sqrt{0.385 K_T A_{tr} + 2.467\frac{d^2 A_{tr}^2}{\left(L/r_y\right)^2}} \tag{9}$$

The moment including the torsional bracing effect, M_{br}, which is derived by Yura et al. (1992), is given by

$$M_{br} = \sqrt{\frac{\beta_T E I_{eff} n}{1.2L}} \quad (10)$$

The effective brace stiffness, β_T, and the effective moment of inertia, I_{eff}, are given by Yura et al. (1992) and the details are not repeated here for brevity. The effective brace stiffness is a function of the girder framing plan, cross frame stiffness and the stiffness of transverse stiffeners. These bracing stiffness components are related by their reciprocals and therefore the one of least magnitude carries the most significance, which is the stiffness of the girder system.

3 DEMONSTRATION BRIDGE FIELD SPLICE AT PIER IN PEAK NEGATIVE MOMENT REGION

A preliminary design (Kim & Sause 2008b) of the demonstration bridge proposed the use of simple spans for dead load made continuous for live and superimposed dead load effects. This was done in anticipation that CFTFGs can be most useful when accelerated construction is needed. The use of span-by-span erection and only a few diaphragms increases the speed of erection. This arrangement greatly reduces the demands on the negative moment region at the pier. The design approach at the pier location is the CFTFGs resist dead loads from CFTFGs and concrete deck as simple spans. The CFTFGs are then spliced and made continuous for superimposed dead and live loads. The longitudinal reinforcing steel in the deck was sized to resists the tension forces at the pier resulting from the negative bending moment. The bottom flange resists the compression forces from negative bending. Because the deck reinforcing steel was size to resist the negative bending tension forces, tension stresses in the tube are very minimal and resisted by the tube, neglecting the concrete fill. The advantages sited for the CFTFGs are lost in negative bending. The CTFTGs revert back to performing basically as a plate girder with no benefit of the concrete filled tube. The CFTFGs for the demonstration bridge were designed to retain their continuity indefinitely. The performance for simple span conditions under full design loads (PennDOT 2007) was not checked due to continuity.

A bolted mechanical splice (Figure 2) was chosen over other options on the basis of constructability and long-term maintenance. Several methods for making the continuity splice at the pier were considered. Azizinamini et al. (2005), Lampe & Azizinamini (2000), Barber (2006) and Wasserman (2005) presented various methods for making simple span steel bridges continuous for live load with mixed results. Use of a concrete end diaphragm, with or without a kicker plate at the bottom flange, was generally employed in this previous work. This approach was not chosen because calculation revealed that with a relatively shallow depth of the CFTFG, the stresses near the bottom flange in the concrete diaphragm could not be accommodated.

Concerns over the quality of the field welding required at the kicker plate and ultimately concerns for future deck replacement ruled out further consideration of this technique. PennDOT plans for the eventual replacement of their concrete bridge decks, using a heavily reinforced concrete diaphragm would complicate deck replacement, as the resulting reversal stresses in the concrete diaphragm may be difficult to predict and accommodate.

The bearing stiffeners are located outside of the splice region to also act as connection plates for the end diaphragms near the pier. The girder web at the centerline of pier and permanent bearing was checked, because of bearing stiffeners are located outside of this region, and satisfied AASHTO Section D6 requirements because the web splice plates are considered doubler plates that strengthen the web against crippling and bridge over the web seam

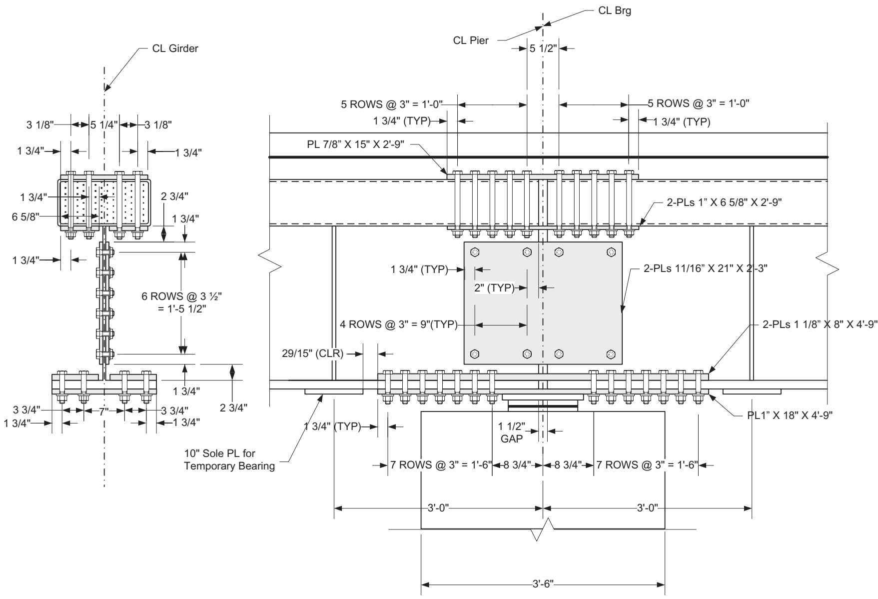

Figure 2. Final splice design.

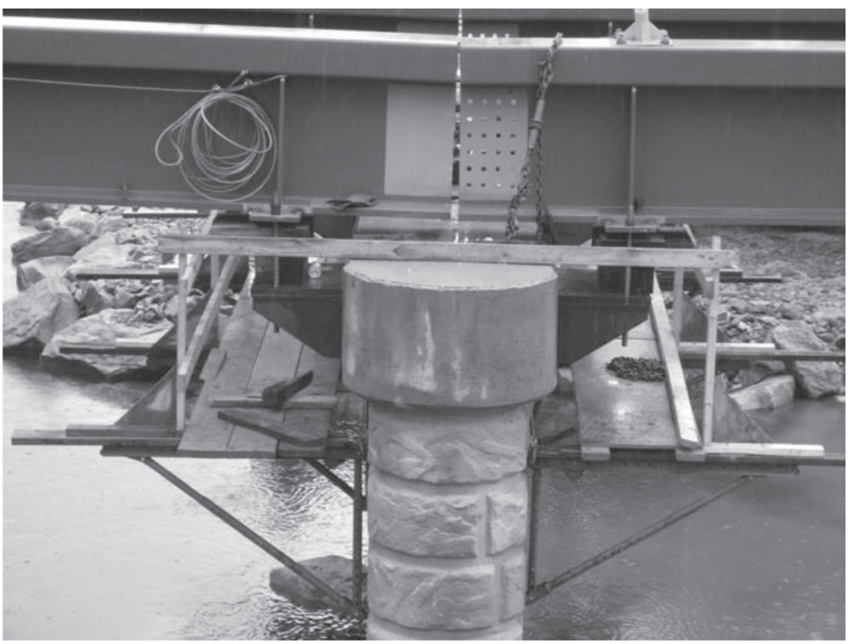

Figure 3. Picture of tube girder at continuity splice during girder erection.

The bearing stiffeners at the end diaphragm locations were necessary for the LTB resistance as mentioned previously by Kim & Sause (2005b). The need for a hand-hole in the tube was eliminated by specifying high strength bolts with the desired bolt grip through the entire HSS top flange. To account for field conditions that could deviate from calculation, a large tolerance in the gap between the girder ends was specified as well as field drilling one side of the splice. On the shop-drilled side of the splice, the tube was filled with concrete in the fabrication shop. Greased bolts were placed in the bolt holes then removed after the concrete cured. The tube on the field-drilled region of the splice was not filled with concrete in the fabrication shop. This region was grouted in the field after the splice was bolted up. A picture of the proposed splice at the pier during erection of the girders appears in Figure 3. The remainder of the tube was filled with concrete in the shop.

It should be noted that the bearing sole plate at the pier was welded to the bottom flange splice plate. Because of this, the bottom flange splice bolt spacing exceeds AASHTO sealing requirements. The sealing requirements were relaxed because the splices are well protected from moisture, and have a redundant corrosion protection system of painted weathering steel.

The bolted field splice was designed similar to a splice near a dead load contraflexure point of a plate girder bridge. Elastic distribution of the stresses across the cross section was assumed with a linear strain profile. The influence of the bearing, nearby diaphragm and complex geometry creates a region of non-uniform stress distribution. However, there is little research on field splices placed in these conditions and this remains an area of need for future research. In addition to the elastic negative moments produced from the design loads, the splice was also analyzed for the effects of load reversal during future deck replacement. This did not govern the design but it is an important consideration for future designs.

4 CONSTRUCTABILITY-ERECTION OF DEMONSTRATION BRIDGE

The preliminary design of the demonstration bridge (Kim & Sause 2008b) included precast post-tensioned concrete deck slabs. Precast post-tensioned deck slabs along with CFTFGs together form a system well suited for accelerated construction. However, further considerations in the final design phase of the demonstration bridge eliminated the use of precast deck panels to simplify the design. A conventional cast-in-place reinforced concrete deck was used.

After the reinforced concrete deck was placed, continuity was established at the pier with the bolted splice. Unlike the preliminary design utilizing precast deck panels, the exterior, fascia CFTFGs are subjected to torisonal loading during the deck placement. For conventional plate girders, the torsional loading produces significant flange lateral bending stresses. Therefore, a detailed erection analysis was undertaken to verify the constructability of the deck placement.

The substructures were constructed in the normal fashion. The girders were erected on the permanent abutment bearings and a temporary erection bearing supported from pier brackets (Figure 4). Attention was paid to the detailed gap between the girders. Since the girders were set as simple spans, they experienced significant rotation at the supports. The permanent bearings at the far abutment were reset after the deck was placed and the girders rotated into their final position. The use of an integral abutment at the other end of the bridge eliminated the need to reset the bearings at that location.

The bridge deck was subsequently placed as a simple span bridge in the positive moment regions with the exception of the blockout regions at the pier and abutments. A blockout region 6 ft long was provided directly over the splice region to allow for splicing the girders after the remainder of the deck cured (Figure 4). Blockouts were provided at each abutment because an integral abutment was at one end and a full depth concrete diaphragm was at the other end.

As the girder deflected from its cambered position it theoretically assumed the shape of the profile grade, however, the bolt holes on one side of the splice were only subpunched in the shop and then reamed in the field prior to making the connection to account for inaccuracies in camber. The girder splice was then made at the pier, (See Figure 5), leaving several rows of bolts out of the bottom splice plates.

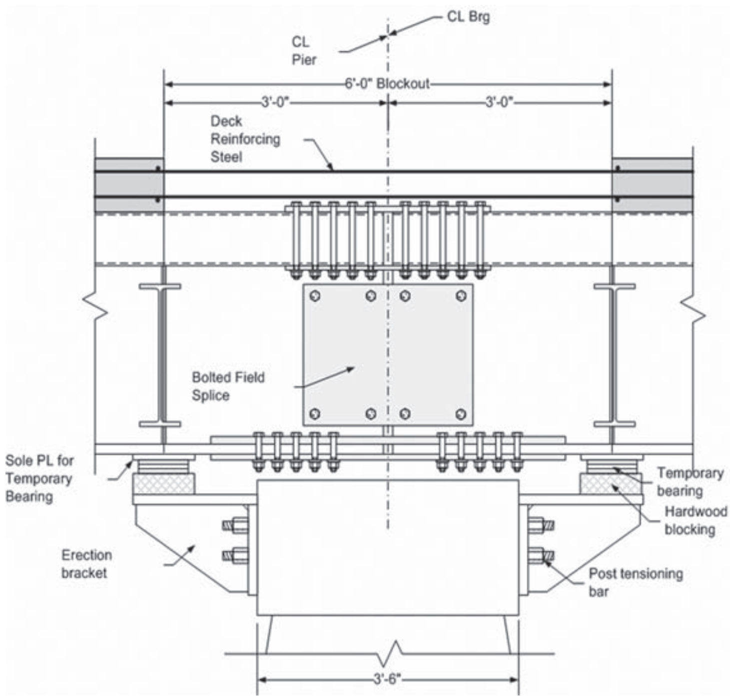

Figure 4. Erection stages 1 and 2.

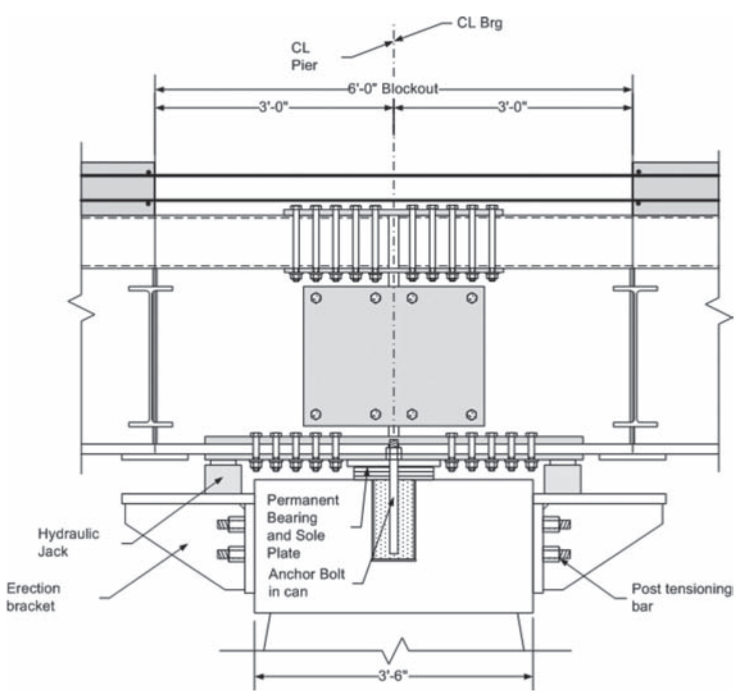

Figure 5. Erection stage 3.

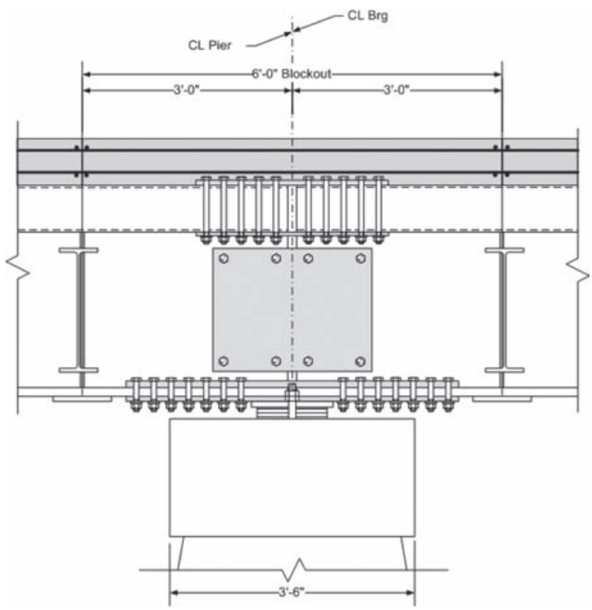

Figure 6. Erection stage 4.

The girders were raised using hydraulic jacks positioned on the erection brackets, Figure 5. The temporary bearings were removed and the CFTFG's were brought to rest on the final bearings. All girders were raised simultaneously so there wouldn't be any undue stress in the diaphragms or the deck.

Finally, the remaining bolts were installed in the bottom flange splice. The anchor bolts were swedged and placed into preformed holes in the pier cap and grouted. The temporary erection brackets were removed. The field-drilled side of the splice was then grouted. After that, the deck blockout regions were placed.

To allow access from the top of the deck to the work area around the splice, the contractor was permitted to form the blockout region at the pier using removable deck forms. The temporary sole plates near the pier were left in place for future bearing replacement. The holes in the pier cap for the temporary erection brackets were then grouted. (Figure 6).

5 CONCLUSIONS

The successful completion of this demonstration bridge project effectively showed how CFTFGs are a viable bridge alternative when erected span by span. It also demonstrated the enhanced performance of CFTFGs over typical plate girders. CFTFGs have increased strength and stability during steel erection and deck placement as verified during construction of the SR 1003 over Tionesta Creek project. These attributes eliminate the need for many lines of diaphragms or cross frames that are needed in typical plate girder bridges. Further, the CFTFG allow engineers to use shallower sections in locations where limited depth is a concern. Erecting CFTFGs as simple spans over obstacles such as traffic lanes result in much safer and faster bridge construction. With fewer diaphragms, the steel erection can proceed more rapidly.

It should be noted that although CFTFGs have several stated advantages over conventional plate girders the cost of the demonstration project came in at $270 per square foot when compared to a similar plate girder costing about $225 per square foot. The cost increase was a direct result of the $4.25 per pound cost of the fabricated structural steel. The higher costs are attributed to this project being a first, and should decrease as more CFTFGs are constructed.

Constructing the field splices at the piers, with the temporary bearings also resulted in a couple of weeks more construction time which are somewhat offsetting when compared with fewer diaphragms and connection plates to fabricate and erect.

ACKNOWLEDGMENTS

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NOTATIONS

A_{tr}—Transformed section area (using the short-term modular ratio) (in^2)
C_b—Moment gradient correction factor
C_{bb}—Moment gradient correction factor corresponding to the unbraced segment under investigation, assuming the adjacent brace points provide perfect bracing
C_{bu}—Moment gradient correction factor corresponding to the girder when it is braced only at the ends of the span (without interior bracing within the span), obtained by applying Equation (6.10.8.2.3-7) to the entire girder span
d—Section depth (in)
D_c—Depth of the web in compression (in)
E—The elastic modulus of steel (29000 ksi)
F_{nc}—Nominal flexural resistance of a compression flange (ksi)
F_y—Specified minimum yield strength (ksi)
F_{yc}—Specified minimum yield strength of a compression flange (ksi)
F_{yt}—Specified minimum yield strength of a tension flange (ksi)
I_{bf}—Moment of inertia of the bottom flange about the vertical axis (in^4)
I_{eff}—Effective vertical axis moment of inertia of the girder to account for singly-symmetric sections (in^4)
I_{tf}—Moment of inertia of the top flange about the vertical axis (in^4)
I_{xx}—Moment of inertia of the section about the primary axis of bending (in^4)
K_T—St. Venant torsional inertia of the transformed section (using the short-term modular ratio) (in^4)
L—Span Length (ft)
L_b—Unbraced length (ft)
M_{br}—Moment including the torsional bracing effect (ft-kips)
M_{cr}—Critical moment corresponding to elastic lateral torsional buckling between braces (ft-kips)
M_{cr}^{br}—Elastic LTB moment including the torsional brace stiffness (ft-kips)
M_{cr}^{ubr}—Elastic LTB moment for the girder without interior bracing within the span (ft-kips)
M_d—Ideal design flexural strength that corresponds to buckling between the brace points (ft-kips)
M_d^{br}—Design flexural strength for torsionally braced CFTFG (ft-kips)
M_n—Nominal flexural resistance (ft-kips)
M_{ncc}^{sc}—Flexural strength based on strain compatibility (ft-kips)

M_s—Cross sectional flexural capacity equal to the yield moment (ft-kips)
M_u—Factored moment (ft-kips)
M_{yc}—Yield moment with respect to the compression flange (ft-kips)
M_y^{sc}—Yield moment based on strain compatibility (ft-kips)
M_{yt}—Yield moment with respect to the tension flange (ft-kips)
M_y^{tr}—Yield moment based on linear elastic transformed section (ft-kips)
n—Number of interior braces within the span
r_y—Radius of gyration (in)
S_{xxbot}—Section modulus about the primary axis of bending corresponding to the bottom flange (in^3)
S_{xxtop}—Section modulus about the primary axis of bending corresponding to the top flange (in^3)
α_s—Strength reduction factor
α_s^{br}—Strength reduction factor for torsionally braced girder
β_T—Effective brace stiffness (kip-in/radian)
ε_{cu}—Maximum usable compressive strain in the concrete infill (in/in)
ε_y—Yield strain (in/in)
ϕf—Resistance factor for flexure

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Chapter 11

The Queens Boulevard Bridge—the NEXT generation

D. Tuckman & M. Deitch
Hardesty & Hanover, LLP, New York, NY, USA

ABSTRACT: This paper and presentation will provide a detailed view of the benefits the Northeast Extreme Tee (NEXT) Beam delivers in the design, fabrication and construction process. Utilizing the Queens Boulevard Bridge over the Van Wyck Expressway as a case study, the paper will highlight the benefits of flexibility in design, speed and ease of both manufacturing and construction. The bridge was designed in 2009 and is currently in the preliminary phase of construction. The paper describes the project background and details the reasons the NEXT beam was selected for Queens Blvd. Situations that are optimal for using the NEXT Beam will be described and the inherent advantages this system realizes in terms of stability and weight (dead load) will be shown. Additionally, the paper will touch on the LRFD Design requirements for the NEXT beam.

1 INTRODUCTION

The Northeast Extreme Tee (NEXT) Beam (Figure 1) is a new beam standard developed by the Precast/Prestressed Concrete Institute Northeast (PCINE) Bridge Technical Committee. Over the past five years the technical committee, which is comprised of the New England States and New York State Department of Transportations, Precast Manufacturers and Consultants, has focused their activity on developing regional standards for use on accelerated bridge construction. Prior to the NEXT beam, adjacent box beams often proposed for 50 to 120 foot spans. However, several details prevented the girders from being erected quickly including transverse post tensioning and shear key grouting. Long term maintenance problems have also been observed in box beam structure including longitudinal cracking in the reinforced concrete deck, positive moment cracking in the beam at locations near interior piers due to creep and shrinkage, and negative moment cracking in the reinforced concrete deck (Weidlinger 2004). The NEXT beam is intended to compete with adjacent box beams in the 50 to 90 feet span range (PCINE 2010). It eliminates the labor intensive details that adjacent box beams require such as transverse post tensioning and grouting shear keys by using a full depth reinforced concrete deck. The full depth reinforced concrete deck also helps prevent the longitudinal cracking observed in adjacent box beams.

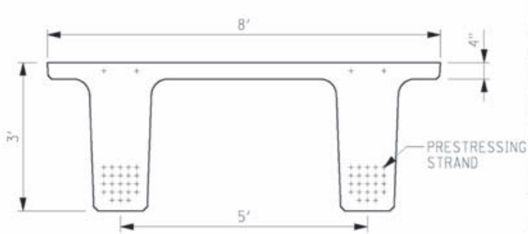
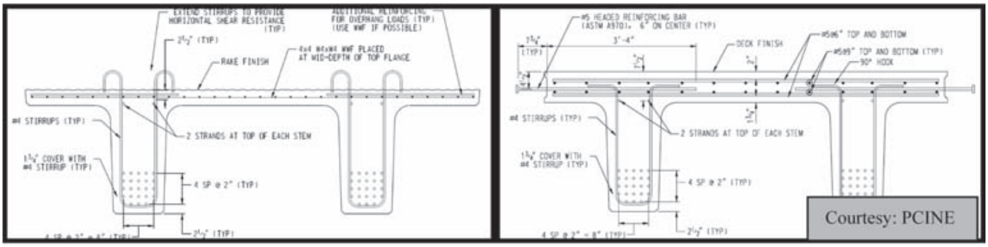


Figure 1. Typical NEXT beam section.



NEXT F Beam (4 inch Flange) NEXT D Beam (8 inch Flange)

Figure 2. NEXT Beam types.

The goals of the new NEXT Beam section are to provide a fast construction option for variable width bridges with spans up to 90 feet. The section resembles a standard double tee, except that the stems are wider in order to handle the moment and shear demand for bridge loadings. There are two different NEXT beam standards; the NEXT F Beam and the NEXT D beam (PCINE 2010). The NEXT F beam has a 4 inch flange intended to provide formwork for a cast-in-place concrete deck. The NEXT D beam has an 8 inch flange that eliminates the need for a reinforced concrete deck, but closure pours are required to connect the adjacent flanges of the beam. The NEXT F and NEXT D have similar maximum span lengths, allowing the engineer to decide on the most appropriate and economical section depending on the unique constraints of each individual project and the needs of the client. Figure 2 shows the NEXT F and D Sections.

The NEXT F section was determined to be the most appropriate section for the Queens Blvd Bridge for reasons discussed in Section 4.1. For the purposes of this paper, the discussion will focus on the NEXT F section, henceforth referred to simply as the NEXT Beam.

The NEXT Beam configuration is resistant to lateral buckling. Additionally, there is approximately 10% savings in dead load with the NEXT Beam configuration compared to an equivalent prestressed concrete adjacent box beam configuration. Notable cost savings can be realized at the substructure from the decreased superstructure dead load.

The NEXT Beam also creates a highly redundant system. If a section of the NEXT Beam is damaged, its load path will be redistributed amongst the adjacent tee beams and will not result in structure failure. Additional advantages are also seen with respect to utility support detailing, since the natural shape of the beam provides for utility bays without requiring any special details, as opposed to using prestressed concrete adjacent box beams which require removing an entire box beam from the framing and using special slab details to span the added bay.

The top flange of the NEXT beam is kept thin and is intended to provide a deck form for a cast-in place concrete deck, thereby saving substantial time during construction. The top flange of the NEXT beam can also be adjusted to a variety of widths ranging from 8’-0” to 12'-0" using magnetic side forms. With this approach, designers can specify precise beam dimensions that can produce site specific bridge beams without significant added cost.

2 PROJECT BACKGROUND

The existing Queens Boulevard Bridge over the Van Wyck Expressway is being replaced to accommodate the operational and geometric improvements on the Van Wyck Expressway. This project is one of four bridges being replaced to provide operational improvement on the Expressway. In addition, the Queens Boulevard profile will be raised to

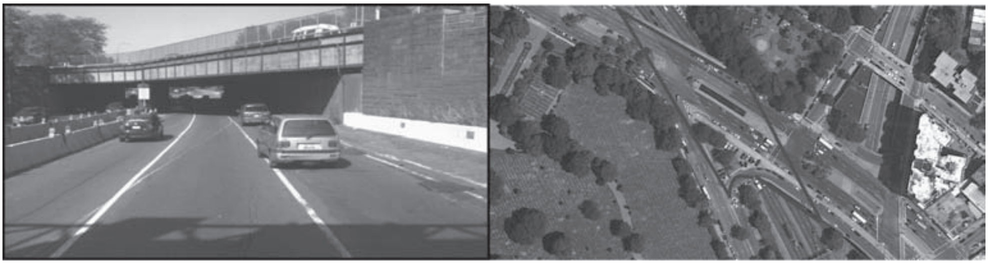


Figure 3. Existing Queens Blvd Bridge over the Van Wyck Expressway (Aerial and Elevation).

accommodate the changes in structure depth, profile and super-elevation on the Van Wyck Expressway.

Constructed in 1953, the existing Queens Boulevard Bridge is classified as an Urban Minor Arterial and has a wide heavily skewed two-span superstructure. The inner bays of the existing superstructure are supported on a series of multiple steel rolled stringers set normal to and continuous over the center pier. Steel through girders run along the outer bays of the framing system to accommodate the roadway curvature and skew. The structure carries three lanes of traffic for both the eastbound and westbound Queens Boulevard, one parking lane in each direction, and a wide median. The bridge crosses over the Van Wyck Expressway just north of Main Street. The Briarwood subway station is located five meters behind the west abutment of the bridge. A pedestrian tunnel providing access the Briarwood subway station originates on the north side of Queens Blvd east of the bridge and crosses beneath the Van Wyck Expressway near the south end of the Queens Blvd Bridge. The existing plan and elevation are shown in Figure 3.

The replacement structure will carry three 12 foot traffic lanes, a 12 foot turning lane, an eight foot parking lane and a 15 foot sidewalk in both eastbound and westbound directions. The existing steel railing will be replaced with concrete parapets.

3 WHY CHOOSE THE *NEXT* BEAM?

Five feasible alternatives were selected for evaluation during the study phase for the replacement of the Queens Boulevard Bridge over the Van Wyck Expressway. Three of the alternatives were discarded after evaluation and will not be thoroughly discussed herein including a Continuous Through Girder (Alternative 1), steel multigirders (Alternative 2), and a single span tied arch (Alternative 3). These were discarded based on community input, long term maintenance concerns, and redundancy issues (Alternatives 1 and 3). The two remaining alternatives were the Adjacent Box Beam (Alternative 4) and the NEXT Beam (Alternative 5), which are discussed in detail below.

3.1 *Alternative 4: Prestressed concrete adjacent box beam*

Alternative 4 consists of a two span adjacent prestressed concrete beam structure with a monolithic concrete deck. The abutments and center pier required for this option will be longer than those required for Alternatives 1 and 3. The required length, however, is mitigated by introducing a skew to the boxes. The relatively lightweight superstructure will allow the foundation to be smaller per unit length than for the other alternatives. Aligning the support beams in this manner greatly reduces the span lengths as well as the required superstructure depth. The favorable span to depth ratio allows for a low profile structure that is less visually intrusive.

The development of this alternative was heavily influenced by the community groups' input. They favored a slender structure which would be less visually intrusive than the other alternatives. The beam and deck layout will run past the outer limits of the sidewalks, creating open areas along the northeast and southwest corners. These open areas offer opportunity for streetscape improvement and parking for the near-by police station. Compared to Alternatives 1 through 3, this option will require minimal or no temporary shoring thus lowering construction costs. Unlike the other alternatives, this option presents a highly redundant system. If a beam is damaged, its load path will be redistributed amongst the adjacent beams and would not result in structure failure.

Although this option would entail a longer underpass for VWE traffic, the aesthetic attributes and highly redundant nature are very favorable. The prestressed concrete adjacent box beam structure was initially recommended as the preferred alternative. After laying out the framing of the proposed structure, further investigation identified a challenge in the framing of the proposed structure. Due to the width of the structure, which was also skewed to the piers and the highway, difficulties arose in providing transverse post-tensioning. This led H&H to look into using another prestressed member with a relatively shallow superstructure depth that did not require transverse post tensioning or deck formwork.

3.2 *Alternative 5: Northeast Extreme Tee Beam (NEXT Beam)—Preferred Alternative*

During the study phase, new information had been brought to the attention of NYSDOT and Hardesty & Hanover when attending the 2008 Concrete Bridge Conference on the development of a double tee pre-cast beam (NEXT Beam) for medium span bridges. The beam depth (maximum 36") and load carrying capacity of the NEXT Beam appeared to be a perfect fit for the Queens Boulevard Bridge over the Van Wyck Expressway superstructure and its geometric constraints.

The NEXT Beam section, developed by PCINE, is an alternate solution for mid-size span bridges (50' to 90' span lengths). The beam is designed to act as a simple span under dead load, and continuous under live load with the use of a closure pour at the pier. The beams inherent T-shape allows for several benefits including eliminating the need for formwork, providing inherent utility bays between the stems, and allowing for an adjustable roadway width. The width of the beam can vary between 8 feet and 12 feet using the same self stressing formwork. Developments in pre-cast beam forming techniques make adjustment of the forms very easy, using magnetically attached side forms which can be adjusted very easily to produce a top flange of any varying width. The use of adjustable forms will be beneficial during stage construction sequences due to the relative ease with which the skew of the superstructure

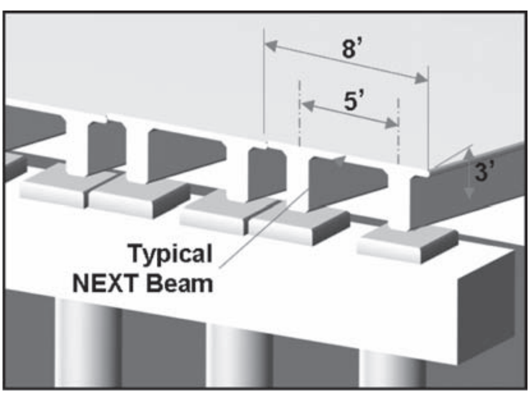


Figure 4. Northeast Extreme Tee Beam (NEXT Beam)—Proposed Section.

and oddly shaped deck areas that will be formed at staging limits will be accommodated. The stem is fixed in a standard double tee form with 5 foot center-to-center spacing, see Figure 4. The adjustable width beams provide the designer with the option of using wider beams to reduce the number of beams and therefore reduce number of bearings and seats.

Rapid construction techniques allow the NEXT to be erected quickly and easily. A major advantage over the adjacent box beam alternative is there are no transverse post tensioning or shear key requirements which are labor intensive and costly. Long term maintenance is reduced because the NEXT beam is a complete concrete structure. Stage construction can accommodated easily with the NEXT beam. The only connection that is required is the closure pour in the reinforced concrete deck.

4 PROPOSED QUEENS BOULEVARD BRIDGE

4.1 *NEXT Beam Framing*

Hardesty & Hanover is the first consulting firm to utilize the NEXT Beam unit for the New York State Department of Transportation (NYSDOT) through the design of the Queens Boulevard Bridge, see Figure 5. The Queens Boulevard Bridge replacement over the Van Wyck Expressway (VWE) utilizes a two span NEXT Beam structure, comprised of 114 NEXT Beams supporting 6 lanes of vehicular traffic with the addition of 1 turning lane, 2 sidewalks and four large plazas for pedestrian use. Aligning the beams no greater than 30 degrees to the abutment face results in average beams lengths of 68 feet to a maximum length of 88 feet for the east and west spans respectively. The thirty degree maximum skew was chosen because it reached the limiting span length for the NEXT beam, and also because skews greater than 30 degrees can complicate details and cause cracking in the top flanges at the ends of the beam. The lengths of the NEXT beam in the east span will vary to avoid encroachment in the park located along the north portion of the east abutment. For the Queens Blvd Bridge, the width was not varied because the stems were required to line up over the pier for the closure pour to make the beams continuous for Live Load.

The NEXT F beam (4 inch flange) was chosen over the NEXT D beam (8 inch flange that acts as deck) for multiple reasons. The beams are framed at a 45 degree skew to traffic traveling on Queens Blvd causing most of the beams to cross over a location where the cross slope changes and in many cases where the cross slope breaks from positive to negative (or vice versa). A cast in place concrete deck allows the haunch, or buildup, on the beam to vary and accommodate the changes in cross slope, whereas a NEXT D beam would be unable

Figure 5. Northeast Extreme Tee Beam (NEXT Beam).

to accommodate this without a variable depth asphalt overlay. In a structure with no skew, this would not be an issue and the NEXT D beam could be considered as a viable alternative. Additionally, there are weight and size restrictions to transport the beams to the project location in Queens. The NEXT D beam weighs an extra 0.4 kip / linear foot than the NEXT F beam because of the full depth precast deck, which limited the length of the beam that could be transported.

The NEXT beam is designed for a maximum compressive strength of 10 ksi. The deck slab will be a fully reinforced 8" slab and will use lightweight high performance concrete (minimum strength of 4 ksi) as approved by NYSDOT to assist with maintaining a light superstructure. The concrete deck will be composite with the tee beam section. The top flange will act as a form for the cast-in-place concrete deck. To enhance the longevity of the deck, NYSDOT has requested the use of stainless steel rebar for the 8" deck reinforcement. The stainless steel rebar has a higher initial cost, but a life cycle analysis revealed that the stainless steel deck reinforcement would reduce costs over a 75 year period.

Diaphragms are typically used in multi-girder bridges to provide better live load distribution and lateral support of the top flange. The NEXT beam configuration provides the same benefits as double tee beams which are extremely stable and resistant to lateral buckling. The cast-in-place concrete deck will provide for the live load transfer between beam stems. The use of intermediate diaphragms will not be required with the NEXT beam configuration. End diaphragms will still be used to support the un-stiffened slab edge at the supports as well as assist with detailing to provide for continuous sections under live load where possible. The end diaphragms can be cast-in-place in the field. The NEXT Beam framing is designed as a simple span beam for dead load, and is made continuous for live load with the installation of a closure pour during construction. The top flange of the NEXT beam is coped to for the first 12 inches to provide easy access to pour the cast-in-place diaphragms as shown in Figure 6.

There will be approximately 10% savings in dead load with the NEXT beam configuration compared to the prestressed concrete box beam configuration as shown in Table 1. An additional superstructure dead load reduction is achieved by use of lightweight concrete for the deck. Cost savings will be realized at the substructure due to the decreased superstructure dead load. In addition, since the NEXT beam width (8') is twice the width of the adjacent box beam (4') which was originally proposed for this structure, crane erection time will be more efficient by allowing for half the number of crane picks.

4.2 *Bridge geometry*

The bridge will be replaced in staged construction, with the new alignment overlapping the existing. The proposed horizontal alignment will match that of the existing. The west

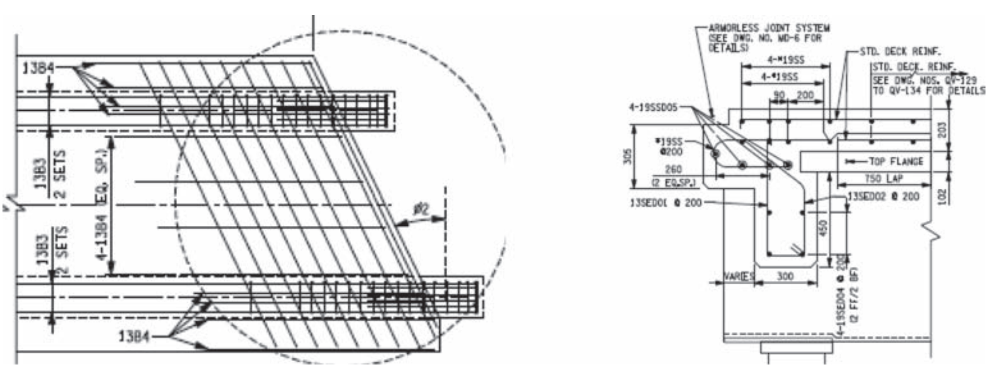


Figure 6. NEXT Beam diaphragm details.

Table 1. Dead Load Comparison.

Beam Type	Depth (in)	Weight k/ft	Deck thickness (in)	Deck weight k/ft	Total weight k/ft
NEXT BEAM (1 beam = 8 ft)	36	1.33	8	0.8	2.13
ADJ. BOX BEAM (2 beam = 8 ft)	39	1.69	6	0.6	2.29

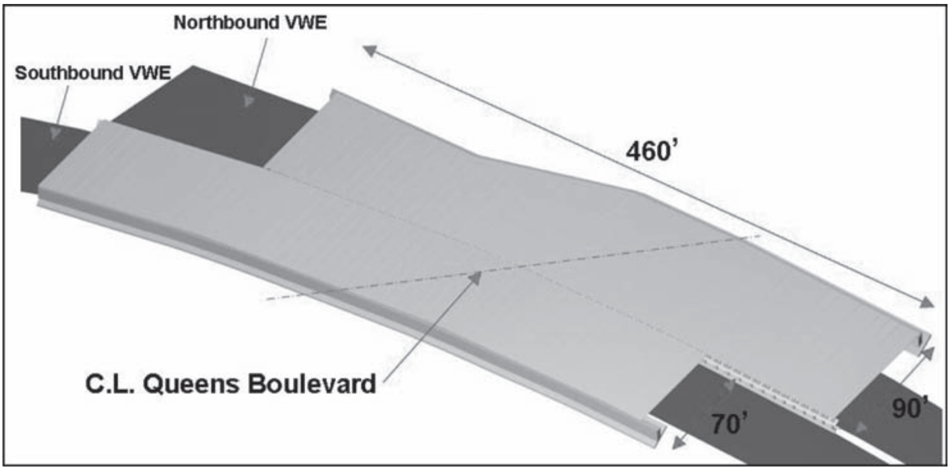


Figure 7. NEXT Beam—Proposed Framing.

abutment will be modified to minimize the potential construction impacts on the subway station located along the west abutment. A new east abutment will be constructed east of the existing abutment to provide the necessary width for the reconfigured Van Wyck Expressway. A new center pier will be constructed east of the existing center pier due to the alignment changes on the Van Wyck Expressway. The substructure will run parallel with the VWE resulting in a very heavy skew of approximately 65 degrees as seen in Figure 7.

4.3 *Alignment restrictions*

The limitations for the development of the proposed vertical alignment were just as restrictive as that for the horizontal alignment. Due to the widening along the Van Wyck Expressway, the Queens Boulevard Bridge superstructure length and depth must be increased. The structure length is amplified more so by the skew. The proposed structure depth is controlled by the vertical clearance requirements below the Queens Boulevard Bridge. The existing Van Wyck Expressway profile cannot be adequately lowered because of the proximity of the subway pedestrian tunnel which runs below the Van Wyck Expressway near the west abutment of the bridge. In addition, the Van Wyck Expressway vertical profile is also controlled by a bridge carrying the expressway on the south approach of this bridge. In order to minimize the effects a profile raise would have upon the neighboring roads, it is imperative that the superstructure depth be kept to a minimum. The NEXT beam 36 inch shallow depth accommodates these geometric restrictions.

4.4 *Utilities*

The Queens Boulevard Bridge over the Van Wyck Expressway carries several utilities including a 12 inch diameter steel water pipe, a 12 inch diameter gas main, and several 3.5 inch

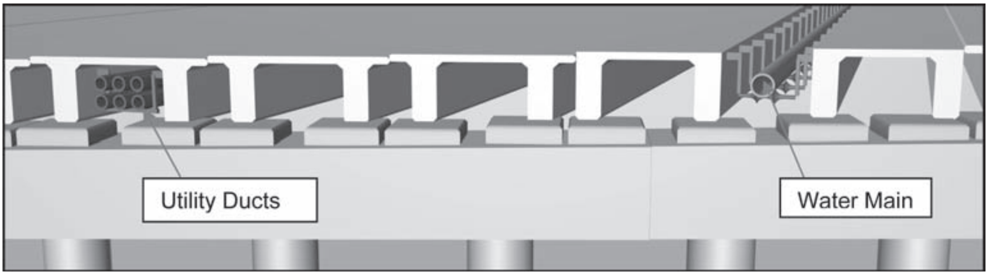


Figure 8. NEXT Beam Utility Supports.

conduits. The vertical clearance requirements on the Van Wyck Expressway limit the structural alternatives to shallow structures that will have difficulty accommodating utilities. The NEXT beam configuration will readily accommodate under bridge utilities much more efficiently compared to other superstructure types. The prestressed concrete box beams would require special details to accommodate utilities since there is typically no space between the beams. Box beams would need to be eliminated to accommodate these under bridge utilities, and the deck would span between boxes and require additional formwork which lessens one of the main benefits of the concrete box beams. The NEXT beam system is able to support several utilities between the stems. Inserts will be used during the pre-casting phase to accommodate utility support connections. Figure 8 shows typical utility configurations.

New York City Department of Environmental Protection (NYCDEP) requires that the steel water main be pre-assembled prior to installation, and lowered into position from above the roadway surface. A special detail is used on Queens Boulevard to allow for this installation by forming the NEXT Beam flange to end at the outer face of each stem. Two of these special beams are positioned to form a "gap" at the roadway surface to allow for the steel water main to be lowered into position. Stay-in-place forms will be used for this bay. Since a fully reinforced 8" deck is used with the NEXT Beam design, there is no special deck detail needed. Also, with the adjustable top flange form using magnetic rails, the same NEXT Beam form is used to fabricate this special beam.

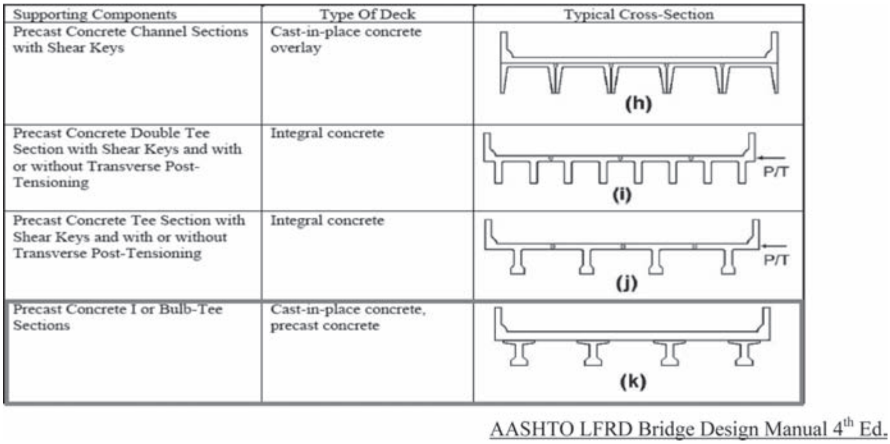
5 DESIGN GUIDELINES

5.1 *Structural analysis*

The NEXT beam should be designed in accordance with the procedures in AASHTO LRFD Bridge Design Specifications. The live load analysis can be completed either using the approximate methods described in AASHTO 4.6.2, or a refined method such as a finite element analysis as described in AASHTO 4.6.3 (AASHTO 2007). If using the approximate method, the appropriate deck superstructure type to use from Figure 9 is Cross Section K.

The refined method was used to design the Queens Blvd Bridge because of the heavy skew of the traffic to the structure. A SAP2000 finite element model of the superstructure and substructure was created to obtain the controlling forces and moments. This model was also used for the seismic analysis.

The SAP2000 Model is a full scale FEM structural analytical model. The superstructure was modeled (Figure 10) using the bridge module with seven total lanes on the bridge. Deck shells were used to model the composite reinforced concrete deck. The NEXT double tees were modeled using linear elements meshed at the tenth points along the beam. Two bearings support each end of each beam were modeled using link elements. Rigid elements are used to connect the link bearings to the NEXT Beam member. Two additional rigid elements frame from the member to the composite deck to accurately model the 5 foot spacing between



Supporting Components	Type Of Deck	Typical Cross-Section
Precast Concrete Channel Sections with Shear Keys	Cast-in-place concrete overlay	(h)
Precast Concrete Double Tee Section with Shear Keys and with or without Transverse Post-Tensioning	Integral concrete	P/T (i)
Precast Concrete Tee Section with Shear Keys and with or without Transverse Post-Tensioning	Integral concrete	P/T (j)
Precast Concrete I or Bulb-Tee Sections	Cast-in-place concrete, precast concrete	(k)

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Figure 9. AASHTO Superstructure types for distribution factor determination.

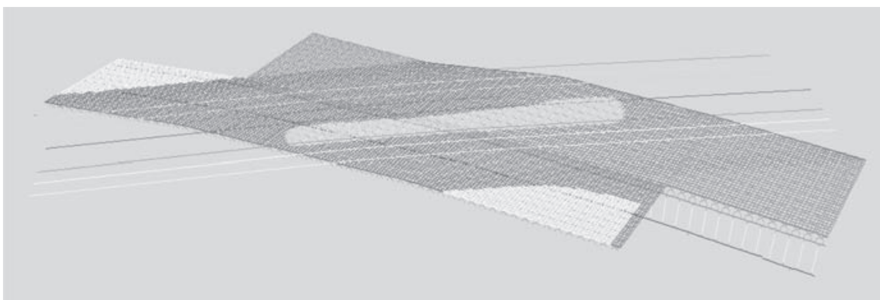

Figure 10. Queens Blvd Bridge SAP2000 finite element model.

stems. Additional loadings are applied above the deck shells to account for pedestrian loading, parking lanes, and superimposed dead load. Live Load was applied to the structure in accordance with the requirements of AASHTO LRFD Section 3.6. The SAP model was able to account for the skew of the beams in relationship to the direction of traffic and the stiffness of the reinforced concrete deck, thereby more accurately computing the maximum design moments and shears that each beam carries in comparison to using the 'approximate method' provisions in AASHTO 4.6.2 (AASHTO 2007).

The structure was designed to be simply supported for dead load and continuous for live load. Live load continuity increases structural efficiency and eliminates the joint at the center pier. Eliminating joints has the advantage of reducing future maintenance efforts at the joints and reduces the likelihood of water reaching the bearings in these locations. A closure pour is used to make the beams continuous over the center pier. The members that frame the triangular plaza areas in the northwest and southeast quadrants of the structure are designed to be simply supported for dead and live load because there are not beams in the opposing spans in these areas.

5.2 *Section properties*

The NEXT beam cross section can vary in width between 8 ft and 12 ft depending on the requirements of the project. For Queens Blvd Bridge, an 8 ft section was used with a composite reinforced concrete deck. The section properties of a typical beam are listed below in Figure 11.

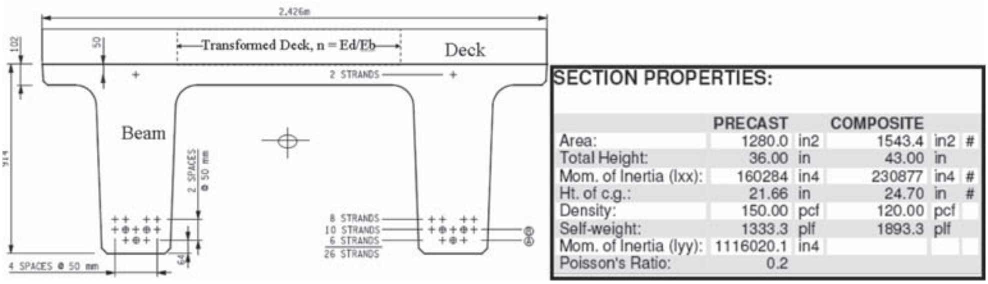

Figure 11. Queens Blvd Bridge NEXT Beam section properties.

The maximum number of prestressed strands in the cross section is 50. Up to 46 strands may be placed in the two webs, and up to four strands may be used near the top of the cross section to support reinforcement and control end stresses.

5.3 *Limit states*

The NEXT beams, like other prestressed concrete members, are designed to meet the limit states in AASHTO 3.4.1 (AASHTO). The main limit states considered are Strength I, the basic load combination relating to Normal Vehicular Use, Service I, the Load Combination that checks compression in prestressed concrete components, and Service III, the Load Combination for longitudinal analysis relating to tension in prestressed concrete superstructure, and Fatigue. Strength II is an optional limit state that most agencies require relating to Permit Vehicles.

5.4 *Controlling temporary stresses*

Temporary tensile stresses are governed by the limits in AASHTO 5.9.4.1.2 (AASHTO 2007) listed below.

In areas other than the precompressed tensile zone and without bonded reinforcement:	$0.0948\sqrt{f'_c} \leq 0.200(ksi)$
In areas with bonded reinforcement sufficient to resist the tensile force in the concrete:	$0.24\sqrt{f'_c}$

Temporary Compressive stresses are governed by the limits in AASHTO 5.9.4.1.1 (AASHTO 2007) and is limited to 0.60*f'ci, where f'ci is the compressive strength of concrete at prestressing. When a designer is trying to maximize the efficiency of a beam, debonding can be used to control temporary stresses near the ends of the beam.

When utilizing debonding to control stresses, the following guidelines should be followed. No more than 25% of the total number of strands should be debonded with a maximum of 40% of the strands in a row. No more than 4 strands should be debonded at the same longitudinal location to avoid stress concentrations. The debonded strands should be distributed symmetrically about the centerline of the member and exterior strands in a row should not be debonded (AASHTO 2010). These guidelines limit flexural stresses without sacrificing the integrity of the beam.

Debonding 50 percent of the strands for the first 6 inches is recommended to control cracking. The limits above do not apply for the first six inches. Draped strands are not permitted for the NEXT Beam because it would require placing holes in the bottom of the self-stressing formwork to place the tie-downs (PCINE 2010).

5.5 *Service stresses*

Service stresses are governed by Service I and Service III for compression and tension respectively. The tensile service limit generally governs at midspan for the NEXT beam but both should be checked. The tensile stress limit for structures in the northeast should be [0.0948*sqrt(f'c)] (ksi) (AASHTO 2007) because of the heavy use of deicing salts.

5.6 *Strength design*

The NEXT beam needs to be checked in the strength limit state. The nominal moment capacity can be calculated in from the provisions in AASHTO 5.7.3.1 (AASHTO 2007). At the maximum span, the strength limit state often will not control the design of the NEXT beams but it should always be checked.

6 CONCLUSIONS

The design of the NEXT beam system offers typical design and fabrication details which will result in a high degree of efficiency. This option will require minimal or no temporary shoring, resulting in lower construction costs. This option creates a highly redundant system. If a section of the NEXT beam is damaged, its load path will be redistributed amongst the adjacent tee beams and will not result in structure failure. The development of this superstructure also maintains the slender structure appearance which was favored by the community groups' input, to be less visually intrusive than other alternatives. With these favorable attributes, the newly developed NEXT beam was chosen as the preferred alternative for the Queens Boulevard Bridge superstructure. The structure is currently in the preliminary stages of construction with a scheduled completion date of 2013. The NEXT beams are scheduled to be fabricated next year and will be installed using staged construction during 2012. Future work will describe construction means and methods and contain construction photographs.

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Consultant	Prestressed/Precast Concrete Institute Northeast (PCINE)
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Chapter 12

New tools for inspection of gusset plates

C. Higgins & O.T. Turan
School of Civil and Construction Engineering, Oregon State University, Corvallis, OR, USA

Q. Nguyen
School of Civil Engineering, Purdue University, West Lafayette IN, USA
Formerly School of Civil and Construction Engineering, Oregon State University, Corvallis, OR, USA

ABSTRACT: After the collapse of the I-35 W Bridge in Minneapolis, Minnesota, gusset plate connections now must be evaluated by transportation agencies. Connection evaluations require accurate as-built drawings. A new methodology has been developed that permits rapid collection of accurate field measurements of connection plate geometry. The method uses close-range photogrammetry techniques to rectify field-collected digital images. Consumer-grade cameras are used to produce scaled orthographic photographs (orthophotos) of the bridge connections. The approach enables rapid and accurate collection of field measurements and allows for direct assessment of the connection using the high fidelity results. Implementation of the techniques does not require specialized knowledge and it has been practicably employed under field conditions using current technology and personnel.

Keywords: steel, truss bridges, gusset plates, photogrammetry

1 INTRODUCTION AND BACKGROUND

1.1 *Introduction*

Transportation agencies in the United States must now evaluate their inventory of gusset plate connections due to the collapse of the I-35 W Bridge in Minnesota. There are approximately 465 steel deck truss bridges and 12,600 other truss bridges within the National Bridge Inventory (NTSB Recommendations 2008). These bridges are undergoing additional scrutiny because load paths are typically non-redundant, thus failure in a truss member or connection may cause the structure to collapse. Connection evaluations require complete and accurate as-built drawing sets and condition reports. Current methods to measure, collect, and archive field data are time consuming and subject to errors at all stages. Additionally, field data will need to be archived to monitor and evaluate changes over the life of the structure. Sketches, notes, and *qualitative* photographic images are not be sufficient to definitively identify time-dependent changes.

Tools that can effectively capture field data to provide evaluation inputs hasten the complex and time consuming task of steel truss bridge evaluations. This paper reports on methods to create orthographic digital photographs (orthophotos) that enable metrification of steel truss bridge gusset plate connections. Data extraction from the images can be directly ported to CAD and finite element analyses (FEA) to determine connection ratings. These combined techniques enable rapid and accurate quantitative field geometry acquisition and evaluation of connections. Integration of field data collection and analysis tasks further streamlines bridge management efforts.

1.2 *Background*

Close-range photogrammetry for metrification has been used previously by many researchers (Heuvel 1998, Criminisi 2000, Arias et al., 2004, Mills & Barber 2004, Chandler et al., 2005, Tommaselli et al., 2005, Jáuregui et al., 2006, Rodríguez et al., 2008). The present study differs from previous work because for evaluation of truss bridges, photographs cannot easily be taken from a mounted stationary position. In the field, photographs of bridge gusset plates will likely be taken from a snooper with both the snooper and bridge in motion from wind and traffic, or by climbing on the structure, and thus it will not be feasible to obtain stationary positions to correlate stereo or multi-station images.

Photographs capture a real world image and place it on a two-dimensional image plane. When an image is captured with a camera and lens, it generally contains perspective distortion (parallel lines converging at a finite point), as well as other distortions due to the lens characteristics (such as barrel distortion or pin-cushion or other aberrations). To remove perspective, the image can be rectified using a mathematical transformation which maps elements in the real world image to those in the photographic image plane. Removal of barrel distortion or pin-cushion for some lenses requires lens correction parameters applied during post-process images. Alternatively, flat field lenses can be deployed that do not have these features.

In the present case, barrel distortion and pin-cushion are minimized by using lenses that minimize these distortions. Since gusset plates are typically flat, relationships between their features in a two dimensional plane can be used to simplify the process. One of the most common techniques for image rectification is the direct linear transformation (DLT) algorithm. This transformation requires that certain geometrical characteristics be established between in the real world image and the image plane so that the image can be rectified. In the present case, point correspondences are used to map points between the real world image and the photographic image plane based on central projection shown in Figure 1. Central projection maps the common points between planes and preserves lines in both planes.

It is possible to establish the geometrical features to provide a true dimensional scale such that the resulting transformation is not only rectified, it is scaled to the real world dimensions. This enables metrification from the rectified image. For implementation of the DLT algorithm, the image plane and real world planes are assumed to be homogeneous. A transformation matrix, H, is determined to satisfy the following equation for each control point, i:

$$H\begin{bmatrix} X_i' \\ Y_i' \\ 1 \end{bmatrix} = \begin{bmatrix} X_i \\ Y_i \\ 1 \end{bmatrix} \tag{1}$$

where X' and Y' are photograph image plane control point coordinates, X and Y are real world control point coordinates, and H is a 3×3 matrix. To solve for the transformation matrix H,

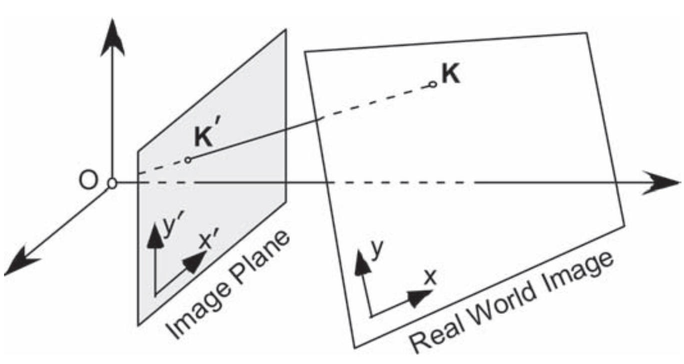

Figure 1. Correspondences between photographic image plane and real world image.

known control points need to be established in the real world image that can be captured in the image plane. A minimum of four control points are needed (x and y coordinates each provide a degree-of freedom (DOF) for a total of eight DOFs and one more is provided by scale, thereby providing nine DOFs to solve for the matrix). If more points are available, error estimations can be made. The method generates a two-dimensional transformation based solely on the control point coordinates, thus, camera parameters are not needed. Such DLT algorithms are used in commercial photogrammetric software.

2 IMPLEMENTATION

2.1 *Image targets*

In the present work, reference targets were developed that can establish control points on the gusset plate image. These targets establish nine (9) control points which provide more image constraints than the minimum required to solve for the transformation matrix, thereby enabling error estimation. The control points are ideally spread throughout the image. The reference target control points standoff from the surface of the gusset plate, which is necessary for clearance over bolt and rivet heads, but this standoff distance causes the reference target to appear larger in the photograph than if it were to lie explicitly on the gusset plate surface. The offset can be corrected assuming the camera is a single point and using similar triangles to calculate the apparent size increase of the reference target compared to if it were to lie on the gusset plate surface. The standoff distance for the reference targets used in this study is approximately 51 mm (2 in.) from the gusset plate surface. The actual lengths of the gusset plate features including the offset correction can be calculated as:

$$L_{actual} = \frac{D}{D - D_o} L_{image} \tag{2}$$

where D is the distance from the camera to the gusset plate surface, D_o is the target standoff distance, and L_{image} is the uncorrected length measured in the image. Dimensions taken from the rectified image without standoff correction will be smaller than the true dimensions. If the camera location if very far away from the target, the correction becomes small (e.g. correction is less than 5% for camera located 1.5 m (5 ft) from the gusset plate).

To rapidly establish the known control points in the real world image, two reference targets were developed. One is relatively small with 203 mm (8 in.) arms and the other is relatively large with 610 mm (24 in.) arms. The small reference target is used for smaller gusset plates or for splice plates between chord members. The large reference target is used for most typical large gusset plate connections. The targets use 12.7 mm (0.5 in.) square Ultra Corrosion-Resistant Pure Titanium Grade 2 bars, nine (9) aluminum discs that serve as the control points, and an aluminum encased round ceramic magnet. The titanium bars are placed in a cruciform shape. The nine aluminum discs are equally spaced along the titanium bars. The aluminum cylinder base is bolted along with the ceramic magnet to the center of the bars. Titanium was used for the arms of the reference target due to the corrosion resistant properties, high strength-to-weight ratio, and low coefficient of thermal expansion. Aluminum was selected for its lightweight and corrosion resistant properties. The magnetic base allows the reference target to be held securely on the gusset plate surface, even with many layers of paint, to be easily reused, and to leave the gusset plate undamaged. The large reference target is shown in Figure 2.

2.2 *Images and image processing*

The image rectification process was written using MATLAB and deploys the Image Processing Toolbox within MATLAB. A digital image of a gusset plate with the reference target is taken. The gusset plate digital photograph is loaded into the program and the

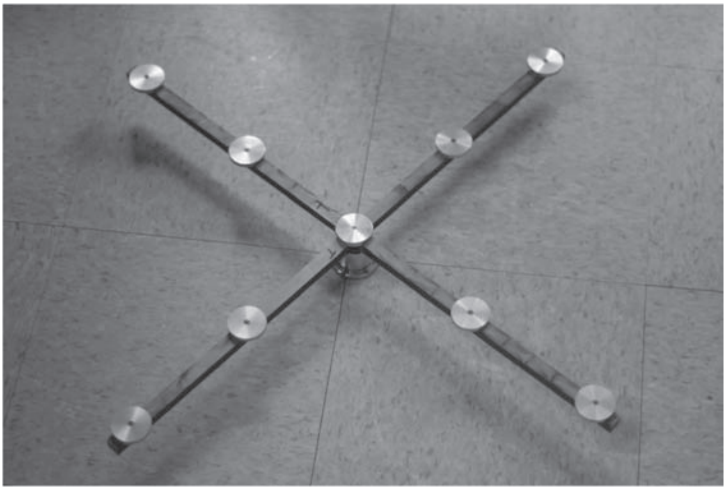

Figure 2. Large image target for establishing known control point in gusset plate image.

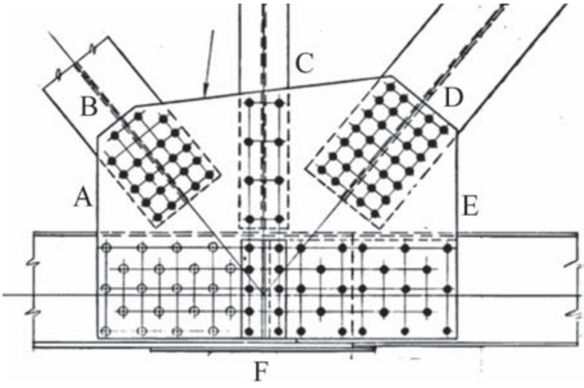

Figure 3. Design drawing of prototype connection to demonstrate methods. (rotated 180º for laboratory convenience).

image pixels of the reference targets are selected by the user. These establish the known real world target coordinates for the DLT algorithm. As part of the processing, an origin is established at the center of the target. The rectified image is presented to the user for validation and then stored for future reference. After processing, a user can query the image to extract geometric data of interest, because the methodology has converted the pixel size to a true-scale dimension (e.g. inch).

To demonstrate the technique, a mock gusset plate was developed. The gusset plate was modeled after a true gusset plate connection on southbound I-5 in Oregon. The bridge is a simple span steel deck truss bridge designed in 1967. Connection U3 was selected for the present study due to the nonsymmetrical design and is shown in Figure 3. The mock gusset plate was made of plywood and has 109—25 mm (1 in.) diameter A325 structural hex-head bolts. Thin steel plates were bonded to the surface of the plywood to enable attachment of the magnetic reference targets to the mock gusset plate. The mock gusset plate was painted grey and the bolts were installed. The edge dimensions of the gusset plate are shown in Table 1.

Images were captured with a digital single-lens reflex (SLR) camera. The camera resolution was 10.9 Megapixel (3872 $\times$ 2592). The lens was an autofocus fixed 50 mm f1.8D lens.

Table 1. Outside dimensions of mock gusset plate.

Side	Dimension cm	Dimension inches
A	87.6	34.5
B	20.1	7.9
C	111.1	43.8
D	36.4	14.3
E	90.2	35.5
F	154.9	61.0

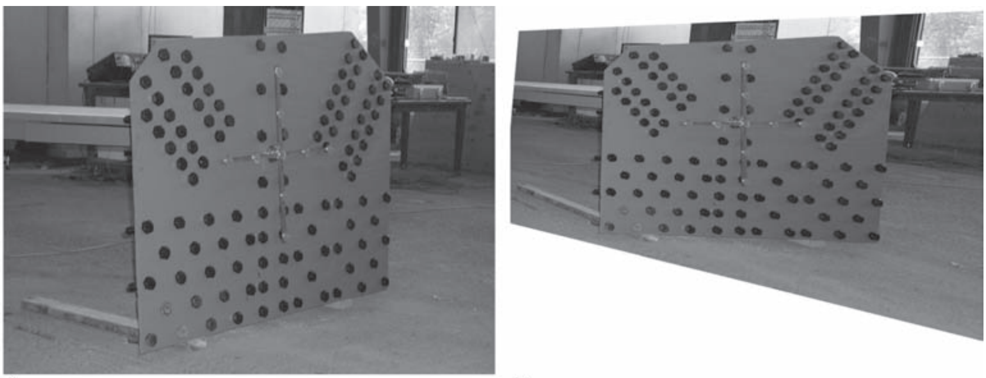
a) b)

Figure 4. a) Original image of mock gusset plate; b) Rectified metric orthograph.

The non-metric digital camera was not calibrated. Field inspectors will likely be using a variety of digital cameras with various lenses and zoom capability and these cameras will likely not be mounted on a tripod or stationary platform while photographs are taken. Controlled calibration may not be feasible or transferable to field applications and thus, cameras and the techniques deployed must be compatible with hand-held operation under realistic field conditions. To simulate such conditions, several different images of the mock gusset plate were taken. Camera stand-off distances from the gusset plate were measured with a Leica DistoTM Pro[4] laser distance meter. The perspective angle of the camera view from the plane of the gusset plate was measured, and the different camera and lens settings were recorded.

A series of images were used to demonstrate the rectification and metrification methodology. The actual measured gusset plate geometry was compared to that measured from the orthophotos. An example image and the corresponding rectified orthograph are shown in Figures 4a to 4b, respectively. This image has strong perspective so as to highlight the features of the technique. Even better results can be obtained if the original image is better composed by taking pictures with the camera more orthogonal to the plate. Using the rectification techniques and the offset correction (Eqn. 2), measurements of the outer dimensions on the gusset plate were taken and compared with the known dimensions. Measurement errors were determined and the largest percent error were found for sides B and D because they are the smallest measurements and the largest absolute error values were found for side F because it is the largest measurement. The maximum error was 3.4 mm (0.14 in.). This is relatively small and is as good as or better than could be expected using traditional methods. More importantly, the member angles, number of bolts, and overall proportion are properly captured for comparison with available design/construction drawings.

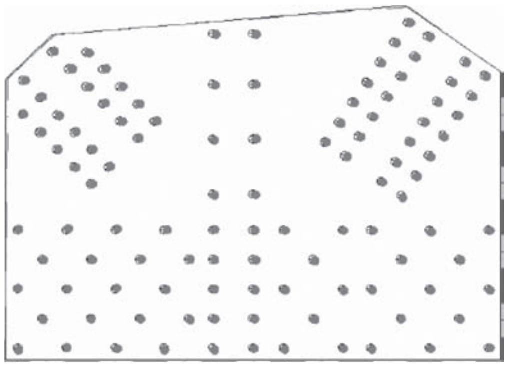

Figure 5. Measured and expected geometric locations of bolts and plate edges.

Using the query options developed in the image processing software developed for this project, the plate boundaries and individual bolt locations were identified (closed circles and dashed lines) and these are shown in Figure 5. The expected plate and fastener geometry is shown underneath (open circles and solid lines). As seen in Figure 5, the geometric properties are well captured by the technique. In addition, the overall capture of the geometric information requires less than five minutes compared to hours with manual methods.

3 CONCLUSIONS

A methodology was developed that permits rectification and metrification of digital images of steel truss gusset plate connections. The approach enables rapid and accurate collection of field geometric measurements, as compared to traditional methods to better inform structural evaluations of gusset plate connections. Software was developed that enables an operator to quickly extract dimensional information from the scaled orthographic photograph (orthophoto). This limits data entry errors and redundant efforts. The availability of scaled orthophotos provides a useful record of field conditions and can be compared with subsequent field inspection results to help identify and quantify long term changes in visual characteristics. Further, scaled orthophotos enables comparison between available drawing sets and as-built details. The implementation procedure is straightforward and does not require specialized knowledge of photogrammetry. It can be practicably employed under field conditions using current technology and personnel. Dimensional measurements from the scaled orthophotos provide results that are as good as or better than conventional field measurements and provide tolerances below what most engineers would find reasonable for gusset plate connection capacity evaluations. The digital gusset plate geometric information can then be used for rapid finite element analysis or production of CAD drawings. These combined techniques enable faster and more refined evaluation of gusset plate connections.

ACKNOWLEDGEMENTS

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4 Bridge rehabilitation and retrofit

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Chapter 13

Delaware River Turnpike Bridge—riveted connection analysis and retrofit

R.J. Schaefer
HNTB Corporation, Wayne, NJ, USA

B.D. Buckman
HNTB Corporation, New York, NY, USA

W. Scott Johnsen & J. Laird
New Jersey Turnpike Authority, Woodbridge, NJ, USA

ABSTRACT: This paper centers on addressing the concerns often noted with riveted fasteners in older steel bridges. It discusses the approach to a recent construction project to perform large scale rivet replacement operations on a circa 1953 steel riveted truss viaduct structure. The content of this paper addresses each aspect of the project's development and execution. The methodology behind the project is discussed, which was based from established research in the field including subject matter limitations and relevance of available design code doctrine for rivet analysis and expected fastener strengths. It is explained how anecdotal construction practices for removal and replacement of rivets were formalized into a coherent plan for developing a scope of contract work. Development of a rivet testing program to establish project specific criteria and protocols to standardize methods of selecting the test rivet population is defined. Strategies were generated to allow for scope of work adjustments based on better or worse than expected rivet strength test results. Finally, the development and evolution of rivet replacement procedures using both mechanical and thermal methods is explained with the benefits and caveats associated with each method. Production rate, accessibility, hazardous lead paint handling, and quality control issues are also covered.

1 INTRODUCTION

The Delaware River Turnpike Bridge is a 6,571 foot long viaduct over the Delaware River with 14 open deck truss spans of 245 feet in average length, flanking a central three span arch truss arrangement with a center suspended span. The bridge was constructed in 1954 and has been kept in good condition by its owners, the New Jersey Turnpike Authority (NJTA) and the Pennsylvania Turnpike Commission, via numerous maintenance and capital improvement projects.

Following the collapse of the Minnesota I-35 bridge on August 1, 2007, truss bridges and their gusset plate connections around the nation became suspect. A detailed evaluation of this structure was promptly directed by the NJTA. Subsequent analysis indicated inadequate connector (rivet) strength as a structure wide concern. A retrofit contract was immediately commissioned by the bridge owners to correct this condition. This paper discusses the approach to the project and is centralized around the large scale rivet replacement, which became a primary focus of the work.

The content of this paper addresses each aspect of the project and its development. Review of existing limited rivet behavior and strength research and analysis methods to determine the strength of the bridge truss gusset plate connections are covered, including their limitations and relevance of available design code doctrine for rivet analysis. Strategies for developing a scope of contract work based on estimated rivet strengths are outlined along with strategies for scope of work adjustments based on better or worse than expected rivet test results. The development and execution of the rivet testing program is explained, including methods of selecting the test rivet population, modification of accepted testing methods to accommodate unthreaded fasteners, and limitations of the testing procedures. The development and evolution of rivet replacement procedures using both mechanical and thermal methods is explained with the benefits and caveats associated with each method. Production rate, accessibility, hazardous lead paint handling, and quality control issues are also covered.

2 DESCRIPTION OF STRUCTURE

The subject structure was constructed in 1954 under Contract PN 4. Morgan Proctor Mueser and Rutledge Consulting Engineers provided design services, and construction was executed by American Bridge. The 31 span bridge viaduct spans over the Delaware River and touches down in Bucks County, Pennsylvania and Burlington County, New Jersey. Fourteen of the approach spans were designed and constructed as simply supported parallel girder and concrete deck spans and are not discussed as a part of this paper. The remaining central seventeen spans were designed as riveted trusses and are the subject of this paper (See Figure 1). Fourteen of these spans are riveted deck truss type structures arranged into two three-span continuous units and two four-span continuous units. The main river span unit is also a riveted truss structure, but it is arranged into flanking 341' spans and a central 682' long arch truss with a suspended deck over the navigable waterway channel of the Delaware River. Conservation of materials is suspected to have been a concern at the time of design,

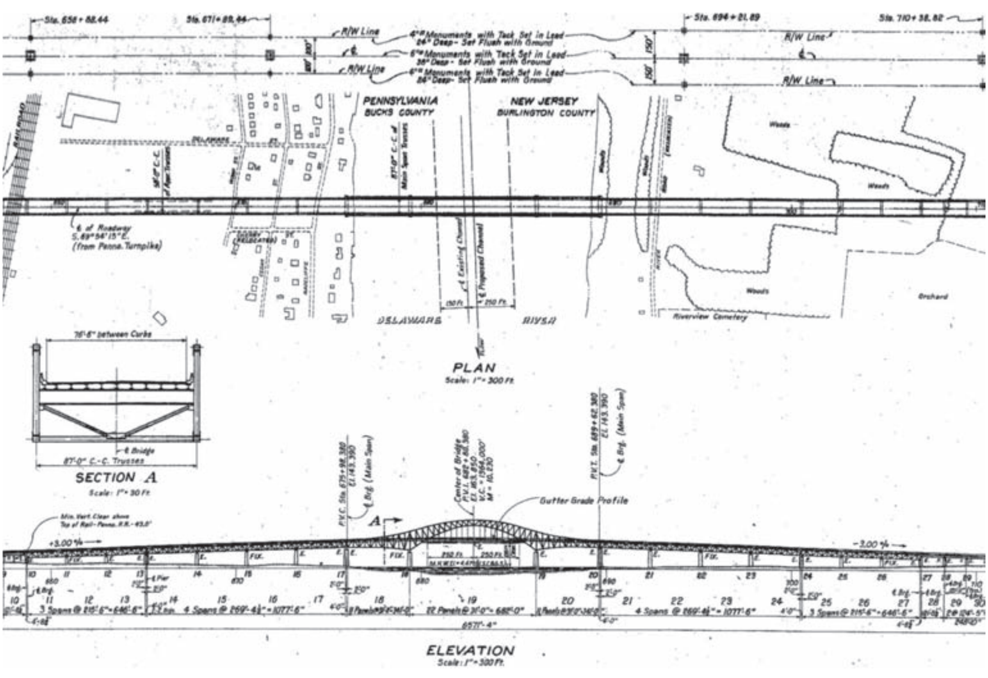

Figure 1. General plan and elevation of structure.

as the truss units are detailed using a variety of predominantly 14" deep wide flange sections fashioned into a simple four chord box truss arrangement.

3 MOTIVATION FOR PROJECT

Preliminary conclusions of the National Transportation Safety Board (NTSB) forensic analysis of the Minnesota I-35 bridge collapse were made available to AASHTO members on January 15, 2008. These conclusions suggested that deficiencies in truss gusset plates were likely contributors to the bride collapse. Two days later on January 17, the New Jersey Turnpike Authority ordered its general engineering consultant to immediately evaluate gusset plate design and the condition of all its major bridges with gusset plate connections.

Subsequent initial analysis of the Delaware River Bridge indicated that 60 years of cumulative improvements had placed additional load on the truss superstructure. These improvements included a 2006 replacement of the original 6¾" thick concrete bridge deck and a 2" thick asphalt overlay with a modern 8¾" thick HPC concrete deck and upgrading the bridge with modern impact rated parapet shapes and median barrier. These improvements slowly but cumulatively increased the bridge dead load by over 30 percent. Based on the conclusions of the initial analysis, the bridge owner commissioned an in-depth evaluation of the structural adequacy of the Delaware River Bridge truss elements.

4 RESULTS OF EVALUATION

Fully detailed gusset plate load rating analyses were performed for the Delaware River Turnpike Bridge based on the FHWA *Load Rating Guidance and Examples for Bolted and Riveted Gusset Plates in Truss Bridges* (FHWA 2009). Per the New Jersey Turnpike Authority *Design Manual* (NJTA 2009), the structure was evaluated using Load Factor Rating methodology as it was originally designed using the AASHTO *Standard Specifications for Highway Bridges* (AASHTO 2002).

Initial impressions from guidance documentation suggested that inadequacies within the gusset plate design with respect to local buckling of gusset plate edges or block shear/Whitmore Section failure would be the dominant modes of failure. However, analysis indicated that a significant number of the gusset plate connections were limited by gusset plate connection rivet group shear capacity. This conclusion was based upon current AASHTO/FHWA doctrine, which indicates that the effective design shear capacity of a high strength rivet may be assumed as 30 ksi. Further investigation of research literature and historical guidance documents regarding the strength of rivets was performed to validate this assumption. The results of this investigation suggested that factored shear resistance values presented in the AASHTO/FHWA provisions may be overly conservative for the high strength rivets used on the bridge. The summation of this research is presented below.

5 THEORY OF RIVETED FASTENERS

As per the original superstructure contract drawings prepared in 1954, the rivets used in truss connections were designated as “High Strength Rivets”. According to the 1953 AASHTO *Standard Specifications for Highway Bridges* (AASHTO 1953), the governing specification listed on the contract plans, high strength rivet steel was to be in conformance with the ASTM A195 material specification. Criteria in the A195 specification published the minimum yield strength of the rivet steel as 38 ksi. Ultimate tensile strength was published as between 68 ksi and 82 ksi. For historical reference, the ASTM A195 designation was withdrawn in 1966 and replaced with the A502 Grade 2 designation.

Continued research indicated ultimate strengths of the subject rivets were typically tested at the higher end of the 68 ksi to 82 ksi range. It was noted that during the design of the original San Francisco-Oakland Bay Bridge (Davis et al., 1940), extensive material testing of undriven high strength rivets indicated average tensile yield and ultimate strengths of 54.5 ksi and 81.0 ksi, respectively. Testing at the University of Illinois at Urban-Champaign (Wilson & Thomas 1938) found similar strengths for undriven rivets as the Bay Bridge testing program. Tensile tests of driven rivets presented in the Illinois report demonstrated a significant increase in strength over the undriven rivets. For rivet stock with an average tensile strength of 79.5 ksi, tensile strengths ranging from 86 ksi to over 100 ksi were measured for driven rivets. The *Guide to Design Criteria for Bolted and Riveted Joints* (Kulak et al., 1987) also notes that a driven rivet's tensile strength may exceed the tensile strength of an undriven rivet by 10 to 20 percent. Therefore, design provisions based on the strength of undriven rivets should be inherently conservative.

Kulak (Kulak et al., 1987) further indicates that shear strengths of driven A502 Grade 2 can be expected to be between 65 ksi and 80 ksi. Shear strength tests of un-driven rivets in the Davis and Wilson testing programs agree with Kulak's values. In Davis (Davis et al., 1940), rivet shear strengths were typically between 70 ksi and 80 ksi for driven high strength rivets. Tests by Wilson (Wilson & Thomas 1938) also found shear strength of driven high strength rivets to be between 70 ksi and 80 ksi in double shear tests. In the same program, average shear strength for undriven rivets tested in double shear was found to be 54.5 ksi.

For mechanical fasteners, ultimate shear stress is typically expressed as a fraction of the ultimate tensile stress. Kulak (Kulak et al., 1987) indicates that the average shear to tensile strength ratio for rivets, based on test results, is about 0.75 and uses this value as the basis of his design recommendations.

6 CURRENT CODE PROVISIONS

Current AASHTO Load Factor Design (LFD) and Load Factor Rating (LFR) provisions (2002, 2008) specify a factored shear strength, φF, of 30 ksi for A502 Grade 2 rivets, which is conservative when viewed against the standing research documentation. Therefore, closer examination of the code provisions was justified to explain the apparent conservatism.

It was found that rivet strength in code provisions was based on an assumed ultimate tensile stress of 68 ksi, the minimum allowable value specified in the ASTM provisions. In addition, it would appear that the code provisions did not consider any increase in rivet strength due to driving, even though the phenomenon is well documented. In the *Guide to Design Criteria for Bolted and Riveted Joints* (Kulak et al., 1987), a value of 80 ksi is presented as a "reasonable lower bound estimate of the rivet tensile capacity" for high strength rivets when considering strength increase due to driving.

As a further measure of conservatism, the code provisions reduced the shear strength of individual rivets to account for the non-uniform distribution of forces for connections with multiple rows of fasteners due to shear lag within the fastener group. To account for this shear lag phenomenon, the code provisions implicitly applied a single reduction factor for shorter connections. For connections longer than 50 inches, the reduction factor is explicitly reduced by an additional 20 percent[4]. Although the actual reduction in group strength effectiveness is gradual with increasing connection length, this code provision applies a step-like function for simplicity. This results in a non-uniform factor of safety for riveted joints.

The overall rivet shear resistance factor is not explicitly presented in Load Factor provisions. In Table 10.56 A in AASHTO *Standard Specifications* (AASHTO 2002), calculations indicated that a fastener shear resistance factor of 0.75 was used for A307, A490, and A325 fasteners.

Table 1 shows calculations for factored rivet shear resistance using both the minimum tensile strength, which is consistent with the noted code provisions, and the expected rivet

Table 1. Rivet shear resistance.

Expression for Ultimate	F_u (ksi)	F_v/F_u	F_v (ksi)	Length factor	φ	φF (ksi)
Minimum	68	0.75	51.0	0.80	0.75	30.6
Expected	80	0.75	60.0	0.80	0.75	36.0

tensile strength when calculated in accordance with the *Guide to Design Criteria for Bolted and Riveted Joints* (Kulak et al., 1987).

Load Factor code provisions specify a factored shear resistance, φF, of 30 ksi for high strength rivets. Therefore, the expected rivet strength represents a 20 percent increase in capacity over the code. If the higher rivet strengths could be demonstrated through testing, the number of vulnerable connections could then be recomputed, and then dramatically reduced so that significant savings may be realized without performing unnecessary work on the subject bridge. Based on the assumption that a rivet testing program with a statistically satisfactory sample size might reveal additional reserve strength in the fasteners of even half of the expected 20 percent gain, the cost savings associated with the reduced scope of construction work would more than outweigh the cost of the rivet testing program.

7 PRAGMATISM OF TESTING/REPLACEMENT PROGRAM

To accurately assess the strength of the rivets on the Delaware River Turnpike Bridge, a rivet testing program was recommended by the engineer. Given the large discrepancy between the minimum and expected rivet shear strength values, it was recommended that a large enough population of rivets should be tested to give a 95 percent confidence limit on the average rivet strength. From the series of tests, the confidence limit could then be computed from the mean strength, μ, and the standard deviation, σ, as per the following equation:

$$f_n = \mu - 1.96\sigma \tag{1}$$

Published guidance for selecting test sample population was researched in current code doctrine. To that end, current AASHTO and ASTM documentation was chosen as the most relevant source material for this guidance. It was found that the AASHTO *Manual for Bridge Evaluation* (AAHTO 2008) recommends a minimum of three samples be used for testing. It was also found that ASTM Designation F1470, the *Standard Guide for Fastener Sampling for Specified Mechanical Properties and Performance Inspection*, recommends the total number of samples be based on the production lot size. For detection of defects relating to tensile strength, F1470 specifies at least seven samples be taken from lots of greater than 500,000 rivets. For detection of defects relating to rivet hardness, F1470 recommends at least 15 samples be taken from lots of greater than 500,000 rivets. If considering each different rivet size in the subject structure as a lot, ASTM F1470 would then dictate a minimum of 15 rivets of each size should be tested.

While this sampling might otherwise prove effective in meeting the bare minimum of prescribed due diligence, there was no means by which it could be substantiated that all rivets in the structure came from the same lot or were of like quality. In light of this, and with the knowledge that even optimistic predictions of rivet strengths would still require much rivet replacement work, it was pragmatic to consider an expanded rivet testing program. A large number of rivet replacements presented a negligible cost opportunity to also harvest a large number of test rivet samples. As will be discussed in greater detail below, it was then recommended to the bridge owner to test a larger number of samples than required by ASTM F1470 in order to provide greater confidence in the rivet strengths.

It should be noted that there is no explicit testing specification available in current code provisions that outlines testing procedures for shear strength testing of high strength rivets. Based on research of the most relevant codes, it was determined that sample rivets should be tested according to ASTM Designation F606, *Standard Test Methods for Determining the Mechanical Properties of Externally and Internally Threaded Fasteners, Washers, and Rivets*. Although noted code provisions are based on tensile strength of fasteners, it was recommended to test the rivets in double shear to measure the shear strength of the rivets directly, which is directly applicable to the intended design loading of the subject structure's rivets.

8 STRATEGY FOR ADJUSTING SCOPE OF WORK BASED ON RIVET TEST RESULTS

As stated above, the use of the rivet shear strength prescribed by current code provisions is expected to be conservative in comparison to rivet strength predicted from previous tests of riveted joints. Therefore, significant savings in materials and construction costs can potentially be realized if testing of the subject structure's rivets can demonstrate even marginally higher strengths. Conservative estimates indicated a potential elimination of approximately 80 percent of gusset plate work locations if rivet strengths of 20 percent higher than expected could be demonstrated. The term "work location" refers to a panel point location within a truss requiring gusset plate retrofit work.

In the Figure 2 below, the number of work locations is plotted with respect to rivet strength, φF.

It can be seen in the above chart that the number of work locations fell off dramatically between assumed rivet strengths of 32 ksi and 34 ksi while the ranges below 32 ksi and above 34 ksi remain relatively flat. Please note that this chart is specific to the subject structure and should not be used as a guideline for rivet replacements on other structures. The relationship between work locations and anticipated rivet strength is highly dependent on the live load to dead load ratio for each structure's configuration.

Based on the above chart and engineering judgment, it was decided to size the construction contract to retrofit riveted connections which did not meet desired capacity needs using an anticipated rivet strength (φF = 33 ksi). This limit was chosen as it both fell cleanly between the published rivet strength specified in current code provisions (φF = 30 ksi) and the expected rivet strength derived from published research (φF = 36 ksi). Based on the above chart, this anticipated rivet strength reduced the total number of work locations by approximately 60 percent. Since rivet production lots were not known, each and every work location was specified to have two rivets tested for shear strength. Rivet testing was explicitly performed as an ongoing construction activity concurrent with the rivet replacement work.

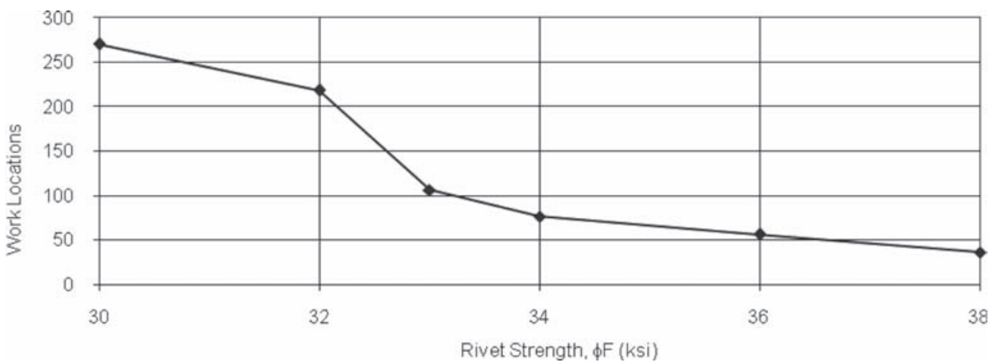

Figure 2. Work locations vs. anticipated rivet strength.

This allowed the engineer and the bridge owner to effect real-time decisions on modifications of the scope of work based on actual rivet test results. By testing each work location, a case-by-case decision could be made to increase rivet replacement schedules where inferior rivets were found. Rivet testing was performed by an independent laboratory to assure impartial test results.

9 CONSTRUCTION CONTRACT DEVELOPMENT

For the letting of the contract to rehabilitate the structure, several logistical obstacles were encountered. First and foremost among these obstacles was the development of a practical method of replacing rivets in highly stressed connections while maintaining unimpeded active traffic on the structure. With the actual rivet strength still an unknown quantity, a high degree of conservatism was employed via limiting the contractor to a "single vacant hole" provision. This forced the contractor to remove a single rivet and replace it with a new fully tightened bolt before removing any additional rivets. As the riveted connections for the subject structure contained more than 50 rivets in all instances, this provision implicitly allowed a maximum 2 percent connection strength reduction during construction operations. It was understood that this provision also created negative impacts for the contractor's production rate. In instances where the contractor was forced to remove more than one rivet at a time, high strength driven-fit drift pins were allowed at the permission of the engineer. For bidding purposes, the work was broken down into individual pay items, which incorporated all work at each gusset plate location as a single pay item. This allowed inclusion of additional work locations as the quantity of as-bid work items could simply be increased to suit the need.

Other project obstacles included the handling and disposal of the lead paint coating on the structure. As this structure was already slated for repainting by the owner, complete removal and restoration of the existing paint system was not a priority. Prior to use of any rivet removal methods, the contractor removed paint coatings within a 3" radius of the rivets to be replaced using needle scalers with vacuum hoods and appropriate containment and safety equipment. Workers performing rivet removal operations were equipped with filter masks and protective gear.

The engineer's estimate of construction for the contract was developed primarily from the access constraints inherent with any long span bridge located at an average of 100 feet above ground or water. More than any other factor, these access constraints drove the project cost estimate. A final estimated unit cost was derived based on the number of rivets to be replaced at each work location after accounting for the use of portable work platforms, articulated bucket trucks, and materials handling. Each work location was therefore estimated at an assumed cost per rivet multiplied by the number of rivets to be replaced at each work location. At the time of the bid opening, it became apparent that all bidders used similar methodology when developing their own unit prices. The low bid winner used an apparent unit cost of $200/rivet. The average unit cost across all bidders was computed to be approximately $450/rivet, with the average of the two lowest bidders computed at $275/rivet.

10 RIVET REMOVAL METHODS—THERMAL VS. MECHANICAL

The contract plans required the use of pneumatically driven impact hammers (known as 'hell dogs') equipped with either chisel tips to 'wipe' off the rivet heads, or equipped with blunt tips to drive the rivet shanks out of its hole. Thermal (torch or arc-lance/plasma cutter) methods were specifically disallowed.

Ultimately, the mechanical methods used to remove the rivets were time consuming (See Figure 3). Where mechanical methods typically require less than five minutes to 'wipe' the head off of a more typical carbon rivet, the high strength rivets used in the subject structure

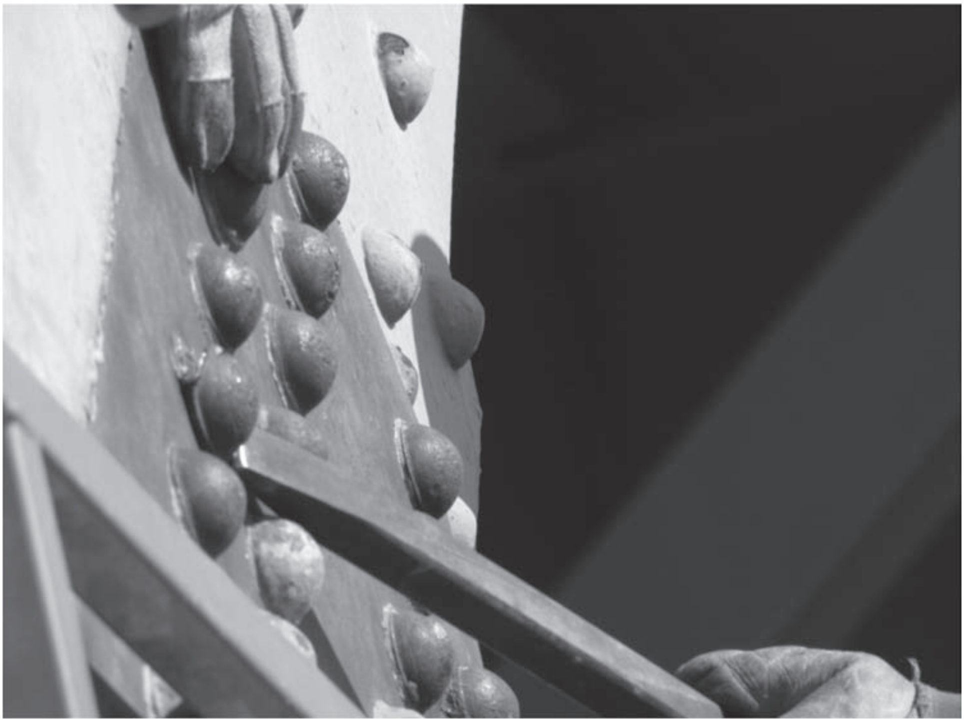

Figure 3. Mechanical rivet removal with hell dog.

were noted to be particularly hard. Removal rates of up to 15 minutes per rivet were noted when using the hell dogs, and wiping chisel heads broke frequently. Much of the kinetic energy from the repetitive impacting of the chisel tips was converted into heat, which elevated the temperature of the removed rivet and the surrounding plate to well over 200 degrees F. The wiping chisel was also difficult to control and gouges were frequently made in the gusset plates. In an effort to accelerate the project schedule, an alternate rivet removal technique using an air-arc gouging process was proposed by the contractor.

In order to compare the efficacy of the air-arc removal method to the hell dog removal method, two test specimens were presented for the demonstration. These specimens were identical, ½" steel plates cut into squares of approximately 12" × 12". ASTM F1852 tension control "buttonhead" bolts were installed in flame cut holes in the plates to simulate the high strength rivet heads. The first specimen was mounted horizontally for removal of the bolt head from the bolts shank. The steel plate was marked around the bolt head with temperature sensitive crayons prior to removal. The marks created by these crayons will melt when the base metal is heated to a temperature specific to each crayon (See Figure 4). For the purposes of this test, 200 degree and 350 degree melting point crayon lines were marked out in a radial spoke arrangement around the subject bolt. Prior to testing, an infrared non-contact thermometer was used to measure the temperature of the cold plate before air-arc removal, which was 73 degrees.

After air-arc removal of the first bolt, the infrared thermometer was used to check the temperature of the base metal directly adjacent to the bolt hole, this was measured at 94 degrees. The accuracy of the thermometer was verified by direct hand contact to the base metal, which was cool to the touch. No melting of the heat crayons was evident away from the heat blast of the air-arc gouging or on the underside of the base metal. The head of the bolt was converted to slag and blown away in the air-arc process. The remainder of the bolt head

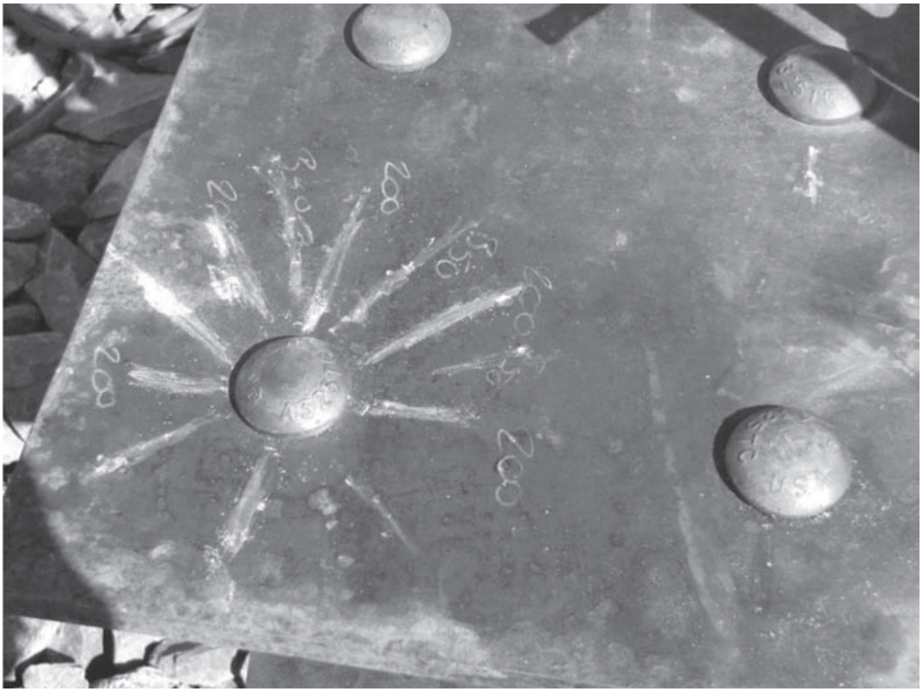

Figure 4. Test specimen plate.

consisted of a ring of metal where the air-arc process did not physically make contact with the bolt head. A hell dog was used to remove the remaining bolt shank. Note the melting/vaporization of temperature sensitive chalk in direction of blast from air-arc removal process (See Figure 5 and Figure 6).

After the removal of three successive rivets on the first test specimen, the temperature in the plate was measured to be 110 degrees. The second test specimen was used to verify that the air-arc method could be used when the plate was oriented vertically, similar to the orientation of the rivets to be removed on the structure gusset plates. This test also yielded successful results.

The air-arc technique was observed to be much faster than the mechanical removal methods stipulated in the contract documents. The cycle time to remove one bolt head via air-arc methods was 45 seconds from initiation of spark to clearing the steel plate of the remainder of the bolt head. The procedure was performed by a single welder operator.

Based on the positive results of the demonstration, the air-arc rivet removal process was approved for use by the Engineer with the following stipulations noted:

- All air-arc equipment operators shall be certified welders. Air-arc gouging procedures shall be performed in accordance with American Welding Society (AWS) D1.5 C–3.2.6. In the presence of the Engineer, each operator shall demonstrate the ability to remove the heads from four button head ASTM F1852 bolts mounted in a vertically oriented 12" × 12" × ½" thick plate without visibly damaging the plate, or raising the temperature of the plate to a temperature of greater than 150 degrees F. Operators which have not operated the air-arc equipment for 30 calendar days or more shall be required to re-demonstrate their ability as outlined above.
- Fifty percent of all rivets removed shall be circled and crossed on the rivet head to remain using a 150 degree Fahrenheit “Tempilstik” heat sensitive chalk before air-arc gouging of the other side rivet. These rivets shall be submitted to the Engineer for review.

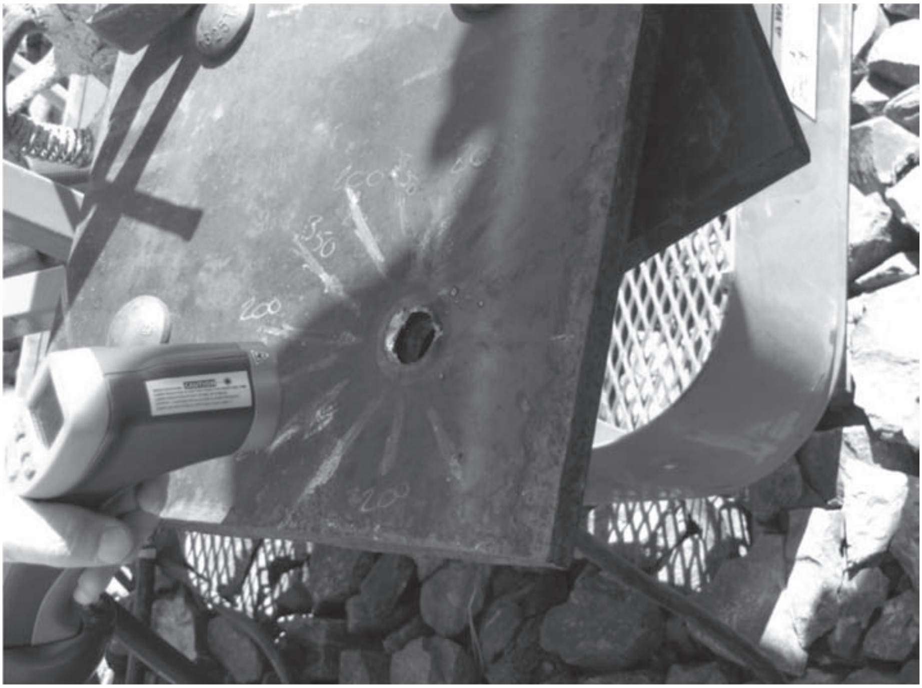

Figure 5. Test specimen plate immediately after air-arc bolt removal.

Figure 6. Test specimen plate.

Air-arc operators that submit rivets with melted chalk shall be subject to recertification or expulsion from the worksite at the discretion of the Engineer.

- The Engineer reserves the right to request recertification of any Air-Arc operator at any time at his discretion.
- The contractor shall take a high resolution photograph of the gusset plate base metal at all locations where air-arc methods are utilized. These pictures will be submitted to the Engineer for review and approval. Base metal which, in the opinion of the Engineer, has been damaged or is otherwise unsatisfactory shall be repaired to the Engineer's satisfaction at no additional cost to the bridge owner.
- A repair procedure shall be submitted by the Contractor clearly indicating his methods of correcting any damage done to the gusset plate when using air-arc methods. Gouges into gusset plates of less than 1/16" shall be ground smooth. Gouges greater than 1/16" in depth shall be repaired via an approved welding procedure. The welding repair procedure shall be signed and sealed by a professional engineer. All welded repairs shall be 100 percent UT tested.
- At no time may more than 25 percent of the heads of rivets in a gusset plate connection be removed without replacing them with new bolts. All rivets with removed heads shall be replaced with fully torqued high strength bolts before the end of the work shift in which the rivet heads were removed. Only one rivet shank may be removed at a time.
- Protection from molten metal spatter and arc light glare shall be provided at all work locations using air-arc equipment and shall be to the satisfaction of the Engineer. The Contractor shall resubmit his fire safety plan to account for the presence of an open-flame at each and every work location where air-arc methods are used. At a minimum, the revised plan will include provision for debris and safety catches which are fire-resistant, minimum safe distance for wet paint or other flammable materials, a manifest of countermeasures for unintentional initiation of fires (i.e. fire extinguishers /fire blankets), and additional first-aid and medical equipment, as required. Limited fire countermeasures and first aid medical equipment shall be kept in the air-arc equipment operator's man-lift or suspended platform at all times.
- The Contractor, at his own expense, shall submit for testing three rivets with its head removed via chiseling and three rivets with its head removed via air-arc gouging in order to verify that the mechanical shear properties of the rivets have not been altered. Testing shall be in accordance with criteria set forth in the Specifications.

Using the above provisions, the Contractor was able to accelerate his rivet replacement schedule at most work locations in the project. The contractor elected to switch back to mechanical rivet removal methods when working on portions of the truss located over active traffic. High ambient winds and close proximity to the roadway made containing the molten metal spatter and shielding the arc light from public view impractical.

Note that the "one vacant hole" rivet replacement stipulation was partially relaxed to allow the contractor to remove up to 25 percent of the rivet heads at one time at any connection. This was done as a concession to the logistics of the air-arc rivet removal methodology. The operator and the equipment needed to power the arc lance fully occupied the work platform or man-lift bucket. A second crew equipped with hell dogs used to drive the shanks out of the rivet holes and install new bolts fully occupied a second bucket. It was impractical to shift crews for every rivet replacement at a work location. The 25 percent criteria was developed to ensure that the design shear area of the connection was maintained while still retaining 75 percent of the gusset plate stability offered by the rivet heads. Even with this provision in place, the work still proceeded at an accelerated pace, and rivet replacement work was completed ahead of schedule.

Regardless of the rivet removal method used, the contractor was required to adhere to strict quality control provisions in the contract documents. The theory of these provisions focused on maintaining a tight bolt arrangement in the final condition; as each rivet shank was removed, it was replaced with a fully torqued A-325 high strength bolt. This process was repeated until all designated rivets were replaced. At that time, all the new bolts were then rechecked for desired installation torque at once. The intent of this provision was to ensure that the tightening of a new bolt adjacent to a previously tightened bolt did not force the gusset plate connection plies together tighter, thus loosening the adjacent bolt.

11 TESTING PROGRAM AND RESULTS

The testing of the designated rivets was performed by the ATLSS Engineering Research Center at Lehigh University. As stated above, the rivets were tested in double shear according to ASTM F606-07, *Standard Test Methods for Determining the Mechanical Properties of Externally and Internally Threaded Fasteners, Washers and Rivets*. Loading was applied with a SATEC 600 kip capacity Universal Testing Machine (See Figure 7). For cost reference, the low bid winning contractor quoted a combined unit cost of \$60.10 to extract with care and test each rivet. The average unit cost across all bidders was computed to be approximately \$200/rivet. The combined rivet testing program was performed at a cost of \$26,000 to the bridge owner.

Note that the Figure 8 above includes a double-shear jig which is not explicitly in accordance with the noted ASTM F606 testing specification. This jig produced more reliable results without jamming of the hydraulic press. As a side benefit, the double shear test intrinsically offers a more thorough testing regimen, as each rivet is essentially shear tested twice.

In total, over 400 rivet shear tests were performed using two rivets removed from each and every gusset plate work location. The raw data harvested from the testing program was then filtered to account for expected standard deviation to arrive at a final design ultimate strength with a 95 percent confidence limit. This ultimate strength data was then factored in the same manner used to arrive at the published allowable shear strength in current code provisions. This yields Equations 2 and 3 below:

$$\varphi F = (95\%\ \text{Confidence Limit}) \times (\varphi) \times (\text{Length Factor}) \tag{2}$$

$$\varphi F = (F_v) \times (.75) \times (.80) \tag{3}$$

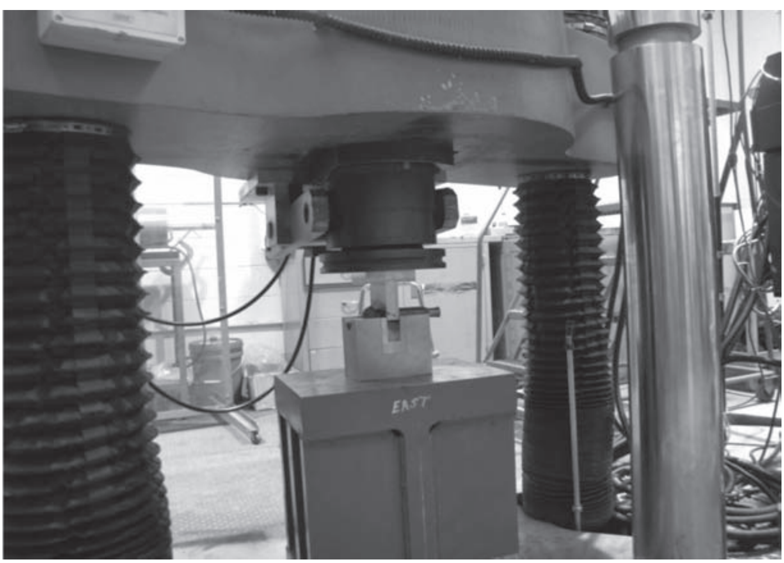

Figure 7. Rivet testing set-up with SATEC 600 kip capacity Universal Testing Machine.

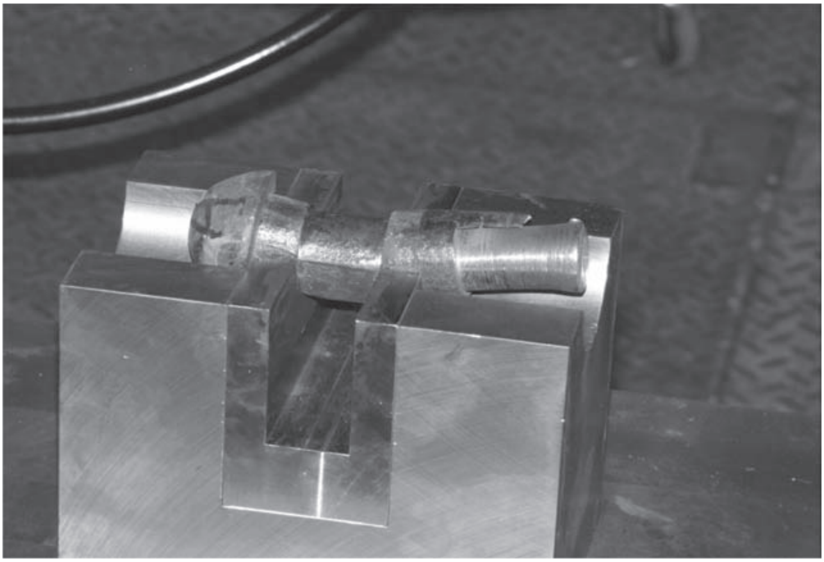

Figure 8. Rivet tested to failure.

Table 2. Rivet testing results.

Evaluation basis/rivet population	Average ultimate shear stress μ	One standard deviation of ultimate shear stress σ	95% Confidence limit $f_n = F_v$	$\varphi F = (F_v) \times (\varphi) \times$ (Length factor)	φF Assumed rivet strength
Overall (PA and NJ approach truss spans and main truss spans)	64.0 ksi	3.5 ksi	57.3 ksi	34.4 ksi	33.0 ksi
3 span truss unit PA	62.9 ksi	3.9 ksi	55.2 ksi	33.1 ksi	33.0 ksi
4 span truss unit PA	63.5 ksi	2.9 ksi	57.7 ksi	34.6 ksi	33.0 ksi
3 span truss unit NJ	64.1 ksi	3.9 ksi	56.6 ksi	33.9 ksi	33.0 ksi
4 span truss unit NJ	64.3 ksi	3.3 ksi	57.8 ksi	34.7 ksi	33.0 ksi
Main span truss unit	64.7 ksi	3.3 ksi	58.2 ksi	34.9 ksi	33.0 ksi

In order to track larger system wide trending in the rivet strengths, the test results were organized by the multi-span unit from which they were removed. As discussed in the introduction, the truss spans of the structure viaduct were constructed in arrangements consisting of a central three span continuous main spans unit, which is flanked by symmetrical four span continuous approach units, which are in turn flanked by symmetrical three span approach units. For simplicity sake, the spans are named for the jurisdictional limits into which they fall, where the center of the main river unit represents the state line between New Jersey and New York. The evaluation of the rivet testing results for all the truss spans is summarized in Table 2 below.

12 CONCLUSIONS

From the above table, it can be seen that the testing results yielded surprisingly consistent results. Where published code provisions prescribed a rivet strength threshold of 30 ksi, and where published research prescribed rivet strength of 36 ksi, the testing program revealed as-tested strengths of 33–34 ksi with an overall average of 34.4 ksi. The assumed 33 ksi used to prepare the contract documents was found to be conservative by approximately 4 percent based on final testing results. The cost of the testing program, though more robust than ultimately needed, was effective in demonstrating an additional 10 percent of reserve strength in the subject structure's rivets. This allowed for a reduction in scope of work in the final rivet replacement contract resulting in a savings of almost $2,000,000 for the bridge owner while also providing a very high degree of confidence regarding the strength and condition of the existing rivets. Please note that the $2,000,000 savings represents 40 percent reduction in total anticipated project cost, where the rivet testing program cost is less than 1 percent of the low-bid contract cost of $3,080,000.

It should be noted that the testing results for the subject structure's rivets are only applicable to the subject structure. The as-built date of 1956 was relatively recent when viewing the total time frame during which structural riveting was the predominant fastener choice for decades. Rivet steel processes were constantly evolving and the ASTM A-195 rivets used to construct this structure represent the best available material in the 'sunset' period of rivet fasteners. Older rivet materials may not exhibit the strengths noted throughout this paper. Newer rivets may exhibit even greater strengths. The intention of this paper is not to challenge existing code provisions, but to provide insight and practical experience for means and methods available to engineers and bridge owners so that they may be able to rationally and economically evaluate their own structures.

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Chapter 14

Rehabilitation of the 31st Street Arch of Hell Gate Viaduct, New York City

M.J. Abrahams
Parsons Brinckerhoff, New York, NY, USA

ABSTRACT: The 31st Street Arch is a reinforced concrete arch bridge that was constructed around 1914. It is part of the Hell Gate Viaduct, which is comprised of a series of concrete arch bridges, steel truss bridges and retained earth structures. The bridge is approximately 70 feet wide, 60 feet high and spans approximately 100 feet over 31st Street and an elevated New York City Transit track structure. It carries four tracks, three of which are active with Amtrak and CSX trains. A series of investigations, including visual inspection, material testing and structural analysis was conducted. The visual inspection revealed that the extent of deterioration was more extensive than anticipated. The presence of loose concrete above the sidewalk, some sections of which are several feet wide, presented a danger to pedestrians below. Based on observations made in the field, additional measures were recommended. These measures included removal of loose and delaminated concrete, testing of concrete and an evaluation of the structure to determine the maximum tension force in the tie rods. Material testing consisted of tests performed on concrete core samples that were extracted from the arch structure. Tests included core testing and petrographic analysis. The original concrete was found to be in poor condition due to extensive cracking, with the major cause attributed to moisture and associated freeze thaw cycles. Prior repairs were found to be cracked and poorly bonded to the original concrete. Based on these findings plans were prepared that included removal of all loose concrete, installation of a new stainless steel drainage system, addition of a cast-in-place reinforced concrete reinforcing arch at each face, and the addition of 4-inch reinforced shotcrete facing. While shotcrete is not a common repair material in the New York City area, it was found to be particularly suitable for this project. The project received the 2010 Outstanding Shotcrete Project of the Year for repair and rehabilitation from the American Shotcrete Association.

1 INTRODUCTION

The 31st Street Arch is part of the Hell Gate Viaduct is a 16,900 foot long 4 track structure that carries three active tracks and is comprised of a series of concrete arch bridges, steel truss bridges and retained earth structures. The 31st Street Bridge is approximately 70 ft. wide, 60 ft. high and spans approximately 100 ft. over 31st Street and an elevated New York City Transit Authority track structure.

The available drawings indicate that the 31st Street Bridge includes an arch slab having a minimum thickness of 4'–9" and retaining walls having a minimum thickness of 3'–0". The retaining walls of the bridge structure are continuous with no expansion joints and are connected to one another by a series of 2 1/4" diameter steel tie rods arranged in a 10 ft. by 10 ft. grid. These tie rods serve to support the opposing retaining walls and to resist lateral soil pressure and train surcharge loads. Design drawings of the bridge indicate that the tie-rods are encased in concrete and anchored to an arrangement of vertical and horizontal steel channel sections that are embedded in the retaining walls. The drawings also indicate that the retaining walls are also connected by transverse walls spaced at approximately 50 ft. Finally,

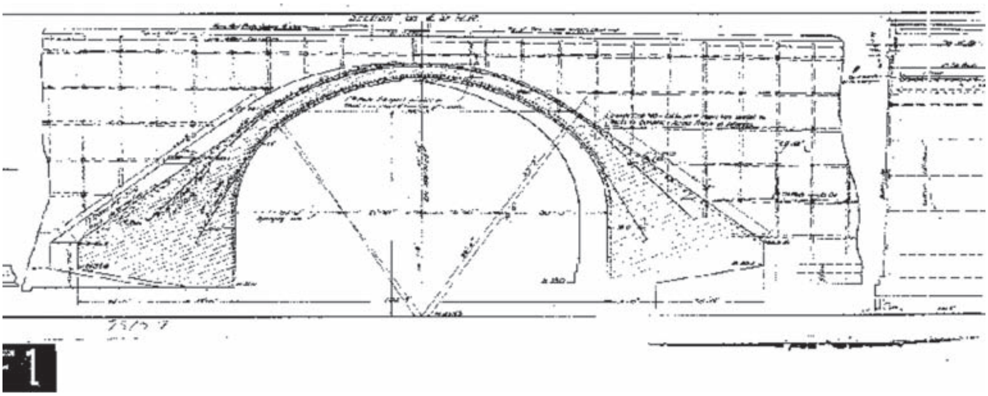

Figure 1. Elevation and section through arch.

the drawings show that the retaining walls have 4" diameter weep holes located a few feet above ground level and the arch slab is covered with a felt waterproofing membrane above an internal water dam and that the inside face of the retaining walls are covered with pitch waterproofing.

Two Engineering News articles relating to the Hell Gate Viaduct Structure were obtained and provided useful information relating to the original design. The article (ENR 1915) is of particular interest as it explains that a "novel type of construction" was used because it gave a large savings in cost over the much heavier gravity sections of retaining wall, and because it was also cheaper than steel viaduct—piers and plate girders—for which the economical height was above 65 ft. in this case. The article states that "economy dictated the use of all available means for reducing the earth pressure without abandoning the idea of solid fill;. The means employed included consolidating the fill by ramming thin layers and providing ample drainage to keep the fill thoroughly dry from top to bottom. The authors note that by employing these methods, "a remarkably steep slope of repose of the fill material could be figured on". This steep slope of repose translates into smaller tension forces in the tie rods. An extract from the Engineering News article (see Figure 1) shows the typical general arrangement of the reinforced concrete viaduct structures.

In 1994, drain pipes were installed on the west side of the south retaining wall and on the east side of the north retaining wall structure in order to improve drainage of water in the retained backfill and eliminate water seepage though cracks in the retaining walls.

At the time of inspection in 2005, the Bridge 6.45 exhibited extensive cracking and spalling of concrete and the drain pipes that were installed in 1994 had not adequately solved the problem of water leaking through cracks in the retaining wall. In cold weather, water seepage through cracks in the retaining walls resulted in the formation of icicles along the sides of the arch and ice patches on the sidewalk below presenting a hazard to pedestrians on the street and NYCT platform.

2 VISUAL INSPECTION, MATERIAL TESTING AND STRUCTURAL EVALUATION

2.1 *Visual inspection*

A visual inspection of Bridge 6.45 was conducted in 2004. The visual inspection revealed that the underside of the arch slab itself was in relatively good condition with localized areas of cracking and efflorescence located primarily along the edges of the arch slab. The conditions were consistent with those anticipated based on previous site visits where observations were limited to those made from the roadway and NYCT station platform levels. However, the close up visual inspection of the retaining walls and sides of the arch slab, found extensive cracking, efflorescence, loose and spalling concrete and active water leakage (see Figure 2). The most severe deterioration was located on the west side of the south retaining wall and the east side of the north retaining wall near the interface of the arch slab and the retaining wall. These locations correspond to the longer faces of the arch walls associated with the skew of the arch relative to the walls. It was clearly evident that the structure had been repaired at some time in the past and that at that time a cementitious coating had been applied to the structure. At the time it was noted that the icicles that formed in the winter due to leaking water as well as the presence of loose concrete above the sidewalk presented a danger to pedestrians below. Within a year a piece of concrete fell from the structure on to the NYCT platform and as a result, scaffolding and netting was installed under and at both sides of the arch. This scaffolding remained in place until the repairs were completed.

2.2 *Material testing*

Material testing consisted of tests performed on concrete core samples that were extracted from the arch structure. The tests conducted included Core Testing (ASTM C42) and

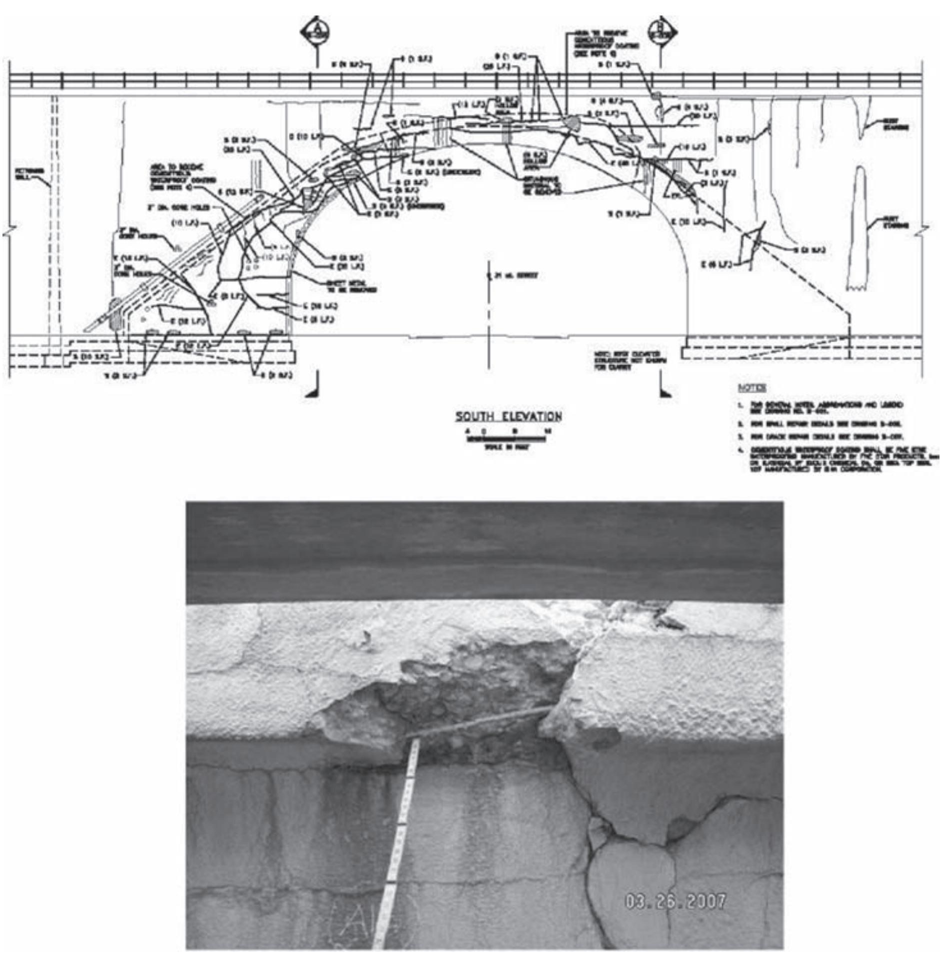

Figure 2. As inspected condition of arch.

Petrographic Analysis (ASTM 856). A total of six 4 inch diameter core samples were taken including four from the northeast side of the structure and two from the southeast side of the structure. Four cores were tested for compression strength. The compression strength ranged from 6,464 psi to 6,925 psi with an average strength of 6,756 psi. The original design strength of the concrete is not known.

Two cores (N-2 and N-3) were subjected to petrographic analysis. Core N-2 consisted entirely of concrete original to the structure and Core N-3 consisted of two different concrete mixes; concrete original to the structure and a newer (repair) concrete. The older concrete of both cores was transected by numerous cracks. Both the old and new concrete are non-air-entrained. Based on the petrographic analysis, the original concrete was judged to be in poor condition due to extensive cracking with the major cause attributed to moisture and associated freeze thaw cycles. The samples also exhibited abundant secondary deposits of ettringite and smaller amounts of calcium carbonate and alkali silica gel deposits; evidence that the concrete has been exposed to tremendous amounts of moisture and has become critically saturated.

The repair concrete of Core N-3 was judged to be in fair condition, however, the old concrete was considered an unstable base because of the extensive cracking. In addition, the bonding agent at the interface of the old and new concrete was often dry and brittle and may be incompatible with the cementitious materials. A thin mortar layer was present over the repaired concrete. This layer was poorly bonded as demonstrated by a crack at the interface with the repaired concrete. Portions of this layer had already de-bonded.

2.3 *Tie rods*

An evaluation was conducted to determine the forces in the tie rods. The methods used in the original design for determining soil lateral pressures were considered inaccurate. While, available literature indicates that a high level of compaction was achieved using mechanical tampers, much of the compaction developed in the clayey soil due to mechanical compaction could have dissipated over time. Thus the fill was treated as normally consolidated.

As described above, the retaining walls are restrained at various levels by the tie-rods and also by the cross-walls located every approximately 50 ft. If these elements completely restrained the retaining walls and prevented them from flexing and rotating as would a conventional retaining structure, they would not allow the formation of an active soil wedge and at-rest lateral earth pressure would develop. However, the tie rods are elastic and will allow the wall to move to some degree. Therefore, it is probable that lateral earth pressure on the retaining wall will fall somewhere between the active and at-rest conditions.

Calculations based on three tracks of E 60 railway loading with current accepted practice for computing lateral earth pressures, indicated that tensile forces in the tie rods are higher than those of the original design and result in a maximum tensile force per tie-rod that is higher than the maximum of 84 kips from the original design, (ENR 1914). However, only one track is typically active at a time, and that is typically an AMTRAK train that is well below an E 60 loading. Thus the in service tie rod stress is approximately 24 ksi. The material used for the tie rods is unknown but structural steel used for bridges at that time (Ketchum 1918) had an ultimate strength of 55–65 ksi and steel used for eye bars had a ultimate strength of 90–105 ksi so the computed in service stresses were judged to be acceptable.

3 REHABILITATION

Based on the observed state of deterioration of the structure, plans were prepared to restore the integrity of the structure. This included a new drainage system, removal of all loose concrete from the walls (which was extensive), injection of cracks in the walls, construction of a secondary reinforced concrete arch under the arch at the exterior walls, and a 4" thick reinforced concrete facing that was tied back into the arch side walls. The drainage system was all stainless steel and made use of large diameter commercial well screens in the arch fill. These types of screens are commonly utilized in the Midwest for wells. Cleanouts were provided at each drain, which was then piped down into new French drains. The new arch supports and some of the spall repairs were anchored back into the existing arch concrete with large diameter reinforcing bars that were placed with adhesive grout. Each anchor was load tested. In order to improve the durability of the repair, all reinforcing steel was hot dip galvanized.

After initially considering cast in place concrete, it was determined that shotcrete would be a preferable way to place the 4" concrete facing (see Figure 3). The shotcrete was also used to backfill all of the areas where spalls had been removed and these were in some cases over a foot deep. The shotcrete was specified to be in accordance with ACI 506.2 *Materials, Proportioning, and Application of Shotcrete*, and ACI 5006.3R, *Guide to Certification of Shotcrete Nozzlemen*. In order to improve the durability of the shotcrete entrained air; steel fibers; and 10% silica fume were added to the mix. The 28 days strength was 4000 psi. Prior to application the existing surface was sand blasted to remove any residual coating. After finishing the new surface received a proprietary cementitious coating to provide a uniform color and to enhance the durability.

After the mix and nozzlemen were qualified, test panels were prepared. These panels remained on site as references during the shotcrete application. A float finish with a +/– 1/4" tolerance was specified as was a 7 day wet cure. While the contractor was able to work off the scaffolding that had been erected to protect the area below, the need to prevent shotcrete and water from the wet cure from falling on to the area below, which as mentioned above included two active NYCTA Tracks added to the challenge. This proved to be very

Figure 3. Shotcrete, note PVC pipe used to guide thickness.

Figure 4. Finished surface, note new drain pipes with cleanout and added arch structure.

successful, (see Figure 4) as both the contractor and his shotcrete subcontractor were up to the challenge. The project received the 2010 Outstanding Shotcrete Project of the Year for repair and rehabilitation from the American Shotcrete Association.

4 CONCLUSIONS

While shotcrete is not a common repair material in the New York City area, it was found to be particularly suitable for this project. It is suggested that it be considered for use in smaller projects, in particular walls that normally would use cast-in-place construction.

ACKNOWLEDGMENTS

The guidance and assistance provided by AMTRAK staff Craig Weed, Vladimir Kreyskop and Manuel Cabrera was critical to the success of this project.

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Chapter 15

Main span hinge elimination and continuity retrofit of the Sandesund Prestressed Box Girder Bridge in Norway

S. Fjeldheim
Bridge Department, Norwegian Public Roads Administration, Norway

S. Kr. Heggem
Johs. Holt AS, Consulting Engineers, Norway

ABSTRACT: The Sandesund bridge, located about 100 km south of Oslo, was built circa 1972–77. The total bridge length is 1521 m and is divided in four sections. The section of the structure crossing over the Torsbekkdalen Valley contains the largest 97,7 m span and is where a change in the static system was considered. The main spans were built by the cantilever construction method and the Torsbekkdalen Valley span was designed and constructed with a hinge in the middle of the span. Over the years, the main span has deflected about 200 mm in the middle. Due to increasing heavy traffic, the hinge was increasingly subjected to live load impact. It was therefore desirable to investigate whether it was possible to introduce continuity in the main span. On the other hand, a change in the static system on a bridge of this size is something that must be taken very seriously. Miscalculations here can have dramatic consequences. In the years 2005–2007 a new bridge was built parallell to the existing bridge, allowing improvements to take place on the existing bridge.

1 INTRODUCTION

Since the mid sixties many prestressed box girder bridges have been built by the cantilever construction method in Norway. The earlier bridges were often hinged in the centre of the main span. Sometimes this was simply necessary for example when having a fixed counterbalance abutment on each side of the main span. The Puttesund bridge in Norway is a typical example for this situation (Teigen & Fjeldheim 1998). Horizontal movements must after all have a place to go. These hinges can only accommodate longitudinal movements, shear forces and torsion. Even though the main span in this case is only 97,7 m, the sag started to develop quite soon after completion and increased steadily. After the first 15–20 years it slowed down. The sag is limited to an affected length of about 30 m in the middle of the span with a maximum value of 200 mm in the centre. The reasons for the sag can be many, and in addition to the impact loading from traffic, long term creep of the concrete can in reality be more than what was used in the original analysis. This sag problem is also observed in other countries. The analysis were performed according to the Norwegian National Codes (NS3473) and (Handbook 185). Figures 1 and 2 show an overview of the whole bridge and Torsbekkdalen Valley section respectively.

2 ORIGINAL PLAN FOR LIFTING THE CANTILEVER ARMS IN TORSBEKKDALEN

Since the sag problem was known already when the design of the bridge was going on, measures were taken in case this problem should later appear. Firstly, the cambers during

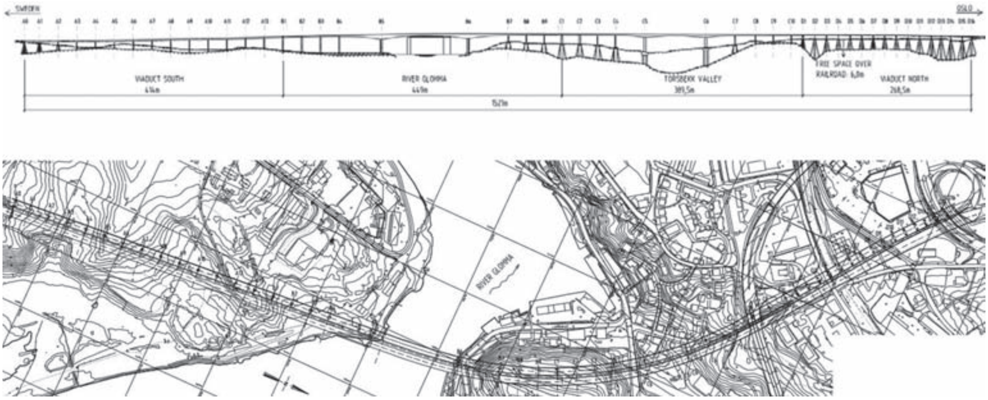

Figure 1. Sandesund bridge.

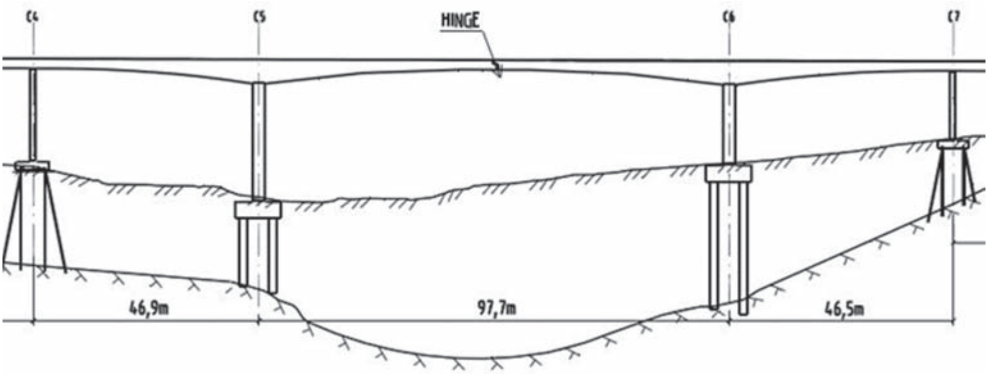

Figure 2. Torsbekkdalen valley section.

construction were somewhat increased (estimated ca. 10–30 mm) beyond what was necessary from the analysis. Secondly, 6 reserve steel ducts for cables were cast in each cantilever arm extending from the hinge and ending almost to the first flanking span column, leaving just enough space for a jack (Figure 3). Everything was prepared for additional cables to be placed into the ducts followed by tensioning. The operation to perform this work was of course included in the design documents. In the event that all 6 reserve cable ducts on each cantilever arm were to be utilized„ this should yield an uplift of about 120 mm in the centre of the main span, as per the original design documents. Unfortunately, due to negligence during construction, some of the reserve ducts were filled with concrete. By applying a camera through these ducts, it was possible to localize the clogged areas. After some chiselling work, it was possible to install about 60% of feasible tensioning force (Figure 4).

3 PROPOSAL FOR CHANGING THE STATIC SYSTEM IN THE MAIN SPAN

When planning the rehabilitation work on the existing bridge, it was the intention to follow the original plan and utilize the reserve ducts and leave it at that. However, after observation of the heavy traffic loads the bridge is subjected to 24 hours a day, and also taking into consideration the improved traffic comfort of eliminating the joint, it was tempting to find out if it was possible to introduce continuity in the main span, since this now is done on the

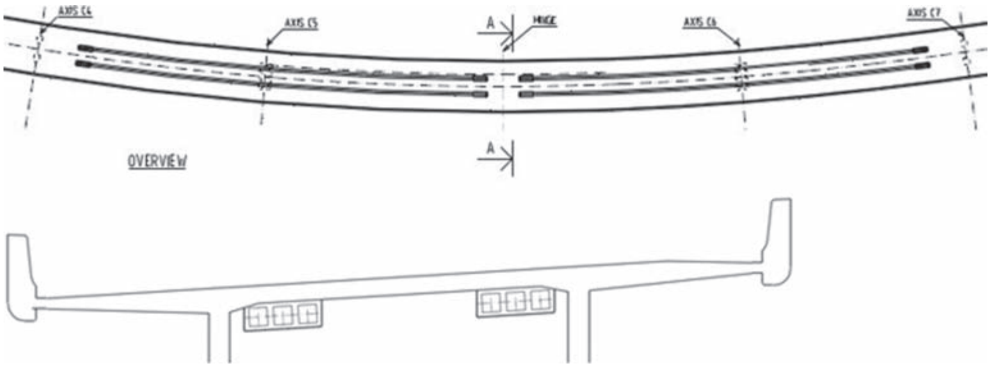

Figure 3. Reserve steel ducts.

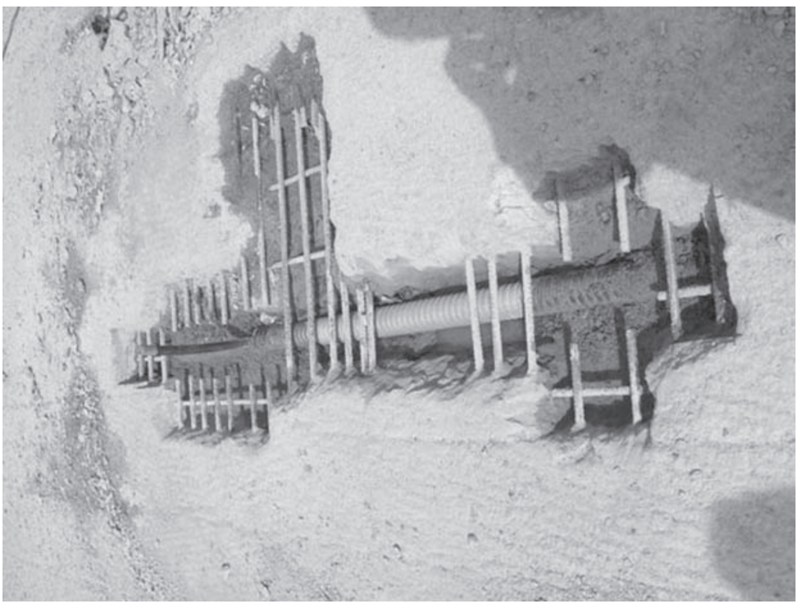

Figure 4. Repair of clogged reserve steel ducts.

new bridge. On the other hand, a change in static system on a bridge is something that must be taken very seriously. Miscalculations can have dramatic consequences. It was decided to perform feasibility calculations on the main columns and foundations, which are supported by vertical concrete piles. If this did not give the desired result, there would be no purpose in further investigating the superstructure. To save time, the model used for global analysis on the new bridge was adjusted to fit the old bridge. The result was encouraging for both foundations and columns since they both seemed to have sufficient capacity for the proposed change in the static system. The substructure behaviour in the flanking spans was not affected since they have sliding bearings. However, the bearings themselves had to be checked for adequacy. Both main span piers are fixed for motion. Since the centre of movement now will move from the main columns to approximately the centre of the main span, the horizontal movements from temperature will increase. Creep and shrinkage effects are both done and will have no consequence on this change. However, the new posttensioning will introduce new creep issues. After investigating the behaviour of the sliding plate on all the bearings in the flanking spans, it turned out that they all have sufficient capacity to accommodate the increased movement. This is also the case for the expansion joints. Thus the basis for a more in depth analysis including the superstructure was justified.

4 ANALYSIS OF THE SUPERSTRUCTURE WITH THE NEW STATIC SYSTEM

As already mentioned, creep and shrinkage are just about done and their contribution to the design forces could therefore be omitted. Traffic loading and temperature would therefore be the dominating factors in the design. What also had to be included was the effect of creep caused by the change in static system. The new forces in the middle part of the main span required 7 external cables positioned as close to the top surface of the bottom of the concrete box section as possible. Each cable had to be stressed 2300 kN. Figure 5 shows the layout of the external cables. Since the external cables must be straight, and the bridge itself has a horizontal curvature, it was not possible to achieve complete symmetry. It was important that the blisters, where the tensioning takes place, were placed in the compression zone of the new static system. From a practical point of view, it would have been easiest to tension all 7 cables in one section. However, this concentrated force would be too large and the risk of cracking the concrete too high. Therefore, it was chosen to distribute the tensioning to 2 sections. This would have been a more conservative detail if the bridge was a new design and therefore it was even more important to do it in this case.

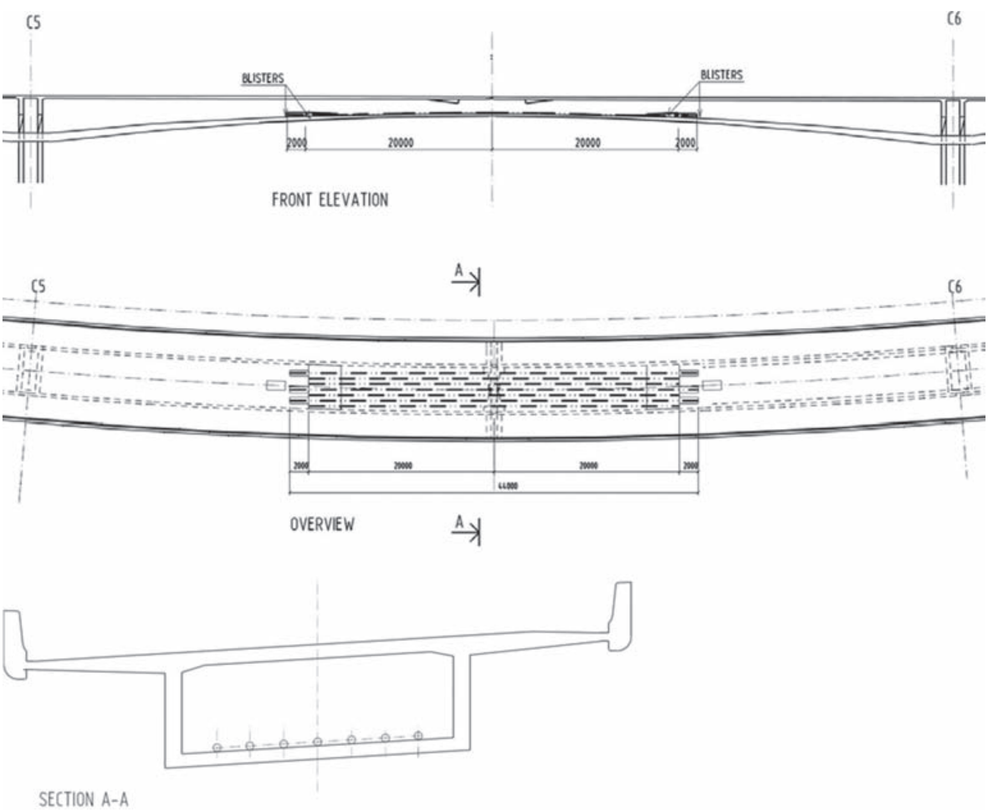

Figure 5. Layout of external cables in bottom slab.

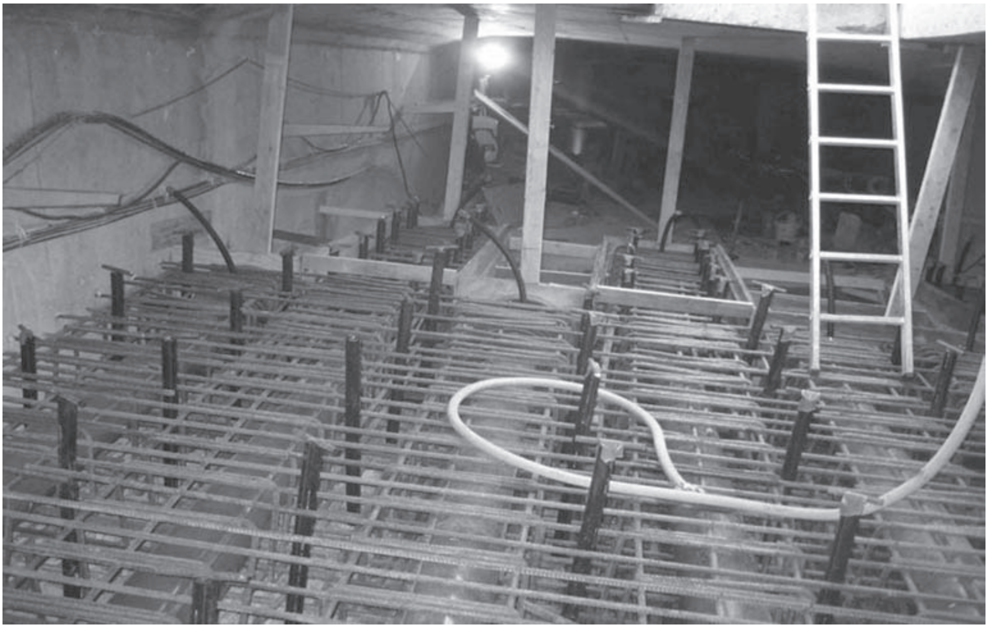

Figure 6. Steel bars and reinforcement in blisters.

The horizontal tensioning force must be transferred to the bridge through friction. It therefore became necessary to drill a substantial number of vertical holes for steel bars through the bottom slab of the concrete box beam section where the blisters were placed for later tensioning. Drilling for dowels also became necessary in order to satisfy the requirement for reinforcement in a horizontal construction joint. However, these holes did not go all the way through the bottom slab and stopped before they reached the bottom reinforcement layer. This operation was not done until the top reinforcement layer in the bottom slab was exposed by chiselling. This prevented any unintentional cutting of this reinforcement. Figures 6 and 7 show the blister areas with steel bars and reinforcement.

5 CHISEL PLAN

A chisel plan had to be worked out for the blisters in the bottom slab and also for removal of concrete in the top—and bottom slab of the box girder cross section over a length of 4 m in the middle of the main span. The 4 m length was required to obtain sufficient lap length for the new reinforcement to achieve continuity in the main span. Figures 7 and 9 shows the chisel plan.

Removal of concrete in the hinge area did not include the box beam webs. It was first of all the top—and bottom slab and the 2 cross beams in the centre. The thickness of the webs in the hinge area is somewhat thicker in order to be able to install the steel pendulum in each web. Figure 8 shows the upper part of a steel pendulum and the cross beams after the chiselling had been going on for some time.

At an early stage of the planning it was considered to remove the steel pendulum since it became only a foreign element after the main span was made continuous. However, it was found to be more beneficial to keep the steel pendulums due to increased safety during chiselling and also when connecting the two cantilevers. From a practical point of view, it became

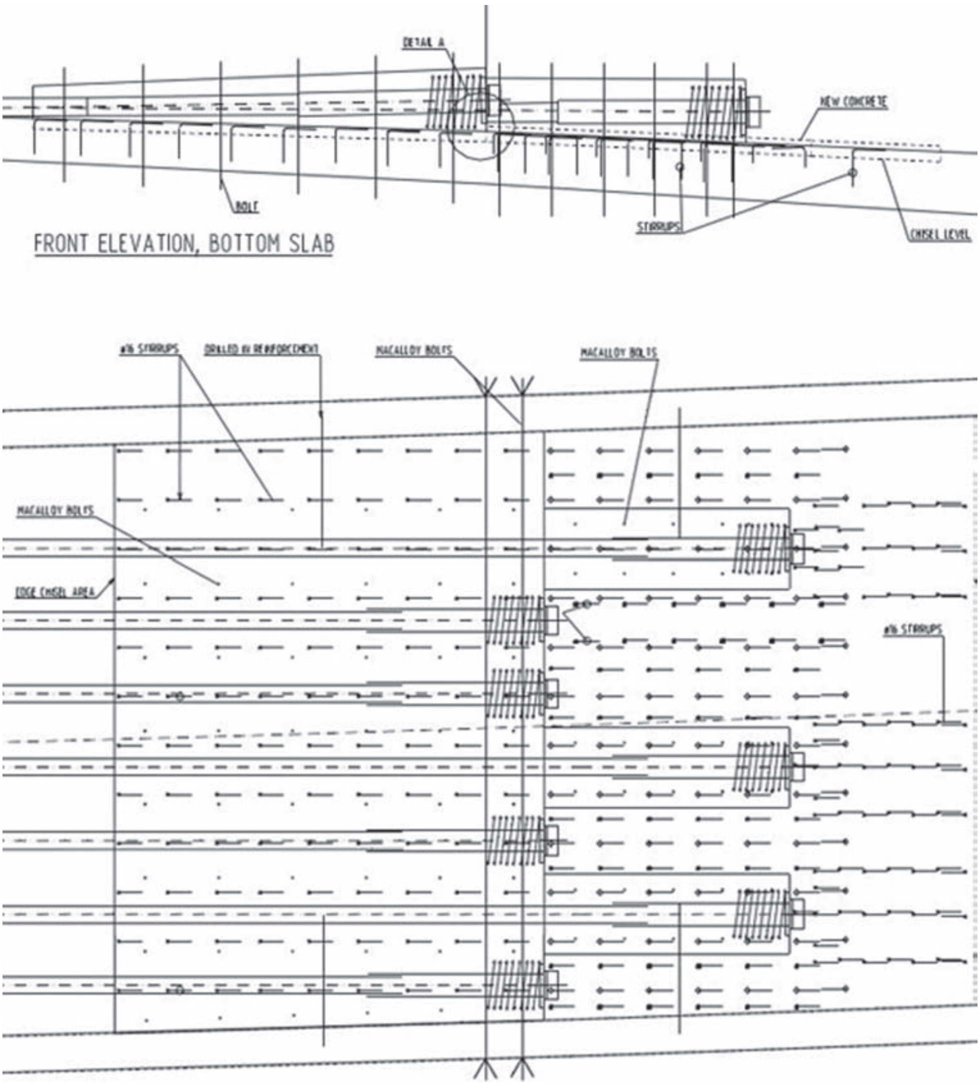

Figure 7. Blister placement and chisel layout details.

almost impossible to drill holes for dowels on the vertical areas surrounding the pendulums, but chiselling was considered to be sufficient with regard to necessary adhesion between new and old concrete. The pendulums did not have any practical consequence that would impair installation of new stirrups in the webs.

When building the main span with the cantilever construction method, all the cables in the top slab ends up in the web area for tensioning. By temporary removing the concrete in the top—and bottom slab in the hinge area as shown on Figure 8, the position of the split forces from existing cables will move forward to a location where the whole cross section still is intact. However, in this case it was not sufficient reinforcement in this area to accommodate these forces. The problem was solved by installing carbon fibre reinforcement plates (CFRP) in a certain area in the transverse direction of the bridge. This work was done before the chiselling work started. Figure 9 shows the position of the plates and Figure 10 how they look after being installed.

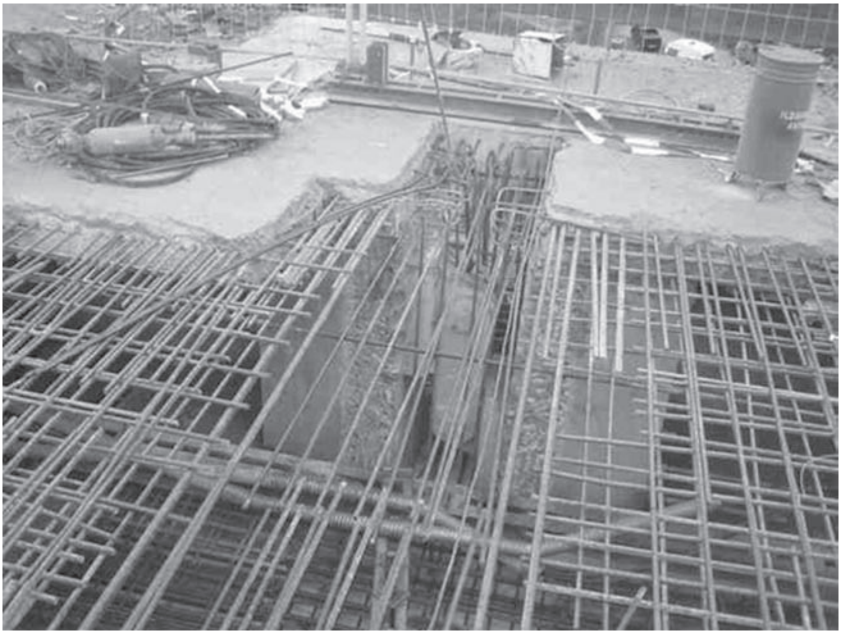

Figure 8. Chiselling in the hinge area.

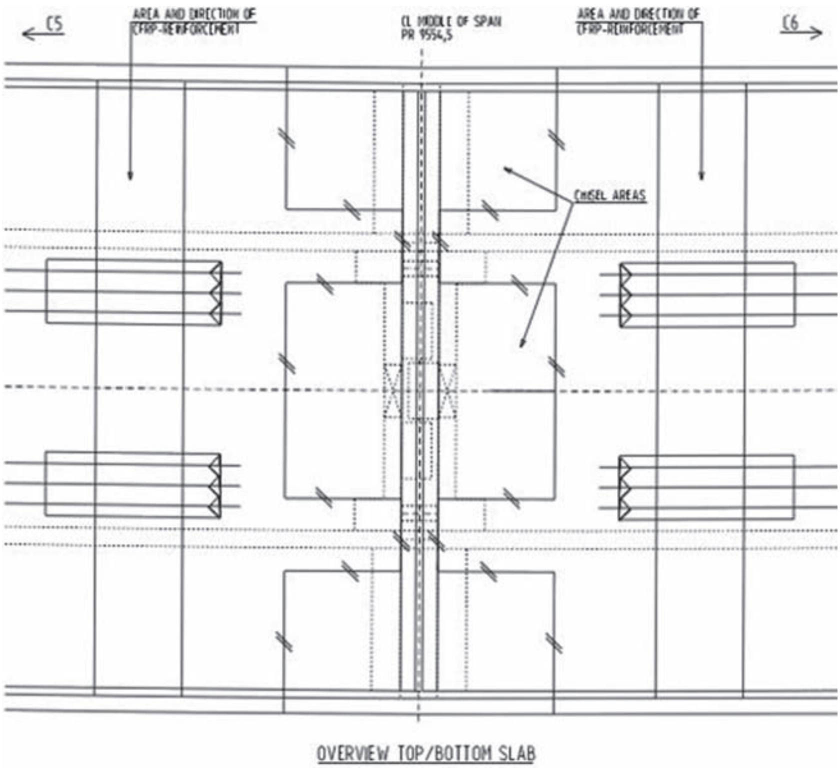

Figure 9. Position of carbon fibre reinforcement plates.

Figure 10. Carbon fiber reinforcement plates.

Figure 11. External cables in position.

The extent of chiselling work for the blisters in the bottom slab also included a certain area behind the blisters themselves. This to ensure that the split forces behind the blisters arising from the tensioning have the necessary reinforcement in place.

When chiselling for the blisters in the bottom slab, concrete was removed to a depth such that all the top reinforcement was visible. This implies that the bottom slab in this phase of construction will be impaired since the end moments in transverse direction at the webs no longer have sufficient reinforcement available. However, if a plastic hinge should develop by the webs, there is sufficient reinforcement in the middle of the bottom slab to accommodate the increased moment. Precautions were taken to limit heavy equipment operation in this area for as long as the cross section was impaired.

6 SEQUENCE OF WORKING OPERATIONS

The existing bridge was closed for traffic during the work operation. The sequence of working operations for a project such as this one is very important. A primary objective of the work was to lift the cantilever arms as much as possible before they were cast together, thereby making a continuous static system. This means that in addition to stressing the reserve cables, as much weight as possible had to be removed. A typical example here is the parapets that in any case had to be replaced. Since a continuous system is much more rigid with regard to deflections than a cantilever system, the new parapets were cast after the continuous system is in place. In 60 m of the main span only, lightweight concrete together with 2 empty ducts with diameter 80 mm in each parapet were used to save even more weight. The coupling of cantilever arms was done when as much concrete as possible had been removed in the hinge area and simultaneously the parapets were removed in the main span. The casting in the hinge area had to be carried out in two stages.

First, only the cantilever parts of the top slab of the cross section were cast. After that, the lower part of the cross beams in the hinge area could be removed. The reason for not doing everything in one operation, is the two vertical bearings positioned in this part of the cross beams. Their task is to accommodate forces in the transverse direction of the bridge and until the continuous system is established, they must be in place. The first casting operation established a connection in the longitudinal direction of the bridge. However, even after curing of the concrete, the lever arms moved a little bit up and down with temperature changes. Full continuity was not achieved before the whole cross section in the hinge area was cast. At the end the cantilever arms were raised ca 120 mm.

The following details the sequence of the major working operations:

- Removal of guardrail and parapets in the main span
- Strengthening of top slab with carbon fibre plates in certain areas
- Removal of concrete by manual chiselling in top of bottom slab in areas where the blisters are located
- Drilling of holes through the bottom slab in blister areas for steel bars and also through the webs. Also drilling for dowels down to the bottom layer of reinforcement in the bottom slab in the same areas
- Placing of reinforcement and grouting of dowels in the new blister areas
- Placing of plastic ducts in blister areas for later installation of steel strands. At the end of the tubes, anchorages were installed for tensioning of the strands
- Placing reinforcement in the blisters themselves and setting formwork
- Simultaneously with the work for the blisters in the bottom slab, concrete was removed by high pressure water chiselling in the top slab in the hinge area. Concrete in the cross beams was removed down to about 100 mm above the location of the two vertical bearings, being there for accommodate forces in the transverse direction
- The reserve cables were tensioned
- A horizontal bracing was established on the surface of the joint to resist both tension and compression
- Reinforcement was placed in the cantilever parts of the top slab
- Casting of the cantilever parts on a rising temperature to avoid cracking.
- No chiselling was allowed until the concrete in the cantilever parts has reached a compression strength of 35 MPa

- Removal of concrete in the bottom slab and the remaining part of the cross beams by water chiselling and sawing
- Placing of reinforcement in top—and bottom slabs
- Casting of top—and bottom slab on a rising temperature to avoid crcking.
- Casting of all the blisters
- Tensioning of steel bars in the blisters
- Splicing of plastic ducts for external cables on top of the bottom slab with the ducts already in position in the blisters.
- Installation of strands in the ducts. The layout of external cables is shown on figure 5 and figure 11.
- Tensioning of external cables
- Injection of reserve cables by mortar
- External cables are injected by grease

Surfaces where ends of reinforcement bars are exposed are sealed with epoxy.

7 CONCLUSIONS

It is always difficult to anticipate all of the problems that will come up during this type of work. When drilling for all of the reinforcing steel, an effort was made to reduce cutting the existing reinforcement as much as possible. It is important to monitor the work, since mistakes can easily be made and the consequences can be serious. Also, caution had to be exercised when chiselling with regard to all the cables in the top slab. The total sag is today reduced from ca 200 mm to 80 mm. No cracking has been observed so far. The behaviour of the bridge is now much better regarding traffic comfort and lower traffic noise. The flanking span bearings and joints seem to perform as expected. The modifications have not effected the load rating of the bridge. All together the final result looks good, and hopefully the lifetime of the bridge is increased.

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Chapter 16

Deck truss bearing rehabilitation for the Benjamin Franklin Bridge

Q. Ye & Q.V. Nguyen
Weidlinger Associates, Inc., New York, NY, USA

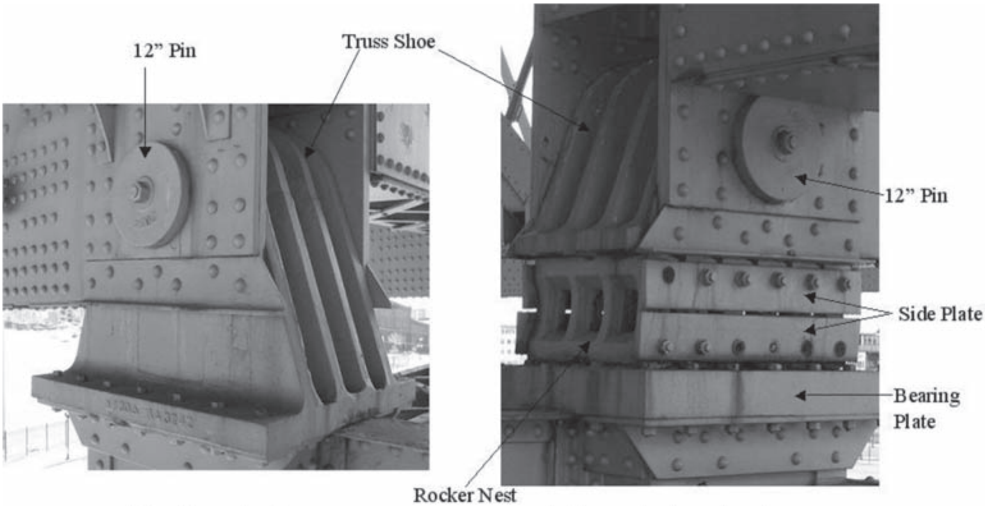
E. Montgomery & M. Venuto
Delaware River Port Authority, Camden, NJ, USA

ABSTRACT: The existing steel truss bearings of the approach truss spans of the Benjamin Franklin Bridge have severe corrosion problems and need retrofit. Four different retrofit options, such as replacing steel rocker nests only or installing seismic isolation bearings, were evaluated based on costs, as well as seismic performances. After comparing pros and cons of all four options, it was found that the seismic isolation option would significantly improve the structural performance and was recommended for design. Lead-core elastomeric bearings were selected for the final design based on their low costs, and good performance. The bearing replacement work is scheduled to start in the spring of 2011.

1 INTRODUCTION

The Benjamin Franklin Bridge, owned by Delaware River Port Authority (DRPA), is a suspension bridge that spans the Delaware River. It was opened to traffic on July 1, 1926. The bridge carries vehicular traffic, pedestrian traffic and PATCO trains. The Camden approach consists of seven truss spans (Figure 1), ten girder spans, twelve stringer spans and an

Figure 1. Deck truss spans.



a. Fixed truss bearing b. Expansion truss bearing

Figure 2. Fixed and expansion truss bearings.

abutment. The Philadelphia approach is comprised of five truss spans, nine girder spans and an abutment.

The existing expansion and fixed steel bearings (Figure 2) in the deck truss spans of both Camden and Philadelphia approaches were found by previous biennial inspections to have severe corrosion problems: broken stud bolts; rusted side plates and rockers; heavy pitting in bearing plates. There was also a concern about the rocker nests being "frozen" from corrosion, thus losing their expansion capability. Therefore, the rehabilitation of these rocker nests has been identified as one of the priority repairs by the biennial inspection reports. Recommendations have been made in these reports to clean the rocker nests and replace the broken stud bolts and corroded side plates.

To address these bearing repairs, as well as other steel repairs in the deck truss spans as recommended by the inspection reports, DRPA contracted Weidlinger Associates Inc. to undertake this deck truss rehabilitation project. The bearing rehabilitation work includes:

1. Field inspection and testing
2. Evaluation of different retrofit methods and recommendations
3. Final design of the selected retrofit option

Even though this project was not a comprehensive seismic retrofit project, it was part of the scope to improve the seismic performance of the bridge, especially the substructures. Seismic loads were determined using response spectra from AASHTO Standard Specifications 2002, corresponding to earthquakes with return periods of approximately 475 years and an acceleration coefficient of 0.125 g. The Benjamin Franklin Bridge, which is classified as an essential bridge, bears an importance classification of I.

2 FIELDINPSECTION AND TESTING

2.1 *Fixed bearings*

The fixed bearings in general are in good condition. There were, however, some slight deterioration of the shoes (<5%) and visible rust stains on the surfaces. Most of the bolts connecting the shoe to the top of the pier bent have varying degrees of head loss. This head loss

Figure 3. Truss pin inspection—surface corrosion behind pin cover.

varies from 10–25% in most cases but there were locations where head loss was up to 75%. The exterior surfaces of the cast pin covers are in good condition but areas of the gusset plate around the pin cover have minor to moderate pitting at many locations (Figure 2a). Areas of deeper pitting were observed at other locations.

2.2 *Expansion bearings*

The expansion bearings (Figure 2b) are in poor condition: heavy pack rust has built up behind many of the side plates, pushing them outwards and breaking many connection bolts; furthermore, the rocker nests themselves have heavy corrosion at their bottom surfaces that are in contact with the steel bearing plates underneath; dirt and debris built up inside the rockers trap water, further acerbating the corrosion problem; the bearing plates also show heavy pitting and section loss due to corrosion; head loss of up to 50% was observed on the bolts connecting the bearing plates to the top of the pier columns.

Thermal offsets at the expansion bearings were recorded during the first phase of the field work in March 2009. During the return trip in July, the thermal offsets were taken so that the measurements can be compared to the original recorded measurements and temperatures. The expansion bearings, or at least some of them, move under temperature rise. The magnitudes of movements are not uniform. There is a large scatter, possibly caused by different factors such as uneven temperatures, different levels of corrosion in bearings or the interactions between bearings and pier bents.

2.3 *Truss pins*

The truss pins are 12 inches in diameter and about 2 feet 9 inches in length. For each pin, there are two covers and a steel rod with nuts for keeping the pin in place. A total of eight pins were inspected and evaluated by ultrasonic testing. During the inspection and ultrasonic testing, the pin covers were removed, revealing heavy corrosion and pitting on the side surfaces of the pins (Figure 3). The ultrasonic testing did not find severe flaws in the pins, even in the areas under and behind the gusset plates, where either high contact stress or great potential of corrosion exists.

3 STRUCTURAL DEFICIENCIES OF EXISTING CONDITIONS

To arrive at appropriate recommendations for bearing rehabilitation or retrofitting, it is important to study the behaviors of the existing bearings, pier bents and transverse frames under dead, live, wind and seismic loads, and identify deficiencies. To fulfill the aforementioned objective, the following evaluations were performed:

– Effects of "frozen" rockers
– Effects of "frozen" pins
– Pier Bent Overturning Analysis
– Overturning of Fixed Bearing at Anchorages
– Out-of-Plane Bending of Gusset Plates

3.1 *Effects of "frozen" rockers*

One of the key issues of this study is the condition of the rocker nests. As previously mentioned, the bottom portions of these rocker nests and their bearing plates are heavily corroded. If these members are fused together by corrosion and freeze longitudinal bearing movements, high stresses would occur in truss members as the deck trusses expand or contract as the temperature rises or falls.

The two-dimensional (2D) SAP2000 (Computers and Structures, Inc)models used for the live load analysis were modified to simulate the behaviors of "frozen" rockers. From the analysis of the deck trusses with "frozen" rockers, it was found that each bearing would develop a longitudinal force in excess of 200kips due to the maximum design temperature rise. This force is too large to be developed in the rocker nests since there are very small contact areas between the rockers and the plates, above and below (Figure 4). In addition, Weidlinger's field inspection team noticed paint cracks in the newly painted rocker nests, indicating there were movements at these locations. Therefore, it is believed that the rocker nests are not "frozen", at least not completely.

3.2 *Effects of "frozen" truss pins*

Similarly, if corrosion along the surfaces of truss pins fused the pins to truss shoes, the deck trusses could be restrained from rotation at their two ends, resulting in additional stresses in truss members. An evaluation of the pin condition is necessary to determine the additional forces that might exist in bottom chords, diagonals and verticals that are directly connected to the pins.

The bridge, built in the mid 1920's, is approximately eighty years old. At this age, it is reasonable to assume that the coating of white lead and tallow applied on the pins at the time of their installation has worn away. With this in mind, and knowing that the coefficients of static friction in clean steel-to-steel contact surfaces range from 0.3 to 0.7, an analysis was needed to investigate the frictional forces that would be developed under live loads, and evaluate whether the pins would rotate under such forces.

The 2D finite element model of the Camden inner truss was modified to simulate "frozen" pins on the bridge. All the pins were restricted from rotation, while the rocker nests were assumed to be able to move freely. Figure 5 shows the moment diagram in a fixed bearing

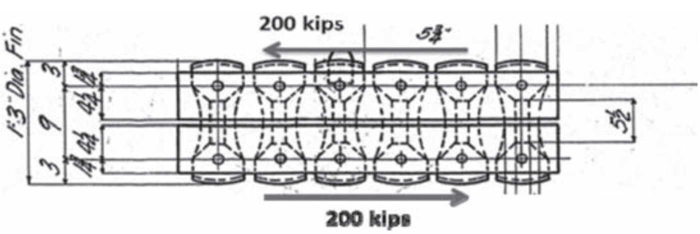


Figure 4. Longitudinal force couple in rocker nest.

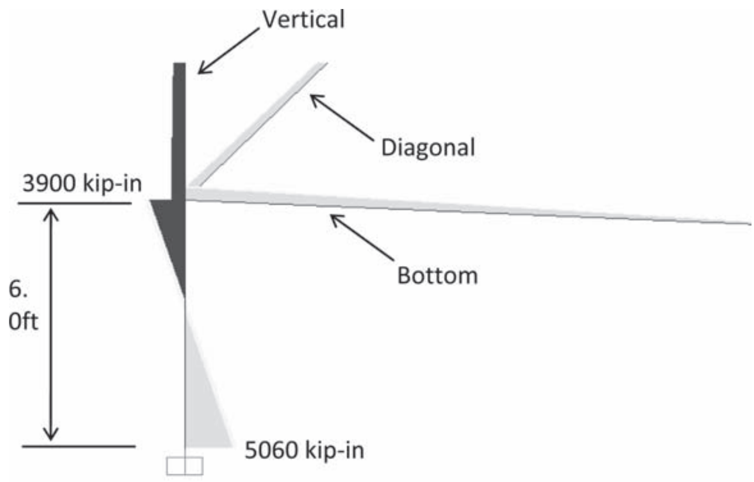


Figure 5. Moment diagram at "frozen" pin at anchorage.

at one anchorage and adjacent trusses under the condition of "frozen" pins. The bending moment caused by a combination of truck and train loads with impacts is 3900 kip-in.

This moment (3900 kip-in) will require a total of approximately 650 kips frictional force around the circumference of the pin to keep it from rotating. The maximum combined vertical service load in each pin is 745 kips, and the minimum is 621 kips. Therefore, the needed friction coefficient to restrain pin rotations ranges from 0.87 to 1.05. Thus, it is possible that the pin would rotate under a moment of this magnitude. However, considering the contact surfaces between pins and truss shoes may no longer be clean, it was decided to check the truss members assuming fully "frozen" pins.

Dead loads were applied before the pins were "frozen", since they were applied when the pins were new and free to rotate. To compare with the ideal condition in which pins are free to rotate, the moments and forces of truss members were obtained from a model with freely rotating pins and stresses were calculated. It was found that "frozen" pins will cause additional bending stresses in bottom chords and verticals, while reduce quite significantly the axial force in bottom chord members due to change of boundary conditions, ending up with approximate the same amount of total stresses.

3.3 *Pier bent overturning analysis*

In additional to the above truss members, pier bents are also significantly affected by bearing behaviors, since they are also in direct connection with bearings. Two types of analyses were conducted: stress analysis and overturning analysis. Stress analysis checks the adequacy of pier bent sectional sizes, while the overturning analysis checks the overall stability of the pier and the bridge. In addition, the masonry piers were checked for seismic uplift forces.

The wind loads and seismic loads in the bearings were obtained from three-dimensional (3D) finite element models of the approach spans (Figure 6). The magnitudes of seismic loads vary from pier to pier. However, in average, the longitudinal seismic force is 200 kips for each fixed bearing and the transverse force is 100 kips for each fixed or expansion bearing.

A stress analysis was conducted on a 3D ANSYS (ANSYS Inc) model of Pier Bent #1 in the Camden approach (Figure 7), the tallest and most critical pier. This pier bent was checked for two combinations of dead and seismic loads: one in the longitudinal direction; and the other transverse. Results from these analyses show that both the stresses and deflections of the pier bent are acceptable.

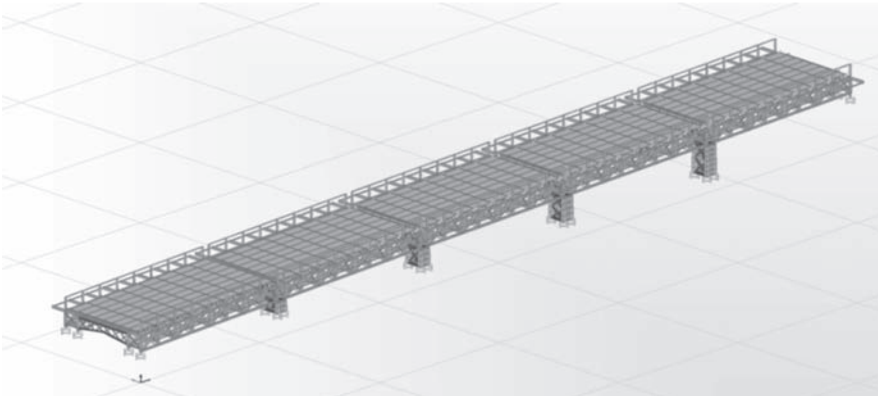


Figure 6. 3D Finite element model for truss spans in Philadelphia approach.

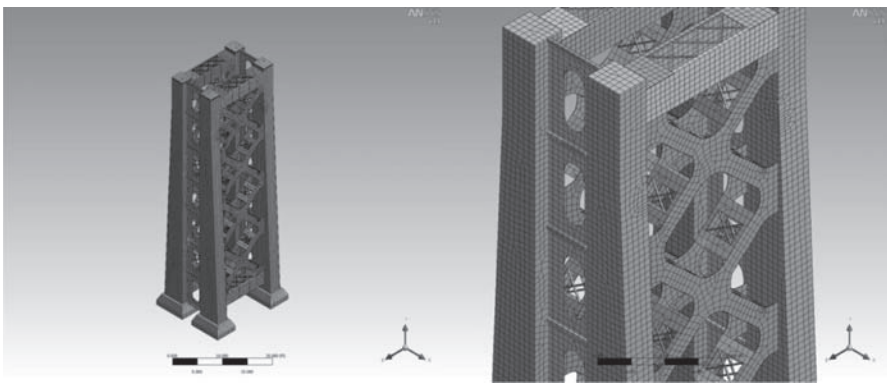


Figure 7. 3D Finite element model for pier bents.

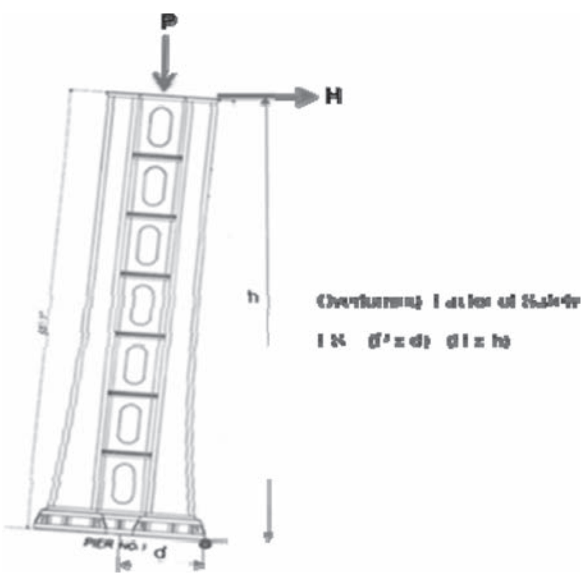


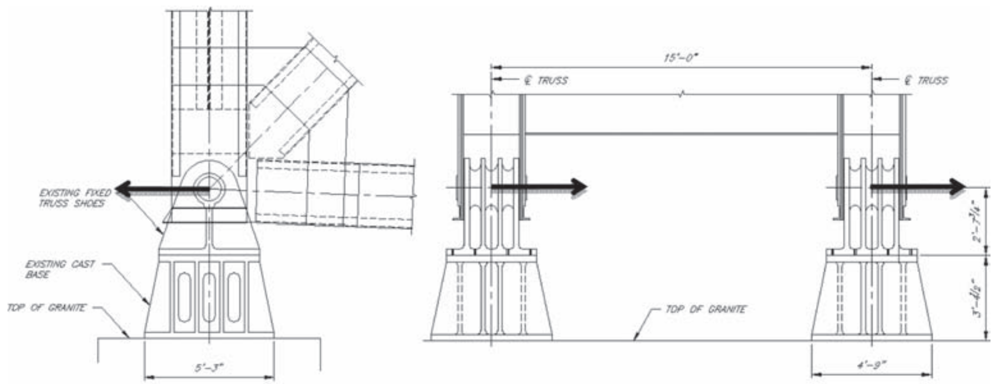
Figure 8. Overturning of pier bents.

Table 1. Pier bent safety factor against overturning—Philadelphia approach.

Force direction	Pier 1	Pier 2	Pier 3	Pier 4
Longitudinal	0.91	0.88	1.27	2.03
Transverse	0.88	1.01	1.43	2.08

Table 2. Pier bent safety factor against overturning—Camden approach.

Force direction	Pier 1	Pier 2	Pier 3	Pier 4	Pier 5	Pier 6
Longitudinal	0.73	0.75	0.79	0.89	1.20	1.78
Transverse	0.75	0.75	0.97	1.20	1.42	2.18



a. Longitudianl elevation b. Transverse elevation

Figure 9. Fixed bearing at anchorages.

The pier bent overturning analysis was performed under vertical dead load reactions and horizontal seismic loads, and the factors of safety (FS) against overturning were calculated (Figure 8). A FS less than 1.0 means instability and imminent collapse of the bridge. In general, the FS should be as high as 1.5 to ensure an adequate safety margin against collapse. Tables 1 and 2 shows the FS against overturning for the Philadelphia and Camden approaches, respectively. As shown, there are many pier bents having FS less than 1.0 in both the longitudinal and transverse directions, and only a few of them have FS larger than 1.5. Pier 5 in the Philadelphia approach and Pier 7 in the Camden approach are not included in the table, but they are believed to have adequate FS against overturning since they are the shortest pier in each approach.

The above findings clearly indicate that the pier bents need to be isolated from seismic loads in both longitudinal and transverse directions to increase the FS against overturning. The latter can be accomplished by replacing the existing steel bearings with seismic isolation bearings, which can significantly reduce the seismic loads.

3.4 *Fixed bearings at anchorages*

The integrity of the fixed bearings at the anchorages was also evaluated, as it sits atop a relatively high pedestal (Figure 9). Table 3 shows the FS against overturning for these bearings. As shown, these bearings are fairly stable in the transverse direction; while in the longitudinal

Table 3. FS against overturning of fixed bearing at anchorages.

Force direction	Philadelphia anchorage	Camden anchorage
Longitudinal	0.82	0.81
Transverse	2.08	2.01

direction, the FS against overturning is around 0.8. The best way to reduce the seismic forces is to replace these fixed bearings by seismic isolation bearings. These bearings, which diffuse the seismic forces by yielding their lead cores or breaking their fuses, will increase the factor of safety against overturning during a seismic event to be above 1.5.

3.5 *Out-of-plane bending of gusset plates*

As shown in Figure 9 above, each existing truss-to-bearing connection consists of gusset plates connected to the pin on both sides of the truss shoe. The total thickness of gusset plates is 2 ½” on each side, which is adequate for transferring vertical loads. However, they may not have adequate strength in the transverse direction since they do not have stiffeners for the out-of-plane bending. These gusset plates shall be checked for the transverse design wind load, which is about 40 kips per bearing, and design seismic load, about 100 kips per bearing.

4 BEARING RETROFIT OPTIONS

Four retrofit options have been considered for the existing steel bearings,each of which has its advantages and disadvantages. They were evaluated by cost, seismic performance, and means of installation. The four options are:

1. Cleaning of existing bearing
2. Replacement of rocker nests in expansion bearings
3. Replacement of rocker nests and all truss pins
4. Seismic isolation, meaning replacement of all steel bearings with seismic isolation bearings

4.1 *Cleaning*

Previous inspection reports have recommended cleaning the existing expansion bearings by sandblasting the exterior surfaces of the rockers, truss shoes and bearing base plates. Recommendations extend the rehabilitation to replacing missing or corroded stud bolts in the bearing assemblies. Under this option, these bearings would only be cleaned and painted, rather than replaced. Therefore, temporary supports or jacking frames will not be needed, resulting in relative low construction costs. However, the seismic performance of the bridge will remain poor. Additionally, cleaning could be very difficult due to restricted access to the interior of rocker nests. As a result, the interior surfaces of rocker nests would not be cleaned, and the heaving pitting in the bearing plates would also remain. In short, the existing rocker nests will continue their deterioration under this option.

4.2 *Replacement of rocker nests*

Heavy corrosion and pitting in the rocker nests and bearing plates indicate these elements are beyond their service life and should be replaced, while the truss shoes may remain. The replacement bearings could be elastomeric bearings (Figure 10) or disc bearings. Both of

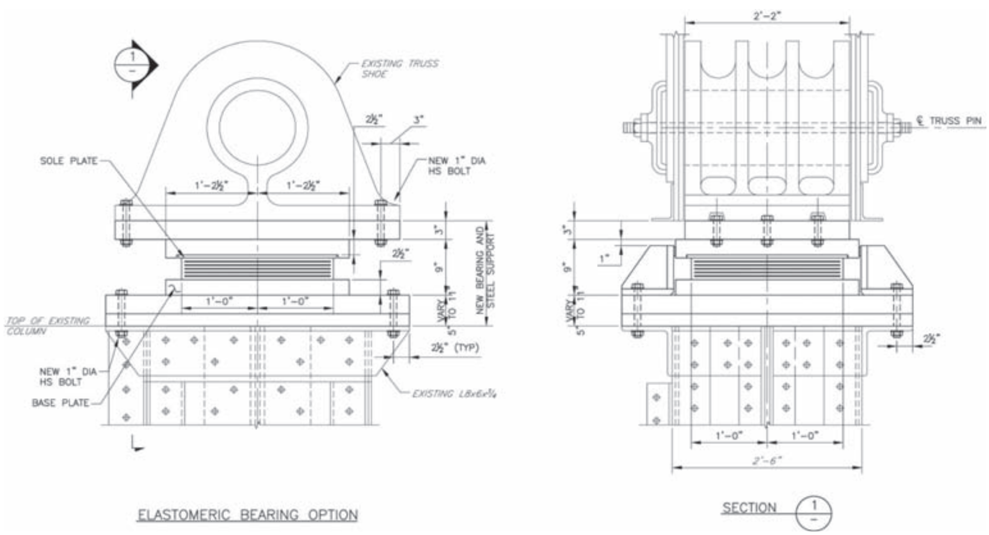


Figure 10. Elastomeric bearing replacement.

them will slide in the longitudinal direction as the temperature rises or falls, while they will be restrained in the transverse direction by stoppers or guides. The unit price of elastomeric bearings is around $3,000 and that of disc bearings is $7,000. The performance and service lives of both bearing types are considered to be similar. Therefore, the elastomeric bearings are assumed for this option.

This option will greatly improve the service performance of expansion bearings under dead, live and thermal loads. However, it will not appreciably enhance the seismic performance of the bridge.

Temporary jacking could be performed ononly one side of the pier bents for replacing the rocker nests. However, there will be eccentricity of the bearing reactions from both dead loads and live loads. To minimize these eccentricities, it is recommended to jack the trusses on both sides of pier bents.

4.3 *Replacement of rocker nests and truss pins*

This option is similar to the 2nd option, buthas the addition of replacing the truss pins, which were made of forged Grade 35 carbon steel. This option was considered prior to the analysis of "frozen" pins. From the analysis, there are only small increases of stresses in truss members under the "frozen" pin condition. In addition, the ultrasonic testing of truss pins did not reveal significant flaws in them, except a few unconfirmed indications in several pins. Therefore, there is no reason for replacing them. However, in any case, since the trusses on both sides of a pier bent will be jacked up during the replacement of rocker nests, it will be very easy to remove the existing pins and install new ones. Therefore, the extra cost to replace the truss pins will be moderate, but all the corrosion issues and the uncertainties in the condition of existing pins will be eliminated. Again, there will not be significant seismic enhancement from this Option.

4.4 *Seismic isolation*

The last option would be replacing all the bearing assemblies with seismic isolation bearings. Three different seismic isolation bearings were evaluated for this option: friction pendulum bearings, disc isolation bearings and lead-core elastomeric bearings.

Friction pendulum bearings are self-centering after a seismic event and have proven to be very reliable. They are, on the other hand, very expensive, costing upwards of $12,000 each.

Disc isolation bearings are also expensive, with estimates as high as $17,000 for each bearing. In addition, the physical size of this type of bearings is much bigger than others in order to accommodate the mass energy regulator (MER) assemblies.

Lead-core bearings (Figure 11) are viewed as a classical way of isolation. During an earthquake, the lead core with low yielding strength under rapidly applied seismic forces will yield. Once the lead core yielded, the isolator has very small horizontal stiffness. This will help to lengthen the period of the bridge, and consequently reduce seismic forces. Displacement of the bridge superstructure is limited to an acceptable level by hysteretic damping of the bearing. Hysteretic damping also contributes to the reduction of seismic forces.

Lead-core bearings have a proven track record as they have been around many years and have been used on many bridges with no reported problems. The unit price for lead-core bearing is about $6,000. Restoring force is not a major concern considering low frequency and low-to-moderate magnitude of earthquake events in this region. Therefore, lead-core elastomeric bearings are recommended for this option.

4.5 *Summary and recommendations*

The pros and cons of the above four options are summarized in Table 4. The service and seismic performances increase from Option 1 to Option 4, while the costs also increase. None of the options, except Option 4 (Figure 12), can reduce the seismic vulnerabilities of the pier bents, masonry piers and fixed bearings at anchorages.

Option 4,using seismic isolation bearings will significantly improve the structures performance during an earthquake. The costs will be higher than other options, but the

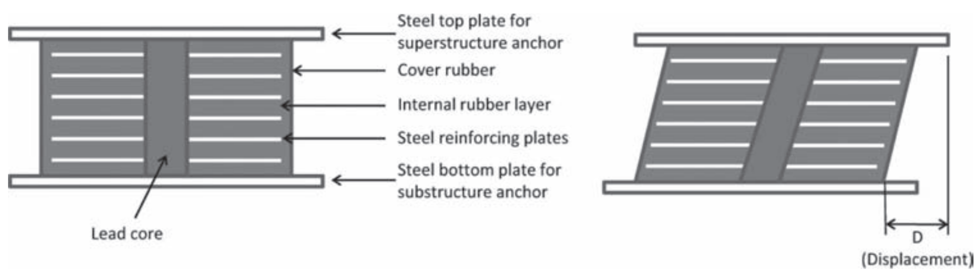


Figure 11. Lead-core elastomeric bearings.

Table 4. Summary of bearing retrofit options.

Option	Task	Cost	Service performance	Seismic performance	Jacking & Temporary supports
1	Clean existing bearings	$250,000	Poor	Poor	Not needed
2	Replace only rocker bearing	$3,700,000	Good	Poor	Needed
3	Replace rockers and truss pins	$4,300,000	Good	Poor	Needed
4	Replace all bearings Assemblies with Seismic bearings	$6,350,000	Good	Very good	Needed

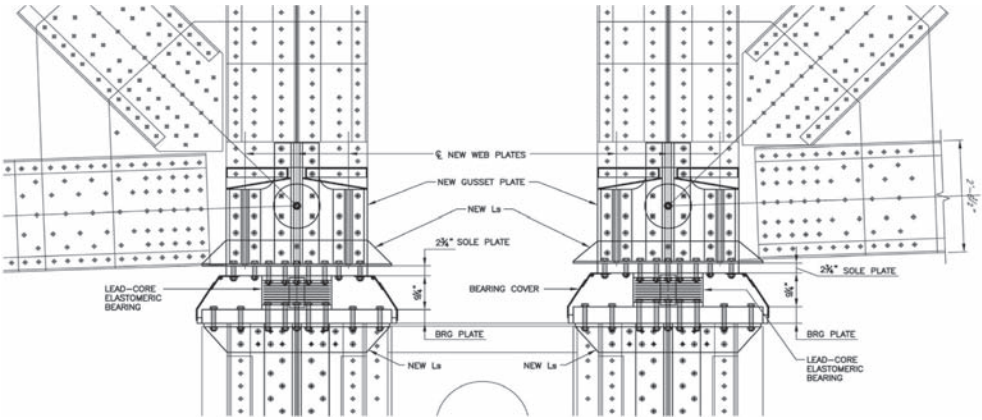


Figure 12. Lead-core bearings on pier bents.

benefits are unparalleled as these isolation mechanisms are seen as the most prudent method to mitigate potentially catastrophic failures during a seismic event. Therefore, Option 4 is recommended for this project.

5 CONCLUSIONS

Based on Weidlinger's field inspection, it was confirmed that the existing rocker bearings are heavily corroded and in need of rehabilitation. However, it is believed that the rocker nests are not "frozen". They will rock back and forth as temperature rises or falls.

During the inspection and ultrasonic testing of eight pins in the Camden Approach, heavy corrosion and pitting on the side surfaces of pins were observed. However, no other severe flaws were found, except for several unconfirmed indications inside several pins. Even though the pins might be "frozen" due to increase of friction over the years, this condition would not significantly increase the stresses in the truss members, and the total stresses are still less than the allowable values.

The stresses in the pier bents are within the allowable values even under seismic loads. However, for most piers, the factors of safety against overturning under these seismic loads are not adequate.

The existing gusset plates transferring vertical loads from trusses to pins are weak in the transverse direction, therefore these plates most likely cannot transfer the lateral forces from either the design wind loads or seismic loads. Strengthening of these gusset plates might be required.

Four bearing rehabilitation options have been studied, which arecleaning of existing bearing, replacement of rocker nests in expansion bearings, replacement of rocker nests and truss pins, and replacement of all steel bearings with seismic isolation bearings. The pros and cons, as well as the costs, of all four options were studied and compared. Option 4, using seismic isolation bearings, will significantly improve the bridge's performance during an earthquake. The costs will be higher than other options, but the benefits are unparalleled during seismic events. Therefore, Option 4 is recommended for this project.

This bridge is probably the first major highway truss bridge in the North-East region to adopt seismic isolation bearings. The knowledge and experience gained from this project is certainly valuable to other bridges in the same region since many of them still have old steel bearings, which are vulnerable to seismic loads.

ACKNOWLEDGEMENT

Delaware River Port Authority is the owner of this bridge and sponsored this exciting project. The writers gratefully acknowledge this support.

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Chapter 17

Low profile uplift bearings for the Atlantic Avenue Viaduct, New York City

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

ABSTRACT: Built in 1901, the Atlantic Avenue Viaduct carries Long Island Railroad's (LIRR) Atlantic Tracks 1 and 2 connecting Jamaica to Downtown Brooklyn. In March 2008, Kiewit Constructors was awarded a $69 million design/build contract to demolish and replace 176 spans and rehabilitate 86 spans of the elevated viaduct. One of the requirements for the rehabilitated and replaced spans is to provide uplift restraint at points of bearing. The designer (HNTB) called for low profile uplift restraint disk type bearings. Well over 700 of these special design bearings have been supplied on this project to date with more in design. The vertical load requirements for these bearings ranges from 160 to 188 kips with horizontal load capacities of 30%. In addition an uplift restraint capacity of 14 to 28 kips was required and due to the low profile limitations a dovetail uplift restrainer was utilized. This paper covers the selection, design, manufacture and testing of these specialty devices with a focus on the benefits these bearings provided the engineers involved in this project.

1 INTRODUCTION

Atlantic Avenue stretches from the Brooklyn waterfront along the East River all the way to Jamaica, Queens and is the sole east-west truck route across Brooklyn. In Flatbush the Atlantic Avenue terminal is the converging point for nine MTA Subway Lines and the Long Island Railroad.

The Atlantic Avenue Viaduct was built in 1901 and carries Long Island Railroad's (LIRR) Atlantic 1 and 2 track connecting Jamaica to Downtown Brooklyn (Figure 1). Time and corrosion have taken its toll on the 100+ year old structure and as a result the MTA and LIRR put out a design/build contract to demolish and replace 176 spans of the viaduct and rehab 86 spans. The successful design/build team was Kiewit along with HNTB. In March of 2008 they were awarded a $69 million Phase I contract. The work includes designing and replacing longitudinal girders and cap beams, bracing, painting, column repairs and installing a fiberglass outside walkway and a center walkway. In addition 10 columns foundations were investigated to determine their condition.

Crews completed demolition and replacement work in 36 hour windows over weekends and were required to have the track operable by Monday morning rush hour. Span replacement took place over 36 weekends. Prefabricated spans were delivered to Kiewit's off site yard where crews placed ties on the new structure prior to installation (Figure 2).

2 BEARING DESIGN

One of the challenges of this project was the transmitting of loads from the superstructure down to the substructure. Typical highway bridges result in horizontal loads that are in the range of 10% of the vertical load. However on rail structures such as the Atlantic Avenue

Figure 1. Atlantic Avenue Viaduct.

Figure 2. Prefabricated spans with ties attached.

Viaduct the horizontal loads approach 30%. In addition certain live loading conditions resulted in uplift loads that ranged from 8% to 18% of the vertical load.

3 BEARING SELECTION

The next challenge the designers were faced with was selecting a bearing system to accommodate the difficult loading schemes in a low profile.

The key was to find a bearing design with a proven track record on rail structures as well as highway bridges. In addition it was important to find a device that has been utilized in the New York City environment.

After a thorough search the engineers at HNTB selected the disk bearing for this demanding project. Disk bearings were developed in the late 1960's as a cost effective means to safely transmit the loads, rotations, and translations of a bridge superstructure to the substructure (Figure 3). The primary component of the disk bearing is the load and rotational element, which is comprised of a polyether urethane elastomer. Due to this material's inherent compressive strength, there is no need for confinement of the elastomer, which acts much like a conventional elastomeric bearing (DeHaven, Western & Watson 2009).

This immediately eliminates the sealing ring problem, inherent with pot bearings. The urethane material used in the disk has outstanding weathering properties and remains stable from –70 to +121 degrees centigrade. Therefore under normal atmospheric conditions there is no problem with the rotational element softening or crystallizing during temperature extremes. The unconfined disk accommodates rotation by the differential deflection of the elastomeric element. The horizontal loads of the structure are transmitted through a shear resisting mechanism (SRM). This ball and socket type connection allows the free rotation of the superstructure up to 4% but inhibits shear from being applied to the rotational element. The standard mechanism accommodates a horizontal force of 10% of the vertical capacity of the bearing device. However this can be easily modified for higher horizontal forces conditions that are commonplace with today's longer spans, curved girders and increased awareness of potential seismic activity.

Now in service for well over 35 years, disk bearings have an outstanding track record on bridges all over the world. One of the reasons for this success is the simplicity in design, which also allows for ease of inspection and maintenance free performance.

Figure 3. Unidirectional disk bearing.

Some notable installations of disk bearings are on the Pasco-Kennewick Bridge in Washington, (Figure 4) the Sunshine Skyway in Florida, (Figure 5) the I-35 W Bridge in Minnesota, (Figure 6) (Van Hampton 2007) the Hoover Dam Bypass (Figure 7) (Cho & Thilmany 2007) and the Manhattan Bridge (Figure 8).

The Manhattan Bridge installation dates back to the Mid 1980's and has some similar loading conditions to the Atlantic Avenue Viaduct since the Manhattan Bridge carries rail as well as vehicular traffic (Watson 2010).

4 ATLANTIC AVENUE DESIGN

Over 700 disk bearings have been supplied to date on the Atlantic Avenue Viaduct. The vertical load varies from 160 to 188 kips which would make these relatively small bearings. However the horizontal loads range from 48 to 57 kips which results in the need for the disk bearings to be modified from the standard 10% design. The SRM, which is typically manufactured from a high strength alloy steel, is then upsized to achieve the level of shear strength required for the high horizontal loads. The guide bars are also strengthened a commensurate amount to ensure load transmission.

One of the extreme design requirements for these bearings is the need to resist the uplift forces. Certain live loading conditions with the trains passing over the spans result in a negative vertical load that the designers wished to accommodate. The conventional disk bearing uplift design results in a modification of the SRM so that it acts much like a trailer hitch on an automobile. By connecting the upper and lower bearing plates with this modified SRM, rotation can occur but uplift is restricted. For expansion bearings the guide bars are also

Figure 4. Pasco-Kennewick Bridge in Washington.

Figure 5. Sunshine Skyway in Florida.

Figure 6. I-35 W Bridge in Minnesota.

Figure 7. Hoover Dam Bypass.

Figure 8. Manhattan Bridge.

modified so that they extend under the upper bearing plate and provide uplift restraint while still allowing translation (Figure 9) (Watson 1986).

Unfortunately all of the uplift modifications result in a bearing with a much higher elevation. Since the Atlantic Avenue Viaduct was a rehabilitation there was limited height requirements for the new bearings and the standard uplift disk bearing design would not fit in the available envelope. As a result the bearing manufacturer's engineers came up with a dove

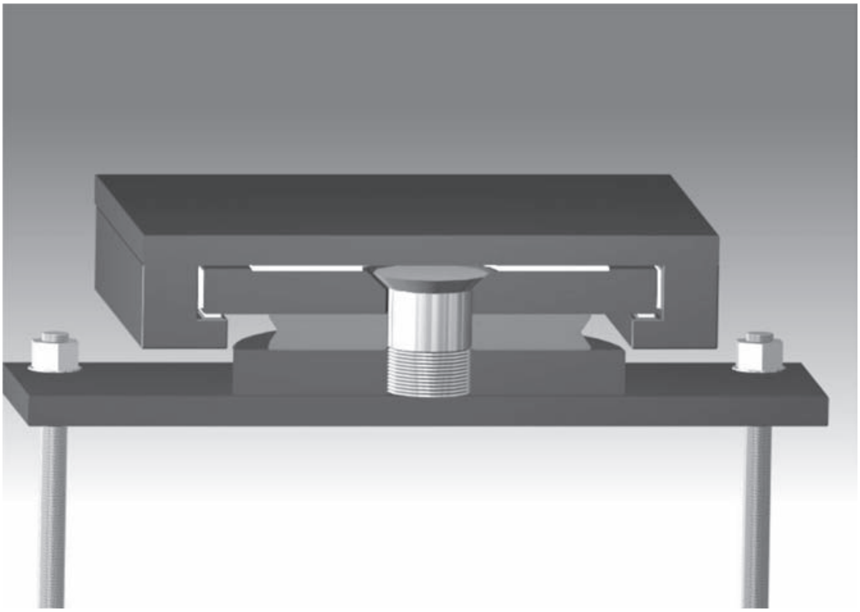

Figure 9. Uplift restraint disk bearing.

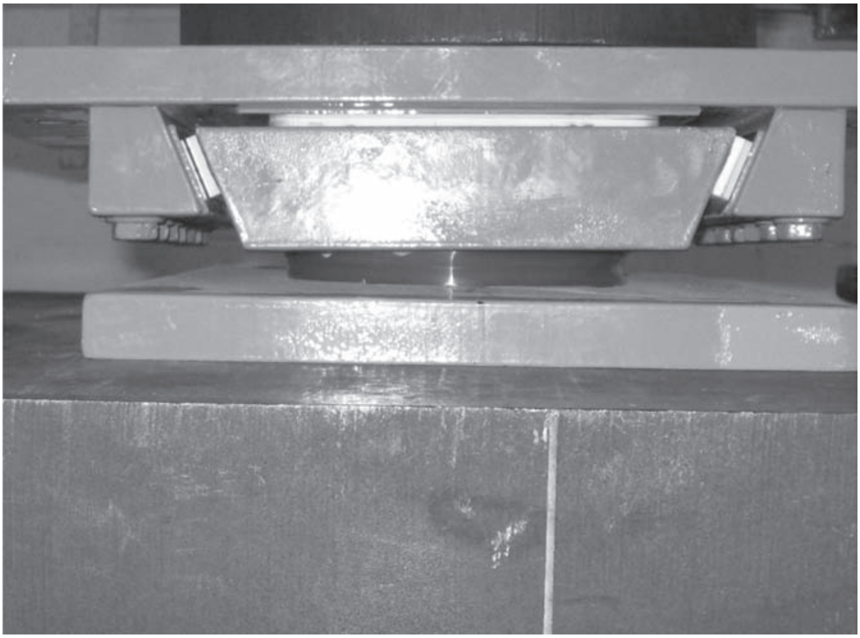

Figure 10. LPUB.

tail guide bar uplift design which reduced the bearing height down to an acceptable level (Figure 10). Extensive laboratory testing was conducted on this modified uplift design to ensure the efficacy of this concept. The low profile uplift design or LPUB solved a significant problem for engineers on this important rehabilitation.

Figure 11. Completed Atlantic Avenue Viaduct.

5 CONCLUSIONS

The Atlantic Avenue Viaduct is a key arterial in the complex New York City Rapid Transit System. Conventional bearing designs were not able to accommodate the challenging load criteria required for the rehabilitation of this essential viaduct. With the disk bearing the engineers at HNTB were able to find a device that was able to meet the challenging criteria. In addition the manufacturer in this case was able to meet the aggressive schedule put forth by Kiewit Constructors with no delays in this important project (Figure 11).

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5 *Bridge replacement & construction*

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Chapter 18

Rapid replacement of bridges using modular systems

B. Sivakumar
HNTB Corp., New York, NY, USA

ABSTRACT: Accelerated Bridge Construction (ABC) applications in the U.S. have developed two different approaches; accelerated construction of bridges in place using prefabricated systems and the use of bridge movement technology and equipment to move completed bridges from an off-alignment location into the final position. Pre-engineered modular systems configured for traditional construction equipment could promote more widespread use of ABC through reduced costs and increased familiarity of these systems among owners, contractors and designers. An objective of the ongoing SHRP2 R04 project, with HNTB as the prime contractor, is to develop "standardized approaches to designing and constructing complete bridge systems for rapid renewals". This paper will discuss the development of modular ABC systems, design of the modular replacement bridge and chronicle the ABC technologies and design innovations. The paper will also review the perceived impediments to greater use of ABC techniques and how standardized modular bridge systems can be a vehicle to overcome these obstacles.

1 INTRODUCTION

Bridge deterioration and the need for replacement continue to be an ongoing problem in the United States. Accelerated Bridge Construction (ABC) techniques have the potential to minimize traffic disruptions during bridge renewals, promote traffic and worker safety, and also improve the overall quality and durability of bridges. ABC has been used on emergency replacement projects as well as the planned bridge replacement project. While most agencies are aware of ABC, very few practice it on a large scale.

ABC applications in the U.S. have developed two different approaches; accelerated construction of bridges in place and the use of bridge movement technology and equipment to move completed (or nearly completed) bridges from an off-alignment location into the final position. Each of these methods is gaining increased acceptance with the use of prefabricated elements to replace bridges in place being somewhat more popular within the industry. The use of bridge movement technology is growing and expected to continue to increase.

Rapid construction of bridges in place offers the promise of limited closures, maybe days or weeks at the most, to allow for the complete construction of a bridge. This type of construction traditionally relies on extensive prefabrication of bridge elements including substructure and superstructure components and the use of cranes to install these elements in their final location, along with innovative contracting and procurement techniques. At that time, special connections designed to integrate the elements into a completed bridge are made and the bridge can be opened to traffic in a very short period of time.

As an alternative to rapid construction in-place is the use of preassembled bridges, completed at an off-alignment location and then moved via various methods into the final location using techniques such as lateral sliding, rolling and skidding; incremental launching; and movement and placement using SPMTs (Self Propelled Modular Transporters). This can be done, depending on complexity, in minutes, hours, or at most several days. Very large and complex structures can be erected in this fashion and the method distinguishes itself from rapid in-place construction most importantly in the scale of the potential project that can be undertaken and by the use of complete fabrication at "off-site" locations.

ABC has yet to gain significant traction in the U.S. Focus group meetings held with representatives from more than 20 DOTs as part of the SHRP2 R04 Project Phase I identified several factors that have contributed to the slow adoption of ABC (HNTB 2009). Despite the gradual lowering of costs and life-cycle cost savings, DOTs are hesitant about using ABC techniques because of their higher initial costs. Utah DOT's ABC projects have demonstrated that with sufficient repetition, precast components can become more economical and their construction more efficient. Transportation agencies are generally risk averse. ABC is perceived as raising the level of risk associated with a project. Designers are also reluctant to suggest innovative approaches due to the added risk. Designers may not see an incentive to be creative under current procurement methods. A resolution to this problem can be found in alternative contracting, specifically using contracting mechanisms that allow engineers and contractors to team up and give the client their best ideas. Such "best value" awards are effective in bringing innovative designs and construction technologies into projects.

There is a culture of using cast-in-place construction for bridge renewals among local contractors. States have also heard complaints from contractors about the diminished profitability of projects using large precast elements (HNTB 2009). Contractors are hesitant to adopt ABC as it involves the use of new technology and want to keep their own employees working rather than subcontracting work to precasters (HNTB 2009). The solution for which is to introduce the industry to precast technology and demonstrate its profitability. With a change of business practices precast construction can even lead to increased profitability. Sufficient repetition is needed to make precast components more economical and their construction more efficient and faster. The only way to build overall efficiency is to build this capability within the local contractors over several projects. This means that for contractor efficiency, rapid bridge replacements must become commonplace. To make ABC projects more economical, agencies should also bundle several bridge sites with similar requirements into a single construction contract.

Part of the mandate of the ongoing SHRP 2 Project R04 titled "*Innovative Bridge Designs for Rapid Renewal*" with HNTB as prime contractor is to develop standard plans and details for promoting more widespread use of accelerated bridge construction. An objective of the ongoing SHRP2 R04 project, with HNTB as the prime contractor, is to develop "standardized approaches to designing and constructing complete bridge systems for rapid renewals". The aim therefore is to develop pre-engineered standards for modular bridge substructure and superstructure systems that can be installed with minimal traffic disruptions in renewal applications. Availability of ABC standards will promote the use of rapid renewal technologies, increase efficiency and reduce costs over time.

2 ABC IMPLEMENTATION

A key objective of this project is to identify impediments and obstacles to greater use of ABC and seek solutions to overcome them. ABC entails prefabricating as much of the bridge components as feasible considering site and transportation constraints. The perceived risks associated with such projects and the need for specialized equipment has led to increased construction costs, which have made bridge owners reluctant to try rapid renewal approaches. Bridge systems that can be built using conventional construction equipment will enable local contractors to bid on rapid replacement projects without outsourcing a large portion of the work to specialty sub-contractors.

A great impediment to rapid construction is the slow engineering process of custom engineering every solution. Rather than custom engineering every solution, pre-engineered modular systems configured for traditional construction equipment could promote more widespread use of ABC through reduced costs and increased familiarity of these systems among owners, contractors and designers. Modular Bridge systems that can be built using conventional construction equipment will also enable local contractors to bid on rapid replacement projects.

Higher initial costs have posed a barrier to introducing new bridge systems that are better suited to rapid renewal projects. The fabricator/contractor is hesitant to invest substantial capital in the equipments necessary for the production of new bridge systems not knowing future demand and whether these equipments will be used in any other bridge projects (HNTB 2009). Therefore, the tendency is to expense the cost of the equipments over the project under consideration instead of multiple projects resulting in substantially increased costs. To insure cost effectiveness any new system must be developed with details that can easily be standardized allowing multi-purpose usage of manufacturing equipments.

The use of prefabrication can reduce traffic and environmental disruption and improve work-zone safety, in addition to offering other advantages depending on site constraints. The successful use of prefabricated elements to accelerate construction requires a careful evaluation of the requirements for the bridge and an unbiased review of the total costs and benefits. This will lead the owner/engineer to the most effective course of action.

In early stages of a construction project, engineers need to assess whether elements of ABC are achievable and effective for a specific bridge location. Traffic management and user delay-related costs associated with bridge construction activities will significantly influence the selection of the most cost effective bridge technology. The reduced installation time for prefabricated bridges will also reduce the costs to highway users associated with traffic queues and detours during the bridge installation. Users incur costs during installation due to increased vehicle miles traveled (using detours) and increased vehicle hours of delay. For some projects, the cost of time ranges from $50,000 to over $300,000 a day. Getting a project delivered even a day sooner by using prefabricated bridge elements can mean significant savings. Many states are looking at improving the aging infrastructure along a corridor approach and here is an opportunity for cost saving when projects or bridges are bundled together. Utah has proven that by bundling projects and allowing several bridges to be prefabricated, the production cost (square foot cost) for the structures is about the same and/or less than their conventional cost.

In past years, the potential weak link in prefabricated systems has traditionally been the connections between components, e.g., the closure joint between deck panels, column to cap, or footing/drilled shaft to column. Whereas the prefabricated components are constructed in controlled environments, the closure joint construction is exposed to the variability inherent in field construction. Transverse and longitudinal deck closure joints are the biggest challenge to achieving long-term durability with minimum maintenance and rideability/smoothness requirements. Recent advances in the use of high-performance materials have introduced a new generation of connections that would be more durable and also low in maintenance. The FHWA has recently published a manual "*Connection Details for Prefabricated Bridge Elements and Systems*" (FHWA-IF-09-010) consisting more than 150 proven details compiled from several states' standards. These connection details provide proven examples of durable connections for ABC projects.

In many cases, foundation and substructure construction is the most costly and time-consuming part of constructing a bridge. To get maximum advantage from the on-site construction speed possible with prefabricated bridge installations, consideration should be given to using prefabricated components for foundations and substructures. A total substructure system may consist of modular abutments and walls, and prefabricated bent cap supported by prefabricated columns. The Iowa ABC demonstration project described in this paper incorporates prefabricated substructure systems which limits on-site construction to the foundations.

3 INNOVATIONS AND ABC DESIGN STANDARDS

There continues to be many significant and real challenges to fully implementing ABC methods nationwide. The R04 study of current ABC practices has shown that these

Figure 1. Tier 1 ABC Project—Jamaica Avenue over Van Wyck Expressway, New York. Weekend replacement using a lateral slide.

challenges can be met and successfully addressed if owners, designers and contractors innovate incrementally and collaboratively (HNTB 2010). Building on previous experience and constantly pushing the envelope will result in continued successes. Owners with sufficient ABC experience are beginning to see contractor acceptance along with schedule and cost savings. Initial added costs diminish with consistent and repeated use of the ABC approach.

An objective of the R04 project is to develop standardized approaches to designing and constructing complete bridge systems. The aim therefore is to develop pre-engineered standards for ABC construction for selected bridge substructures and superstructure systems that can be installed with minimal traffic disruptions in renewal applications. Bridge designs for "workhorse" bridges can be standardized to allow for repetition and pre-fabrication. By standardizing designs the opportunities for local or regional fabricators will be greatly increased, thus encouraging broader acceptance and reducing costs. Development of new construction concepts compatible with rapid renewal will generate greater contractor acceptance (HNTB 2010).

The R04 project takes the approach that for ABC to be successful, ABC designs should allow maximum opportunities for the general contractor to do his own precasting at a staging area adjacent to the project site or in his yard using his own crews. There is the reluctance to subcontract out much of the work to precasters and other specialty subcontractors. Unless precast components are prestressed, there's no need for a precaster. Substructure components are made of conventional reinforced concrete that can be precast by the General contractor without applying for any plant certification. The design of components should be designed to allow the contractor to self perform the precasting by paying special consideration to:

- Components that are simple enough to fabricate
- Components that allow some tolerance for erection
- Maximum repetition of components to reduce formwork cost
- Components that can be erected with conventional equipment like cranes

A preliminary attempt has been made to classify the proposed ABC design concepts into three tiers based on maintenance of traffic constraints. Ideally the aim would be to accomplish renewal of existing infrastructure with absolutely no interruptions to traffic or existing services. However this cannot be realistically achieved in many instances. Hence the Owner and/or Contractor should identify feasible systems that reflect a balance of cost and schedule to meet the specific objectives of every project. To this end, the concepts have been classified into three tiers based on implementation duration as follows:

- Tier 1—ABC Concepts that can be completed over a weekend closures (Figure 1)
- Tier 2—ABC Concepts that can be completed in a few weeks
- Tier 3—ABC Concepts that accelerate larger project, and save weeks and/or months on the overall schedule

4 MODULAR SYSTEMS FOR ABC (TIER 2 CONCEPTS)

The use of precast components and modular systems has the potential to minimize traffic disruptions and improve work zone safety. Precast abutment and pier details have been used sporadically by various Departments of Transportation around the United States with varying results. The main intent is to precast as much of the substructure as possible allowing for faster construction of the bridge and a reduced time of interference with normal system operation. Precast pier components also provide an opportunity to complete tasks in parallel. For example, the foundations can be cast on-site while the precast pier components are fabricated off-site.

Modular deck segments for concrete and steel bridge superstructures up to 140 ft spans that can be transported and erected in one piece provide the ideal building blocks for accelerated bridge construction. By standardizing the designs that will cover the span range from 40 ft to 140 ft, their availability through local or regional fabricators will be greatly increased, reduce lead times, thus increasing more widespread use and reducing costs. ABC construction technologies for the demolition of existing spans and the construction of new spans that can be used by local contractors will be developed in this project to be applied with the standardized modular bridge systems and substructure components. The intent is to develop standard construction concepts for erecting highway structures using adaptations of proven technology that serve multiple functions during a bridge construction process and can be easily adapted from project to project, easily transportable and cost effective.

Using the ranking of proven and innovative ABC concepts as a criterion, a short-list of design concepts recommended for standardizing was prepared (HNTB 2010). These concepts are those most suited for field trials and are considered market ready for ABC use. These design standards will aim to overcome known obstacles to widespread adoption of ABC as a preferred method of bridge replacement. The short-listed concepts suitable for standardization are as follows:

1. Modular superstructure systems
2. Segmental superstructure systems
3. Full-depth precast decks
4. Precast modular abutment systems
5. Precast complete pier systems
6. Erection technologies for modular systems

5 IOWA DEMONSTRATION PROJECT

One of the requirements of the SHRP 2 R04 project is the field demonstration of a bridge using promising ABC concepts. Under the R04 project, a total of $250,000 has been allocated to assist an owner in constructing a bridge project which demonstrates advances in accelerated bridge construction methods. A demonstration project that brings together under one project some of the most promising and innovative modular systems and connection technologies has been designed by the R04 Team and let for construction in 2011. This is a Tier 2 ABC project where the bridge is constructed in place using prefabricated modular components. The demonstration bridge project is located in Pottawattamie County, approximately 6 miles east of Council Bluffs, Iowa (HNTB 2010). The bridge carries US Highway 6 over Keg Creek and replaces an existing bridge that was constructed in 1953 (Figure 2).

The existing 180' × 28' continuous concrete girder bridge (FHWA # 043230, with spans of 81 ft- 48 ft- 81 ft.) was constructed in 1953 and is currently classified as structurally deficient with sufficiency rating of 33. A replacement bridge for this site was being designed by the Iowa DOT Office of Bridges and Structures using conventional design for construction letting in February 2011. The replacement structure will be a three-span (67'–3", 70'–0", 67'–3") 210'–2" × 47'–2" Steel/Precast Modular Bridge with precast substructures and precast bridge approaches. The proposed bridge replacement is intended to increase the structural capacity of the bridge, improve roadway conditions, and enhance safety by providing a wider roadway.

The bridge was originally designed in-house to be constructed with a planned detour with an estimated construction duration of 6 months. HNTB has redesigned this bridge using ABC techniques so that the replacement can be completed in a two-week period. The ABC period of two weeks only pertains to the time that traffic will be disrupted. The total duration for the project, including time for prefabrication, will be considerably longer.

In the ABC design the contractor will be required to:

- Fabricate/cast the modular superstructure units (Figure 3), precast substructure components, and precast bridge approach panels at a casting yard off-site or near the bridge site prior to road closure (outside of the ABC period).
- Construct the drilled shafts (located outside the footprint of the existing bridge) for the new piers prior to road closure (outside of the ABC period).
- An offsite detour will be used during the two-week period road closure to allow field erection and bridge completion.

This application provides a unique opportunity to effectively promote ABC for rapid renewal of the bridge infrastructure and also demonstrate various ABC technologies being advanced in the R04 project. The steel modular option was chosen as the most cost-effective based on early discussions with local contractors and fabricators. Although it will not be fully

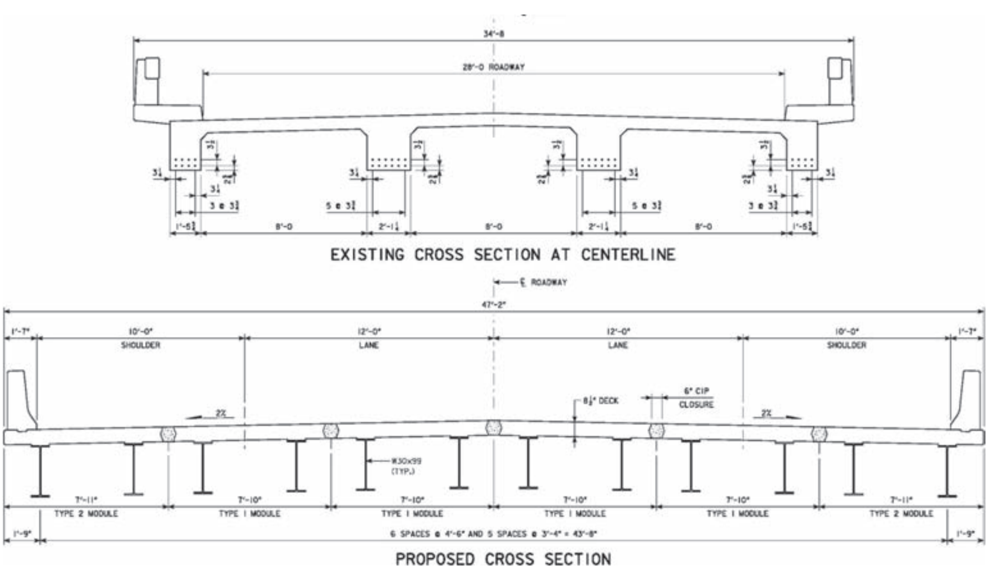

Figure 2. Bridge cross-sections. US Highway 6 over Keg Creek, Council Bluffs, Iowa.

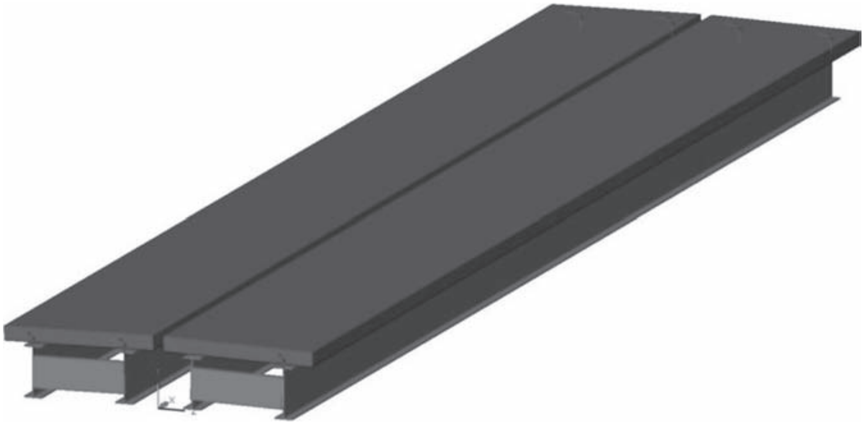

Figure 3. Decked steel stringer superstructure modules.

detailed on the design plans, the contractor will be allowed to propose a precast concrete modular alternative under a value engineering option if it can be constructed within the same ABC schedule and at a lower cost.

6 INNOVATIVE FEATURES

Phase I of the SHRP2 R04 Project identified several promising ABC design concepts and construction technologies that were considered candidates for further development and field trials. In Phase II, these concepts were incrementally winnowed down to a short-list that can be advanced to design standards and the implementation phase. Any technology recommended for field trials must meet minimum standards of apparent readiness for execution, a promise of durability and value to the owner. An evaluation matrix for each ABC concept was prepared and evaluated based upon criteria such as, initial cost, durability, system simplicity, constructability, market readiness for rapid construction, and other factors. Based upon the SHRP2 team's evaluation of this information, the proposed demonstration bridge was conceived and developed with the approval of the SHRP2 R04 Project Panel and in cooperation with the Iowa DOT.

The technologies incorporated into the proposed bridge project have been successfully used in constructed projects drawn from around the U.S. The fact that several diverse structural systems have been assembled and incorporated into a single project reinforces the concept that innovation does not necessarily mean creating something completely new, but rather facilitating incremental improvements in a number of specific bridge details to fully leverage previously successful work.

This demonstration project implements a series of innovations described in the following paragraphs. It incorporates details drawn from these diverse locations and applies them in a single demonstration project that will be visited by DOT and FHWA staff from numerous states. A day-long workshop, including a site visit, will provide an ideal opportunity to promote the dissemination of information to bridge owners around the country. Proposed project innovations include:

- Prefabricated bridge components in the form of:
  - Modular precast concrete deck / steel beam superstructure units with precast barriers
  - Superstructure units which incorporate precast suspended backwall elements to create a semi-integral abutment
  - Precast concrete pier caps and columns

  - Precast abutments and wingwalls with precast barriers
  - Precast concrete bridge approach slabs
- Overall, a complete bridge system will be designed and constructed using superstructure and substructure systems comprised of prefabricated elements. Decked steel stringer system is a proven concept shown to be quite economical and rapidly constructible. Their light weight, easy constructability, low cost and availability were seen as advantages over other systems. The bridge approach slab will also be a precast element.
- As the new bridge has a wider footprint than the existing, it made it convenient to locate the new pier foundations outside the limits of the existing bridge. The construction of the 6 ft diameter drilled shafts for the pier foundations will be completed using conventional construction prior to the closure of the bridge. This work will be done outside the 14 day ABC period as it does not impact traffic.
- The bridge will utilize complete precast piers with precast reinforced concrete square columns and reinforced concrete pier caps with internal voids to minimize the weight for ease of transportation and erection (Figure 4). Based on discussions with local contractors, HNTB decided to avoid prestressing of pier components to allow local contractors to self perform the precasting without having to outsource it to a certified precaster.
- High Performance Concrete (HPC) will be used in all precast components to enhance durability. Each modular system is expected to see a 75–100 year service life due to the quality of its prefabricated superstructure, the use of high performance concrete, and the attention given to connection details.
- Connections of the modular units are important elements for accelerated bridge construction, as they determine how easily the elements can be assembled and connected together to form the bridge system. Connections between the modular segments can also affect the live load distribution characteristics, seismic performance of the bridge, and also the superstructure redundancy. The bridge will have a jointless superstructure. Full moment connections between all precast components will be utilized to emulate cast-in-place construction. They also provide superior durability and minimize long-term maintenance.

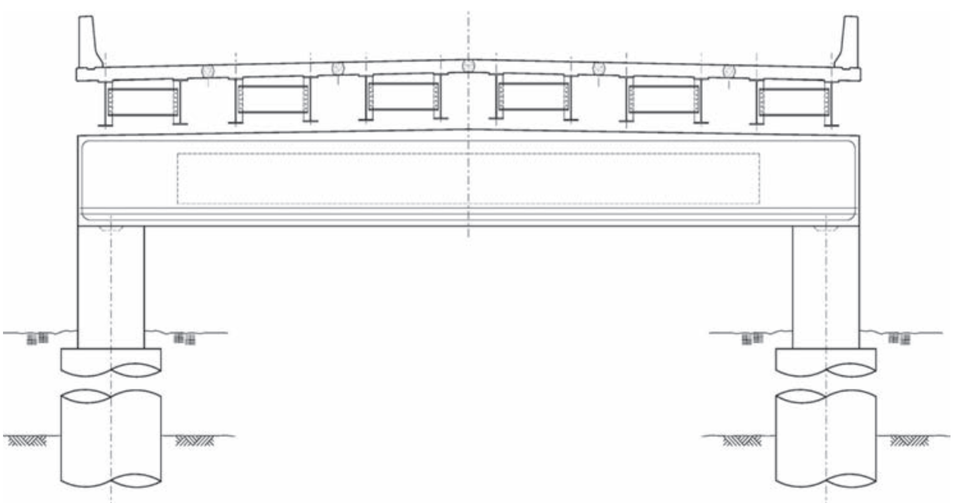

Figure 4. Precast pier elevation.

- Precast pier elements will be connected in the field using grouted splice sleeve couplers. The sleeves are then grouted, in effect making the reinforcing bars continuous through the connection. This allows for the structural connection of precast sections that will develop connections beyond the tensile strength of the bar. They are efficient, easy to construct and ideally suited for rapid construction application.
- Ultra High Performance Concrete (UHPC) will be used for the superstructure joints to emulate cast-in-place construction. UHPC represents a class of advanced cementitious composite materials whose mechanical and durability properties far exceed those of conventional concretes. UHPC compressive strengths range from 24,000 to 30,000 psi. UHPC present an opportunity to significantly enhance the performance of field-cast connections, thus facilitating the wider use of modular bridge systems. Of particular interest here, UHPCs can significantly shorten the development length of embedded discrete steel reinforcement and can exhibit exceptional bond when cast against previously cast concrete. These properties allow for a redesign of the modular component connection to facilitate accelerated construction and enhanced long-term system performance. By late 2010, field-cast UHPC connections between prefabricated bridge components had been implemented in nine bridges in Canada and two in the United States.
- UHPC will be used in the joints between the modular superstructure units and between the approach slab panels (Figure 5). UHPC will be used for longitudinal joints and transverse joints over the piers. Under past projects, a longitudinal superstructure joint consisting of UHPC material has been developed and tested by NYSDOT and FHWA to provide a durable, moment-resisting joint between deck panels.
- The proposed Iowa project will be the first in the US to use UHPC to provide a full, moment-resisting transverse joint in the superstructure at the piers. This detail will allow the superstructure elements to be erected as a simple span and, once the UHPC joints are constructed, perform as continuous joints. As an increased measure of design conservatism, this continuity will not be included in the calculated superstructure capacity. However, the elimination of open deck joints will provide for a more durable, low-maintenance structure in the final condition. A laboratory testing task to be undertaken by Iowa State University using a full-scale joint will focus on characterizing the structural performance of this unique and critical joint detail in the demonstration bridge.
- Self Consolidating Concrete (SCC) will be used to improve consolidation and increase the speed of construction for abutment and wingwall piles (fill pockets) and abutment to wingwall connections. Self-consolidating concrete is a highly flowable, non-segregating concrete that spreads into place, fills formwork, and encapsulates even the most congested reinforcement, all without any mechanical vibration. Abutments and wingwalls will consist of prismatic, precast concrete elements which feature a series of open holes which will accommodate driven steel h-piles (Figure 6).

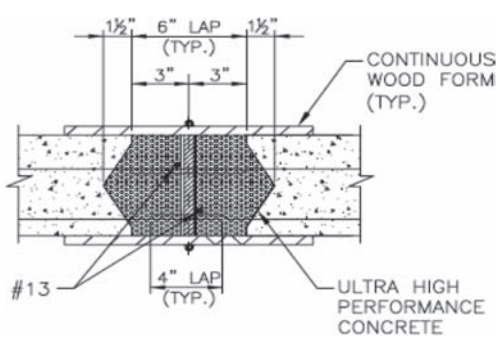

Figure 5. UHPC longitudinal joint between superstructure modules.

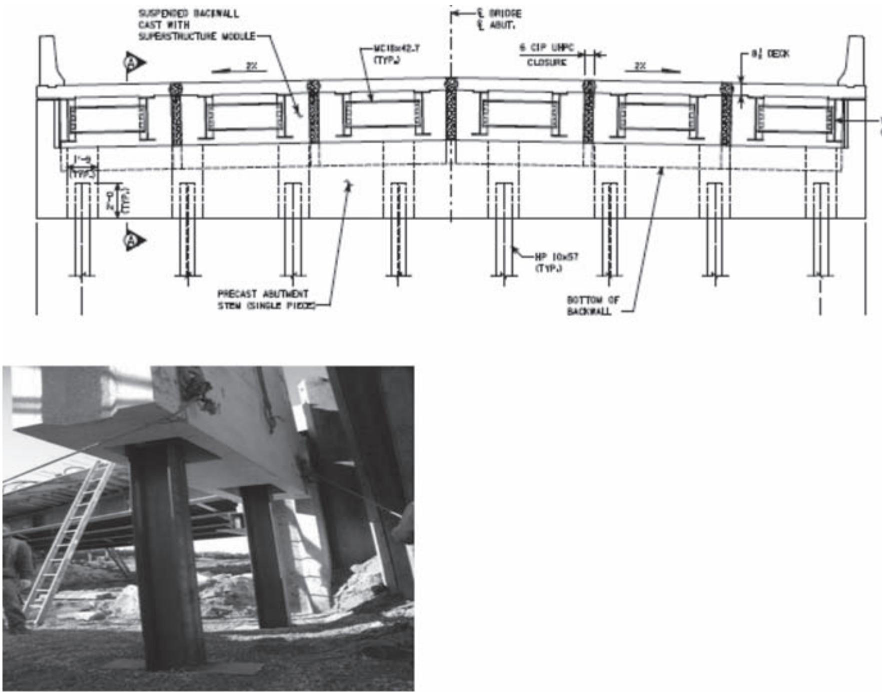

Figure 6. Precast abutment on steel H-piles.

- Use of fully contained flooded backfill: This proven construction method involves placement of a granular wedge behind the abutment backwall that is flooded to achieve early consolidation and significantly reduce the potential for formation of voids beneath the approach pavement.
- To support some aspects of the demonstration bridge innovative concepts, laboratory tests will be conducted including the following:
  - Placement handling and quality of the UHPC material used for transverse and longitudinal joint closures on the precast deck units and at the transverse joint connection between the precast approach slab and the precast abutment. This is especially important in a field-cast, ABC environment
  - Grinding of the UHPC closure joint material for the joints noted above
  - Serviceability and strength of the transverse bridge deck joint at the pier

7 PROPOSED CONSTRUCTION SEQUENCE

Precast modular systems are comprised of separate components fabricated off site, shipped, and then assembled in the field into a complete bridge. Contract documents require the bridge removal and replacement be completed within the two week ABC period, which is defined as the period where there are traffic disruptions on account of construction activities. Contract incentive/disincentive clause will apply for performance during the ABC period. The sequence of construction has been grouped into the following three stages:

Stage I: Work to be completed up to existing bridge demolition (Pre ABC Period)

- Construct drilled shafts to ground level
- Close bridge, enact detour
- Demolish existing bridge

Stage II: Substructure work to be completed after existing bridge demolition (ABC Period)

- Assemble precast pier columns, precast pier caps
- Drive abutment and wingwall H-piles
- Assemble precast abutments and wingwalls

Stage III: Assemble superstructure and approach slabs

- Assemble modular superstructure including precast barriers (ABC Period)
- Assemble precast approach slabs
- Construct UHPC longitudinal and transverse joints
- Grind deck and approach slabs to final grade

The winning bid of $2,660,000 came in about 25 percent higher than Iowa DOT's original estimate for construction. This is to be expected for the first time application of a new technology. There were seven bidders, indicative of the interest in rapid construction among local contractors in Iowa and their ability to self-perform the work. The winning bidder was Godberson-Smith Construction of Ida Grove, Iowa. Though the initial construction cost is higher than the engineer's estimate, the reduced installation time for the prefabricated bridge will reduce the costs to highway users. Reducing the bridge closure from 6 months to two weeks will also result in reduced traffic control costs, reduced impact to the local economy and improved safety. Bridge assembly is planned for the summer of 2011.

8 CONCLUSIONS

We must find smarter and faster ways of rebuilding the nations' transportation system using standardized approaches that allow rapid renewal, economies of scale in manufacturing and construction, reduced traffic disruption and increased safety. ABC has the potential to become a very powerful tool in the transportation industry. This demonstration project can impact the future practices of the industry and the U.S. Department of Transportation. New technologies that are implemented successfully on this project will accelerate the adoption of the innovations in the U.S. This will be accomplished by creating awareness and education related to the innovative features increasing confidence in recommending their use on other projects. ABC is beginning to gain momentum as owners begin to consider the added value of safety and quality of life and the other, less tangible costs, such as the political impacts of long-term road closures for construction, in determining the total cost of a project.

ACKNOWLEDGEMENTS

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Chapter 19

Accelerating bridge construction with prefabricated bridge elements—The latest in new technology for moderate to high seismic regions in the United States

B. Khaleghi
Washington State Department of Transportation, Olympia, WA, USA

K. Thompson
Arora and Associates, P.C., Auburn, CA, USA

P. Chung
California Department of Transportation, Sacramento, CA, USA

ABSTRACT: Prefabricated bridge components are in increasing demand for accelerated bridge construction. Prefabricating eliminates the need for forming, casting, and curing of concrete in the work zones, making bridge construction safer while improving quality and durability. Prefabricated bridges consisting of pre-tensioned girders, post-tensioned spliced girders, trapezoidal open box girders, precast columns, and other types of components often used for accelerated bridge construction; however, bridge engineers are concerned with the durability and performance of bridges constructed with prefabricated members in areas of high or moderate seismicity. This paper presents an overview of the latest ABC connection details being considered for use in moderate to high seismic regions. The benefits of having ductile details and improved inelastic performance may also extend into other multi-hazard areas, such as blast, flood, vehicular impact, etc. Emphasis will be placed on implementation and constructability of the "Next Generation" of seismic details.

1 INTRODUCTION

Over the last decade, the nation's economic and population growth coupled with decaying infrastructure have led to the need to rapidly replace, widen, and build new highway bridges. Transportation agencies are under increasing pressure to reduce congestion, improve mobility and reduce traveler delay, and improve delivery of highway and bridge systems using accelerated bridge construction (ABC) techniques. Agencies are focused on developing a construction approach that can reduce on-site construction time, while mitigating long term impacts to the public and environment.

In general, the fundament philosophy behind current seismic design is to manage and control the location where damage occurs. For example, superstructure and substructure (footings and foundations) components are often designed to remain elastic, and force plastic hinging to occur in well confined, ductile column or pier sections. This concept is often referred to as 'capacity protection" and allows for larger displacements without collapse under extreme load cases. This philosophy is consistent with the development of new details for use in accelerating bridge construction. The intent behind new details developed for the "Next Generation Bridges" (NGB) is to comply with current seismic performance standards, and allow for accelerating the time to construct in the field. The NGB system mainly utilizes prefabricated structural components because the components can be assembled quickly on-site with reduced impact to the site environment and traveling public. The challenge is in how these structural components are connected, and how they perform under seismic, and other multi-hazard loads. Prefabricated structural components can also allow for easy repair and replacement. However, using prefabricated components requires a solid understanding of the systems' seismic performance from moderate to large earthquakes.

2 PREFABRICATED GIRDER CONNECTION

Typical existing prefabricated girders are supported on bearing pads or rocker bearings on top of bent columns and piers. These connections provide minimum restraints to the girders from movements. In areas of moderate to high seismicity, the girders are susceptible to excessive displacement from the supports, and potential collapse due to large seismic excitations. Furthermore, the lack of monolithic action causes the column tops to behave as pin connections resulting in substantial seismic force demands on the foundations of multi-column bents, particularly in areas of moderate to high seismicity. Having a moment connection between the superstructure and column reduces the moment demands to the footing. The top of column moment connection will also help to reduce foundation costs. Therefore, having adequate moment resistant connections between the superstructure and substructure components is a key beneficial factor in developing seismic resistant prefabricated concrete bridge systems. Figure 1 shows an example of a typical monolithic moment resistant connection used at intermediate piers for WSDOT prefabricated girder bridges.

The cast-in-place concrete for the diaphragm section over columns of continuous bridges is completed in two stages to ensure the stability of prefabricated girders after erection, and

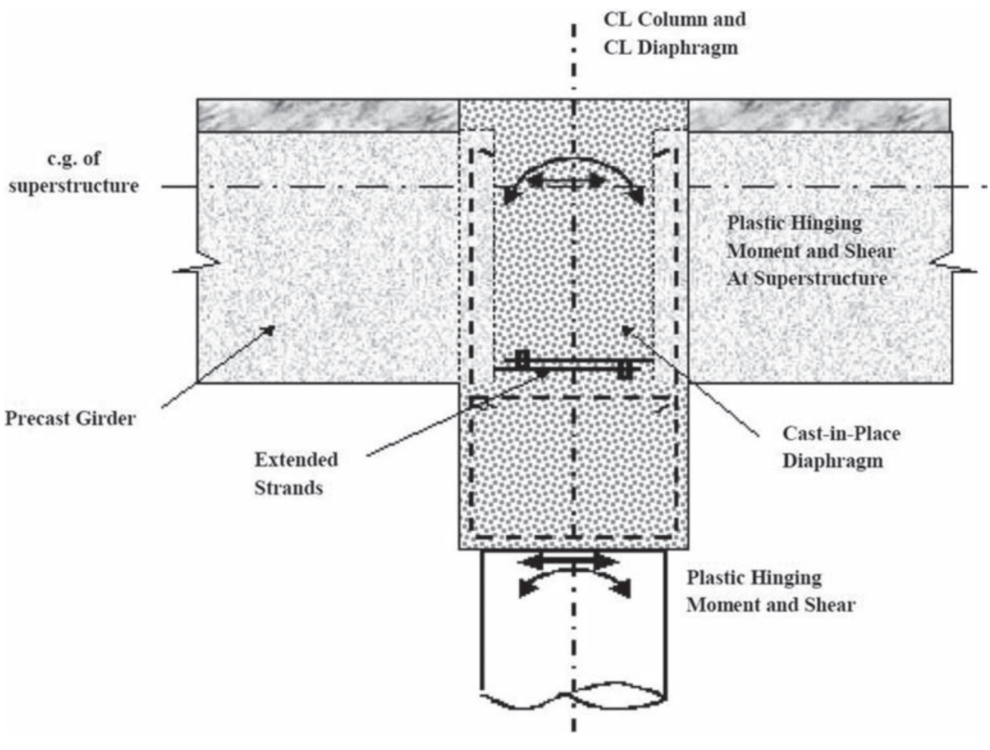

Figure 1. Typical monolithic moment resistant connection.

Figure 2. Precast girder connections—extended strands.

occurrence of initial creep. Extended strands and reinforcing bars are provided to ensure adequate performance of the connection during a seismic event. The design assumptions for fixed diaphragms are:

1. All girders of adjoining spans are the same depth, spacing, and preferably the same type.
2. Design girders as simple span for both dead and live loads.
3. Provide reinforcement for negative moments at intermediate piers in the deck due to live loads and superimposed dead loads from traffic barrier, pedestrian walkway, utilities, etc.
4. Determine resultant plastic hinging forces at centroid of superstructure. The intent here is to ensure that superstructure is capacity protected (remain elastic) while plastic hinges form on top and bottom of the column.
5. Determine the number of extended strands to resist seismic positive moment.
6. Design diaphragm reinforcement to resist the resultant seismic forces at centroid of diaphragm.
7. Design longitudinal reinforcement at girder ends for interface shear friction.

Prefabricated girder connection at end piers are often supported on elastomeric bearing pads at end piers. Semi-integral cantilever abutments (joint less) are used for shorter bridges, and L abutments (expansion joint between bridge and abutment wall) for longer bridges are typically used for prefabricated girder bridges. Bridge ends are free for longitudinal movement, but restrained for transverse seismic movement by girder stops. The bearings are designed to be accessible so that the superstructure can be jacked up to replace the bearings after a major seismic event.

The photos shown in Figure 2 are examples of precast girders with extended strands in the state of Washington. The girders are set in place as shown on the bottom left, with pre-stress strands extended into the bent cap area as shown bottom right.

3 PREFABRICATED BENT CAP

Prefabricated bent cap systems eliminate the need for forming, reinforcement, casting, and curing of concrete on the jobsite removing the bent cap construction from the critical path. Figure 3 shows a prefabricated bent cap under construction in Washington State. The #14 column vertical reinforcement will be placed through sleeves installed in the prefabricated bent cap. Sleeves are made of 4 in. (100 mm) diameter corrugated galvanized metal ducts allowing adequate construction tolerance and room for grouting. The completed bent cap will be performing as conventional cast-in-place concrete pier with column bars extended through the prefabricated bent cap to the top of diaphragm.

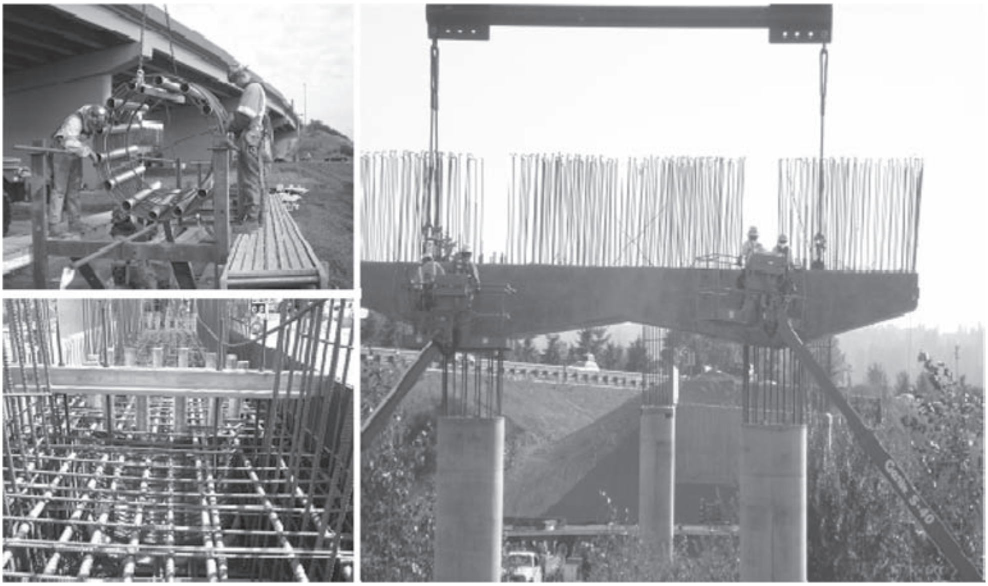

Figure 3. Typical monolithic moment resistant connection—bent cap.

4 FULLY PREFABRICATED BRIDGE BENTS FOR USE IN SEISMIC REGION

Bridge construction with prefabrication of modular components offers an attractive alternative to conventional bridges. The product innovation through Highways For Life (HFL) project consists of a totally prefabricated concrete bridge bent system that can be used in seismic regions. The HFL project consists of a two span continuous bridge on SR 12 and I-5 interchange in Washington State. The proposed system uses a small number of large bars grouted in ducts to achieve the connection between components so that it can be constructed rapidly and safely, and in contrast with systems developed previously, it has the structural resilience to resist earthquake shaking. To apply the system in a wide range of girder bridges, the product innovation will be accompanied by a design methodology, laboratory specimen testing, as well as guidelines for fabricators, contractors, and practicing bridge engineers.

WSDOT HFL project consists of a totally prefabricated bridge bent system, including prefabricated segmental columns, prefabricated bent cap, and prefabricated superstructure as shown in Figure 4. To accelerate construction without sacrificing seismic resistance, the beam-to-column connections are made with a small number of large-diameter reinforcing bars that are grouted into much larger-diameter ducts.

The HFL project will ensure that the product can be deployed in a wide range of applications. HFL project includes four phases:

1. Proof testing of project-specific and alternative-design variations of the system,
2. Development of project-specific and general design provisions and specifications,
3. Development of design examples, and
4. Deployment of the basic system in the field.

5 PREFABRICATED ABUTMENT

In an effort to mitigate traffic impacts, an innovative strategy was employed at a detour for a bridge replacement project on I-40 in California. The idea was to precast as much of

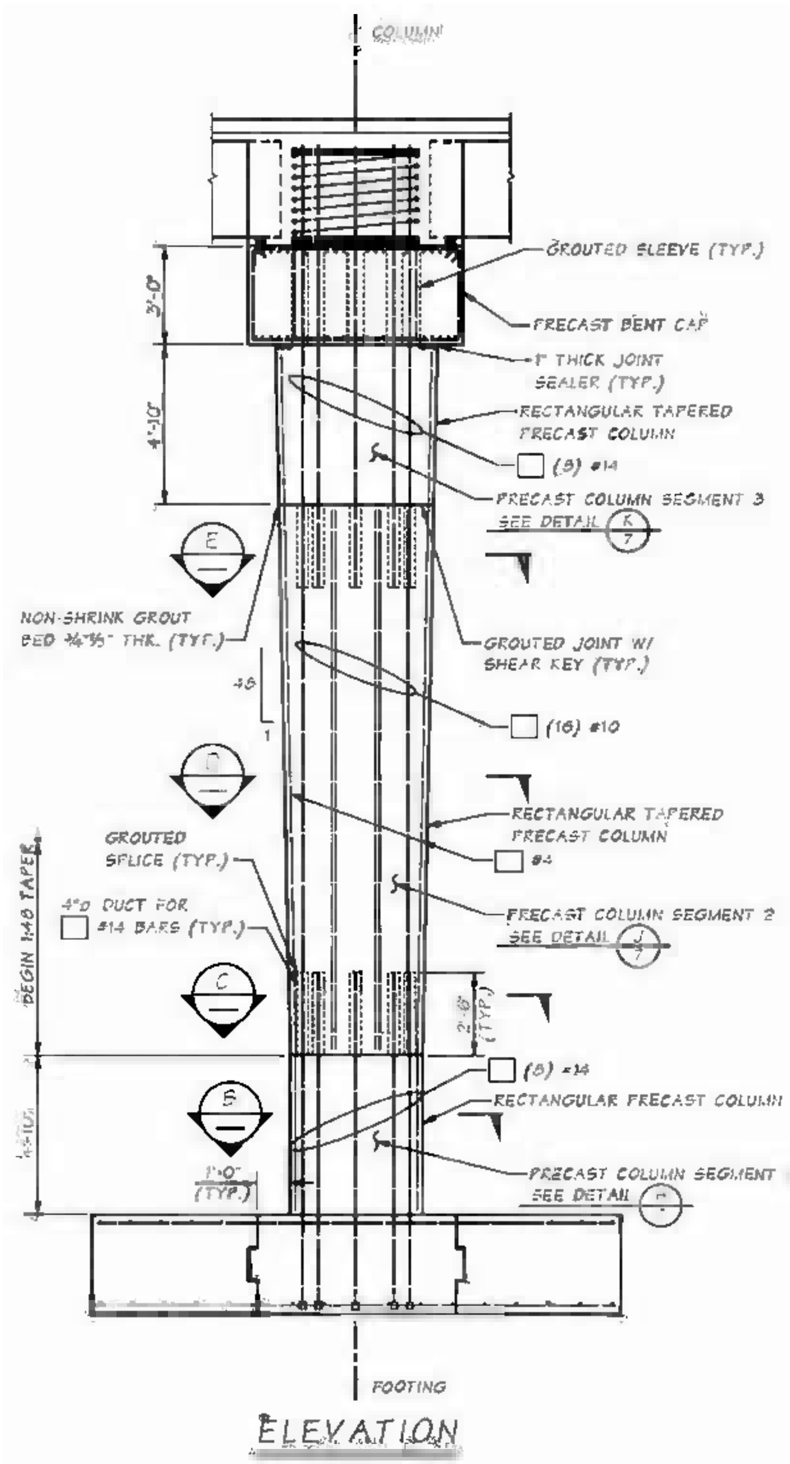

Figure 4. WSDOT fully prefabricated bridge bents for use in seismic region.

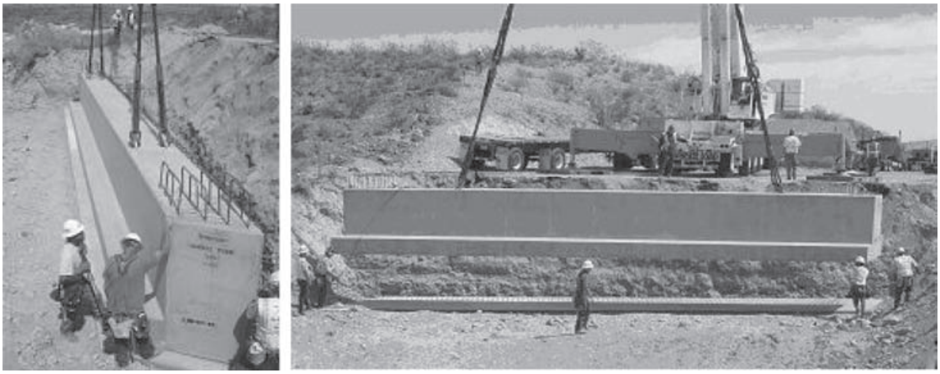

Figure 5. Photos of precast abutment placement.

the structure as necessary to expedite on-site construction. The existing two-span, nearly 106-feet long bridge was replaced with a single-span structure designed to reduce substructure construction efforts. Furthermore, site geology permitted the use of spread footings, thereby facilitating a precast abutment solution. The advantage of this strategy was that the abutments could be placed and the girders set nearly immediately thereafter.

The precast abutments were cast as whole-width pieces fabricated in a casting yard in Perris, California. Each piece, weighing 82 tons, (see Figure 5) was hauled by a special permit truck to the bridge job site. Cast-in-place concrete for a footing shear key, the abutment backwall, the abutment shear keys, and the approach fill wing-walls were considered to have minimal affect on the overall construction schedule. These latter operations proceeded simultaneously with the cast-in-place concrete deck construction.

Site preparation to accept precast abutments was accomplished by placement of a low strength 200 psi slurry pad designed to crush at interface high points when the abutment is placed.

The total bridge replacement time only took twenty-eight days. A 500-ton crane was used to lift and place the precast abutments; a 360-ton crane was all that was required considering the load and crane position relative to the pick and placement of the abutment section, but one was not readily available. The project was a huge success and set the stage for more innovative uses of prefabricated components to reduce construction traffic delays.

6 LESSONS LEARNED

Lessons Learned—Precast structures can successfully accelerate bridge construction. The main challenge is to keep the precast pieces within a practical weight range for transporting, picking, and placing. A segmented abutment design would reduce or eliminate the need for securing transport permits, lessen the premiums paid for trucking fees, and allow the use of cranes already expected on-site for other operations, such as setting girders. Another key advantage to segmenting precast abutment design is that bridges can be built in stages, with traffic allowed on earlier stages as existing structures are demolished to facilitate structure completion.

7 NEXT GENERATION BRIDGES

In California, cast-in-place (CIP) construction has been the preferred practice for a vast majority of bridges for the last 30 years. Typical CIP operation needs lead off, casting and finishing time and also requires a complex falsework system that often restricts the moving traffic during construction. Since 2009, Caltrans and WSDOT engineers have developed "Next Generation Bridge" (NGB) details that will meet current seismic performance requirements and also work towards meeting ABC goals. The main objective for the NGB systems is to facilitate rapid on-site construction that reduces traffic impacts to the surrounding environment, and provide for a faster recovery from a major seismic event.

Prefabricated elements can reduce overall on-site construction time, and also eliminate falsework and other shoring requirements. Recognizing the need for mitigating impacts to the traveling public, bridge engineers are moving towards NGB systems with prefabricated elements in their research and development efforts.

With 37 states in the U.S. having moderate to high seismic regions, the need for better connection details between prefabricated bridge elements is evident. Precast structural components pose a challenge to seismic design because the connection between components needs to have enough capacity to accommodate seismic forces and displacements.

Caltrans in 2009 initiated efforts to develop the Next Generation Bridges (NGB) as the new bridge systems for the future. The NGB is developed to achieve improved constructability, long-term maintenance, seismic performance, and accelerated bridge construction. In 2007, a workshop was conducted with the industry, academia, and peers to develop precast connection details that can be constructible. The plan is to use a shake table for testing to determine the seismic performance of the proposed system.

NGB details have been developed for precast column connections to footings, pile shafts, and bent caps using special longitudinal steel couplers without any post-tensioning. Figure 6, below depicts typical details of precast column (pier) connections to footing, and pile shaft.

The above details are precast concrete columns to footing/pile connections. It is assumed that the longitudinal rebar in the columns are connected to the foundations thru rebar couplers. Caltrans is currently initiating a seismic research project to experimentally test the connections and the rebar couplers.

The steel couplers provide workers a fast assembly process and they serve an important engineering role by providing moment resistance continuity to the column-foundation joint regions. If the test shows favorable performance, the use of steel couplers can lead to innovative repair technologies. A recommended repair technique can be employed by engineers

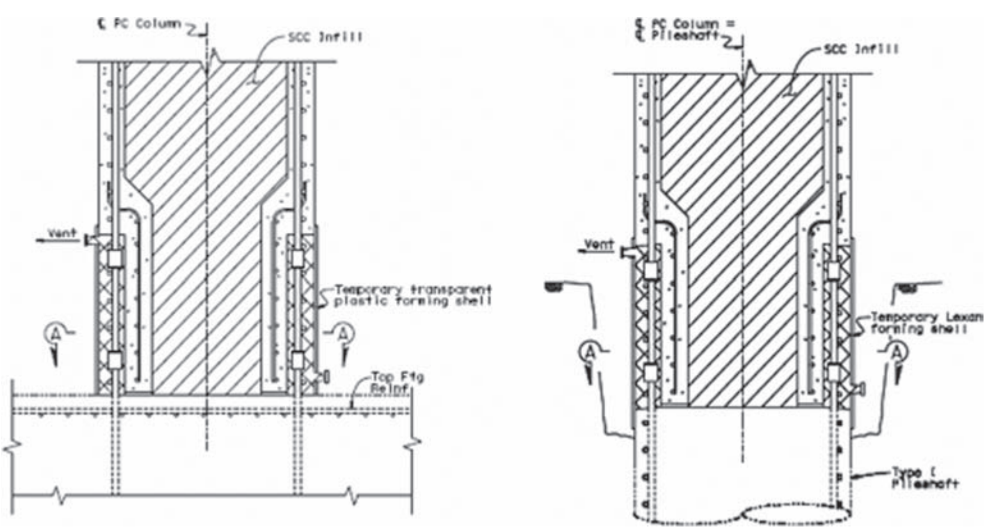

Figure 6. Proposed precast column to footing connection details.

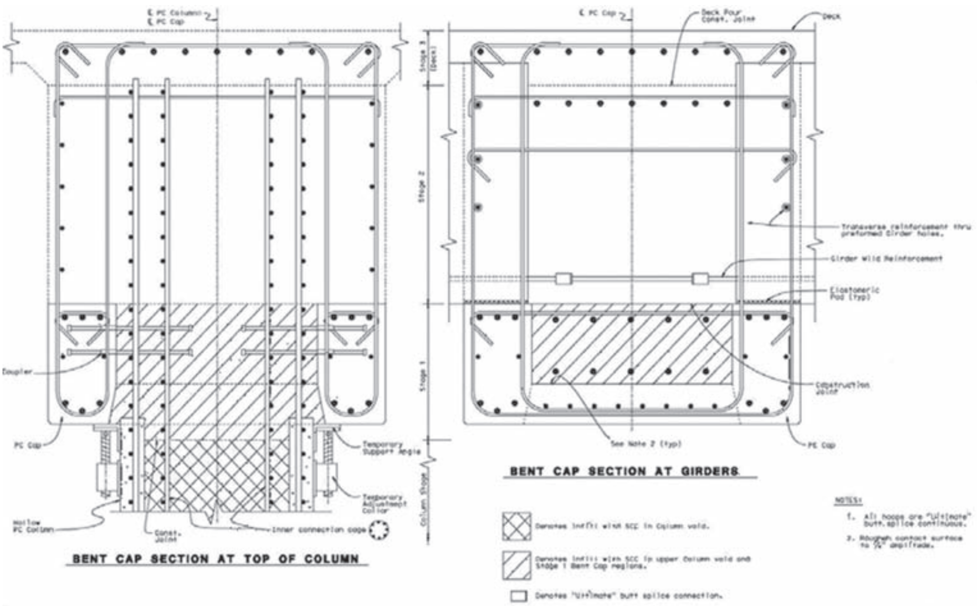

Figure 7. Proposed precast column-to-bent cap connection.

to remove the heavily damaged column segments and install new sections by connecting the reinforcement bars with steel couplers. To test the performance of the steel coupler connections, Caltrans sponsored a research study that will consist of analytical and experimental research. The experimental task will test each column connection component on a shake table using a simulated large magnitude earthquake motion. The test models will be approximately 0.4 scale. The performance of the fixed and pinned test columns will be evaluated based on data obtained in previous shake table tests of fixed and pinned columns. These data along with detailed analytical models will help shed light on the adequacy of the precast column footing connections. The NGB details for the seismic performance of precast column connections to bent caps using special longitudinal steel couplers without any post-tensioning have been developed. Figure 7, below depicts typical details of column (pier) connections to bent cap.

The above new structural details and the research will address an essential need of accelerated bridge construction (ABC) and the NGB that will minimize delay in the operation and repair of bridges. The outcome of the research could be used in partial or total bridge replacement on existing highways or could be used in construction of new bridges in future highways. These research projects are currently underway and the final results are scheduled to be released for detailed design development in late 2011.

8 CONCLUSIONS

This paper provides an overview of successful application of a few precast substructure and superstructure components, as well as new connection details. The design of next generation ABC precast connection details are promising, and consider construction tolerances and seismic design criteria to meet increasing demands for rapid replacement. Some of these details will need validation through testing for general use in moderate to high seismic regions.

ABC will continue to be a focus for transportation owners, for it serves to improve mobility and reduce traveler delay and is now part of FHWA's national "Every Day Counts" initiative.

Engineers must not only be cognizant of the impacts their planned works have on traffic, but must also proactively find solutions to minimize the time spent in the construction phase. Such solutions can improve mobility with minimum time and traffic delays as a basic parameter in improving mobility across the nation.

Accelerated bridge construction is a global issue, and has been a leading focus of national and international research, with numerous funded projects in academia and industry. Recently in the United States, programs have been developed to investigate viable solutions in terms of economics and constructible details to meet increasing elements of rapid construction and reduced traveler delay. The need for accelerating transportation construction is not limited to North America; European and Asian countries have embraced the idea and in many senses are leading the innovations. In the end, widespread implementation of ABC will greatly serve the public by benefiting commerce, the traveling motorists, and the economy of the region and society.

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Chapter 20

Replacement of the Hog Island Channel and Powell Creek Bridges

L. Fernandez
TranSystems, New York, NY, USA

ABSTRACT: TranSystems was the engineer of record for the replacement of the Hog Island Channel and Powell Creek Bridges for the MTA Long Island Rail Road. The bridges located along the Long Beach Branch served the Railroad for almost 90 years. The existing Hog Island Channel Bridge carried two electrified train tracks and consisted of timber pile bents and wide flange steel stringers. The existing Powell Creek Bridge also carried two electrified train tracks and consisted of timber piles and beams. The design of the new bridges took into consideration the difficulty of replacing the bridges while maintaining railroad service on the existing bridges. The design team used precast-prestressed concrete box beams with both galvanized and stainless steel reinforcement for both bridges. Precast box beams allowed for quick installation after the old superstructure was demolished. Construction started in September 2009 and completed by the end of May 2010.

1 INTRODUCTION

1.1 *Hog Island Channel Bridge*

Built in 1926, the Hog Island Channel Bridge was located along the Long Beach Branch of the MTA Long Island Rail Road (see Figure 1). It carried two electrified train tracks and consisted of timber pile bents and wide flange steel stringers. The bridge measured 29 meters from abutment to abutment and was comprised of 3 spans. The timber piles showed extensive deterioration and did not provide adequate lateral support for the superstructure. The wide flange steel stringers, 4 per track, were in fair condition (LIRR 2007).

Figure 1. Hog Island Channel Bridge.

Figure 2. Powell Creek Bridge.

1.2 *Powell Creek Bridge*

Built in 1926, the Powell Creek Bridge was also located along the Long Beach Branch of the MTA Long Island Rail Road, about a mile west of the Hog Island Channel Bridge (see Figure 2). It carried two electrified train tracks and consisted of timber piles and beams. The bridge measured about 20 meters from abutment to abutment and was comprised of 6 short spans. The key issue at the Powell Creek Bridge was flooding during significant rain events. Occasionally the third rail of each track was flooded which affected railroad service (LIRR 2007).

2 DESIGN

2.1 *Substructure*

The design of the new bridges took into consideration the difficulty of replacing the bridges while maintaining railroad service. This meant that most of the work could only be performed during the weekends with either single or double track outages. The substructure would have to be installed without compromising the structural integrity of the existing bridges and the superstructure would have to be designed to allow for quick installation during single or double track weekend outages.

With these controlling construction parameters, the design team used reinforced concrete filled open ended steel pipe piles for both bridges to meet scour depths based on a 100 year storm event (AREMA 2007). The diameter of the piles was set at 50 cm maximum to allow for installation in between the existing steel and timber beams. Altogether there were 96 piles installed at Hog Island Channel Bridge, with pile lengths up to 18 meters, and 48 piles installed at Powell Creek Bridge, with a maximum pile length of 16 meters.

At Powell Creek Bridge the existing 7 piers were replaced with one single pier halfway through the bridge. The new bridge measured about 17 meters from abutment to abutment with a single pier located at midpoint. The new bridge was shorter than the old one since the new abutments were installed in front of the old abutments (see Figure 3).

At Hog Island Channel Bridge the 3-span configuration was maintained. The first and third spans measured about 10 meters long and the second or center span measured 6 meters

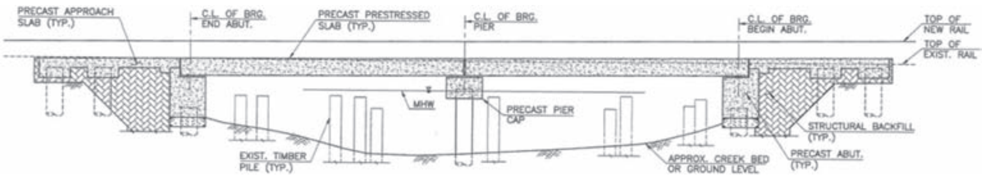

Figure 3. Powell Creek Bridge elevation.

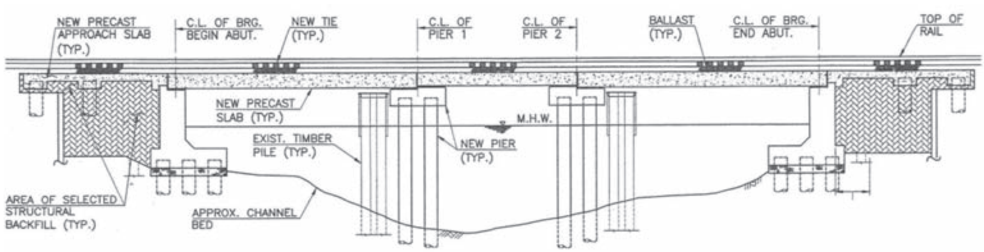

Figure 4. Powell Creek Bridge elevation.

long. The new bridge measured about 26 meters from abutment to abutment. Similar to Powell Creek Bridge, the total length of the new bridge was shorter than the old one since the new abutments were installed in front of the old abutments (see Figure 4).

Both bridges spanned environmentally sensitive wetlands. Therefore the design was coordinated with the New York State Department of Environmental Conservation, the US Coast Guard and the Town of Hempstead (LIRR 2010). To preserve the existing channel bed at Hog Island, the proposed rip rap for scour protection was limited to the surrounding areas of the abutments. It extended 3.7 meters in front of the abutment footings and 1.25 meters in depth. At the Powell Creek Bridge, since the water depth was considerably less, the pile depths were designed to withstand full scour events and therefore no rip rap was required. To reduce the footprint of the wing walls for both bridges, marine grade steel sheet piles were used.

2.2 *Superstructure*

For the superstructure, the design team used precast-prestressed concrete box beams reinforced with both galvanized and stainless steel rebars for both bridges. Precast box beams allowed for offsite fabrication and quick installation after the existing superstructure was demolished. There are a total of 6 box beams across each bridge, 3 per track. Altogether 18 box beams were installed at Hog Island Channel Bridge and 12 at Powell Creek Bridge. The box beams, 53 to 60 cm in depth and normally designed with voids, were filled with concrete to prevent water retention inside the beams during high tides or heavy rain especially at Powell Creek Bridge where flooding was an issue (see Figure 5).

The old bridge at Powell Creek was about 8.5 meters wide, and the new bridge measured about 11 meters wide. Similarly, the old bridge at Hog Island Channel was about 8 meters wide, and the new bridge measured about 11 meters wide.

The original track profile was retained at the Hog Island Channel Bridge. At the Powell Creek Bridge, since flooding was a significant issue, the track profile was raised about 30 cm at the bridge. The new track profile meets current AREMA and Railroad standards.

New fiberglass grated walkways were installed on each side of both bridges to provide access for Railroad forces during maintenance and inspection.

Figure 5. Precast box beams.

The new bridges were designed with independent spans for each track, with a longitudinal joint and symmetry along the centerline of the bridges, so that at least one track could remain in service while construction could be performed on the other track.

The new bridges were designed for Cooper E80 loading (AREMA 2007). The design phase involved periodic meetings with the Long Island Railroad to confirm that the proposed design met current Railroad requirements and preferences.

3 CONSTRUCTION

3.1 *Site constraints*

The bridge at Powell Creek was located on shallow waters. Using a barge for construction equipment was not feasible. The contractor built a temporary trestle adjacent to the bridge designed to carry crane loads (see Figure 6). The Hog Island Channel Bridge was over deep water so the contractor was able to use a barge for construction equipment and crane positioning.

3.2 *Substructure & superstructure*

The footprint of the new bridges overlapped with the old ones. The old bridge at Powell Creek was about 8.5 meters wide, and the new bridge measured about 11 meters wide. Similarly, the old bridge at Hog Island Channel was about 8 meters wide, and the new bridge measured about 11 meters wide. All bridges, both old and new shared the same bridge centerline. This meant that the new substructure would have to be built while the old bridge was in place. 50 cm diameter steel pipe piles were used since this was the maximum pile diameter that could fit in between the steel wide flange and timber beams of the existing superstructure at both bridges. The 50 cm diameter steel pipe piles were installed using vibratory hammers for up to 75% of the total height then the piles were driven down to capacity to its final tip elevation. The piles were cleaned out using an air lift pump to make room for the steel reinforcement and tremie concrete (see Figure 7).

Figure 6. Temporary trestle at Powell Creek.

Figure 7. Steel pipe piles, 50 cm diameter.

The existing Powell Creek Bridge clearance was very shallow, with the bottom of the timber beams only 1 meter from the mud line at the abutments. With this constraint, precast concrete pier caps and abutments were used for quick installation after demolition of the old spans, with galvanized reinforcement due to exposure to salt water (see Figure 8). The existing Hog Island Channel Bridge had a higher clearance with about 2.5 meters to the mud line at the abutments. Therefore, cast-in-place concrete pier caps and abutments were installed with galvanized reinforcement.

Figure 8. Precast concrete pier caps & abutments at Powell Creek Bridge.

Figure 9. Installation of precast box beams.

3.3 *Construction schedule*

The heavy construction, including pile driving, demolition of the old bridges and installation of the new bridges were performed on weekends so there would be no interruption of service for weekday commuters between Long Beach and New York City. This meant 48-hour work shifts on 12 weekends (LIRR 2010).

Construction of the substructure began on October 2009 and only performed on the weekends to avoid interruption of service for weekday commuters. The substructure for both bridges was installed in 10 weekends and completed by April of 2010 and involved a combination of single and double track outages. Since both bridges operated on the same railroad branch and only a mile apart, the bridges shared the same track outages.

The installation of the superstructure was performed in 2 weekends with double track outages. One half of each bridge, carrying one railroad track, was demolished and replaced within one weekend (see Figure 9). The demolition of the old superstructure started on a Friday night. Installation of the new precast concrete slab superstructure and approaches began by 4:00 AM Saturday. The new bridge was ready for railroad forces to fill with ballast and new tracks and ties by 9:00 AM Sunday. Railroad service was restored in time for the Monday morning rush hour.

Construction of the bridges started in September 2009 and was completed at the end of May 2010. The work was estimated at a cost of $24.5 million. The project is a partnership between the Metropolitan Transportation Authority and the Federal Transit Administration (LIRR 2010).

4 CONCLUSIONS

Other types of design considered during the preliminary phase of the project included wide flange steel beams with an open deck system similar to the old Hog Island Channel Bridge. Although this type of structure was easier to build, the long term performance of steel elements in a chloride rich environment was not favorable. A concrete superstructure was the preferred alternative (see Figures 10 and 11).

For bridge replacement projects with very limited scheduling constraints, prefabrication is key. In this project, the designer and contractor were limited to working 48 hour work shifts for 12 weekends to avoid interruption of commuter railroad service during the weekdays. The use of precast-prestressed concrete elements allowed for quick installation after

Figure 10. New Hog Island Channel Bridge.

Figure 11. New Powell Creek Bridge.

demolition of the old structures. Also, the quality of the concrete elements was improved with the fabrication process performed in the controlled environment of a concrete plant.

Proper planning and consistent coordination with Long Island Rail Road forces, the contractor, the engineer and the community were vital to the increased productivity during each weekend worked on site. The project was completed on schedule and within budget.

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6 Bridge management & monitoring

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Chapter 22

Remote structural monitoring systems for long-term confidence in a structure's condition

T. Spuler, G. Moor & R. Berger
Mageba SA, Switzerland

R. Watson
R.J. Watson, Inc., USA

ABSTRACT: Monitoring of bridges can serve wide range of purposes, providing continuous records of any variable in a structure's condition. This may be of special interest to owners of older bridges, who must ensure the ongoing safety of their structures—not only because of the unavoidable deterioration that comes with age, but also due to the increased traffic loading of recent decades, and the fact that bridges which were built in the past to different design and construction standards. An automated monitoring system can be utilized to provide real-time information on any structure's condition, allowing gradual changes over a period of time to be identified and offering notification by SMS or e-mail of any sudden changes in a chosen variable, or exceeding of predefined alarm values. Having such a system in place ensures that any changes in the structure's condition will be recognized immediately, allowing appropriate strengthening work to be planned.

1 INTRODUCTION

Questions may arise during the life of any structure which may cast doubt on its ability to continue to function safely and well—for instance in the aftermath of an unexpected event or where significant deterioration is noted in the course of planned maintenance inspections. Where such concerns arise, it may not be a straightforward matter to properly assess the situation and conclude that the structure is not in need of any remedial action. In such cases, modern technology can offer a means of obtaining the information required to make a sound engineering judgment, which would not have been possible even in the recent past. The use of sensibly designed automated structural health monitoring systems can thus potentially allow costly and disruptive strengthening or other remedial works to be postponed or even deemed unnecessary. Examples of such applications are presented below.

2 THE PURPOSES SUCH AN APPLICATION CAN SERVE

In certain cases the condition of a structure, at a particular point in time, may be known to be safe and satisfactory—for example, as a result of a short-term evaluation as described above. However concern may remain that the condition of the structure could change quickly for some reason, potentially making the structure unsafe or making accelerated deterioration likely. In such circumstances, an automated monitoring system, used purely to monitor the condition of the structure (without any further evaluation of the data recorded) can provide up-to-the-minute, precise information on the relevant variables (Wenzel et al., 2008). Where immediate notification of any change which might indicate a reduction in safety of the structure is required, manual observation is highly unlikely to be practical and provide the required level of certainty.

However automated monitoring systems can monitor for any such changes and provide the required notification, immediately and efficiently, at a fraction of the cost of manual monitoring. For such an application, a monitoring system can be designed to continually measure critical data (such as forces or movements of any part of the structure), and to immediately send an alarm signal, via SMS or e-mail, to the structure's engineers should any pre-defined alarm value be exceeded. This can allow a bridge owner to be confident that any sudden or significant change in the bridge's condition will be known immediately, allowing appropriate action to be taken to ensure the safety of the structure and its users.

3 WEYERMANNSHAUS-VIADUCT—SWITZERLAND

3.1 *Situation and problem statement*

The Weyermannshaus Viaduct shown in Figure 1 is 912 m long, and the height of its deck above the ground varies from 10 m to 15 m. The bridge is a pre-stressed concrete structure, with a maximum bridge width of 40 m. Various renovation activities could be easily planned for this structure, including the replacement of bearings, expansion joints and drainage system. However establishing the condition and necessity for remediation of the concrete structure was less straightforward. An initial detailed visual inspection of the bridge identified localized surface cracking of the concrete, as shown in Figure 2. The cracks were concentrated at the locations of the coupling joints of the pre-stressing cables. The appearance of these cracks could not be explained, and it could not even be established if they appeared immediately during the construction of the bridge due to the effects of pre-stressing, or later during its operational phase.

The authorities decided to further analyze the overall condition of the bridge before starting the general renovation, using only non-destructive methods. The analysis should identify

Figure 1. Weyermannshaus Viaduct, Berne—Switzerland.

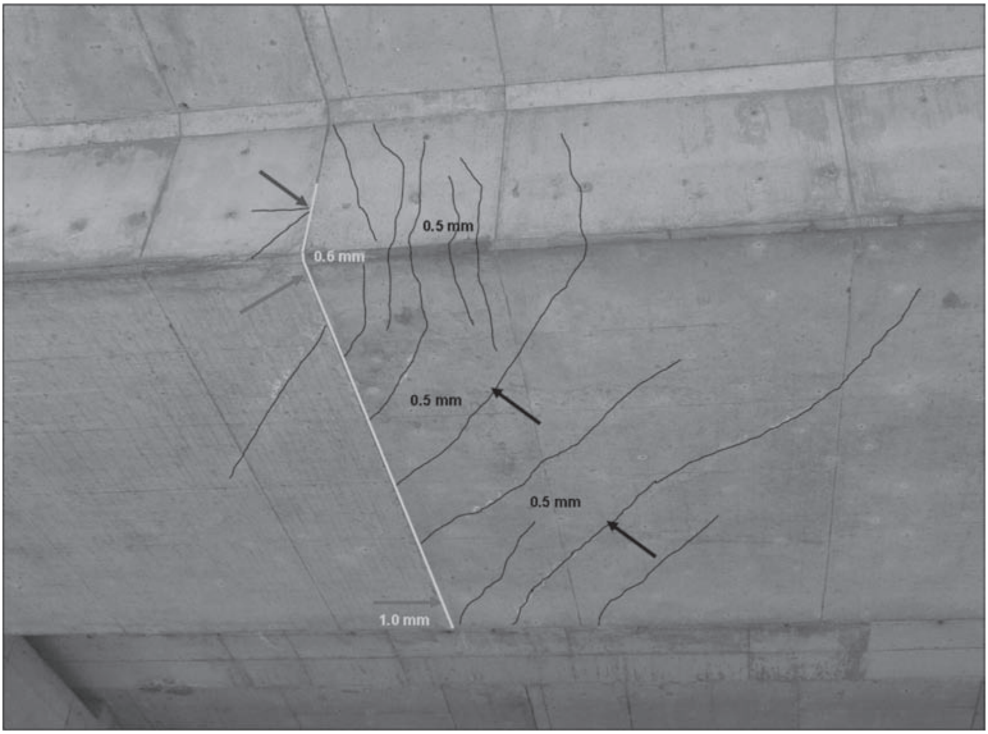

Figure 2. Cracking at location of coupling joint of pre-stressing cables.

the cause of the cracking and assess the condition of the structure, drawing conclusions about the remaining life expectancy of the bridge.

The bridge monitoring project was announced by the Swiss federal roads authority and the Swiss Federal University EPFL. It was mainly driven by the uncertainty about the overall condition of the bridge. Assessment by experts could theoretically conclude that the observed cracking (see Figure 2 on the next page) was superficial only and that the bridge is generally healthy, or that the structure should be considered to have a 'fully-cracked' cross section resulting from serious overloading.

3.2 *Goals of the project*

The authorities commissioned a local engineering company, already involved in the overall project and known as specialists for bridge condition assessments, to manage the monitoring project. They determined that a permanent monitoring system would have to be installed to confirm their assessment model on an ongoing basis. The main purpose of this monitoring project is to give answers to the following questions:

- Should the cross-section of the bridge be considered to be "fully cracked"?
- How is the strength of the structure affected by the established cracking level?
- Are the pre-tensioning tendons suffering from fatigue?

The static model developed included all important key figures for both theoretically 'non-cracked' and 'fully cracked' concrete conditions. The evaluated situations represent the boundaries of the assessment. The monitoring system installed should allow the bridge condition to be assessed much more accurately.

While previous assessments by bridge experts, based on the limited information available, did not conclude that the bridge was likely to be in a poor condition, the monitoring was expected to confirm this hypothesis in a relatively easy and economical way. Failing this, the safety of the bridge may have had to be ensured by strengthening works, potentially at much greater expense.

3.3 *Main characteristics of the chosen monitoring elements*

Based on the decision that only non-destructive system could be applied, it was decided to install a remote monitoring system (*ROBO®CONTROL* by Mageba SA, Switzerland), tailored for this application to measure primarily crack widths. The system measures crack movements at 16 locations, at the pre-stressing coupling joints. The layout of the sensors at one coupling joint is indicated by their references R1–R6 in Figure 3, while the locations of temperature sensors is shown by their references T1–T6.

The crack movements are measured with inductive LVDT sensors and LVDT current converter couples. The signal converter is placed close to the measurement position and transforms the physical parameter to a stable current signal according to the industrial 4 to 20 mA interface. The sensors have a measurement range from just +/−1 mm with a linearity better than 0,3% and repeatability of 0,15 μm. Due to the LVDT principle the resolution is practically infinite (depending on the selected measurement range and amplification). In the project's preparation stage, test routines showed reliable resolution of 0,5 μm.

Environmental conditions are monitored by four temperature sensors, which were drilled into the concrete, and two meteorological stations measuring temperature and humidity of the air. The measurement frequencies can be varied between 1 Hz and 500 Hz. A data pre-analysis can be programmed to filter the data output and to ensure an adequate data supply while limiting transmission costs.

The calculation model developed for the structure allows characteristics such as the tension in the reinforcement of the concrete and the condition of the pre-stressing tendons to be deduced from the measured crack widths. The analysis required a frequency of 500 Hz to ensure adequate results, as lower frequencies wouldn't permit measurement of the effects of vehicles moving at up to 120 km/h. The data was processed locally and saved on a hard disc.

3.4 *Calibration of the monitoring system*

To facilitate calibration of the system, the bypass motorway was temporarily closed to traffic while a truck with a known weight of 40 tons passed over the critical sections of the structure at different, predefined speed levels.

As expected, variation of the speed had no impact (Wenzel et al., 2005), meaning the crack width during the passing of a slow truck is comparable to the crack width during the passing of a truck traveling at 80 km/h.

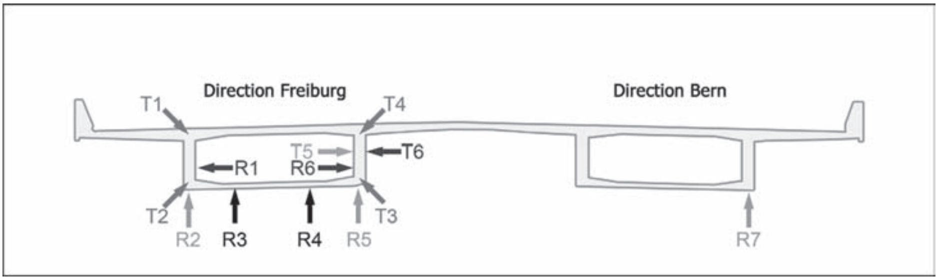

Figure 3. Locations of movement sensors (R1–R6) and temperature sensors (T1–T6) at one coupling joint.

Due to only minimal noise practically no digital signal conditioning is necessary. To make the manual determination of the maximum crack movement during the passage of test trucks more convenient, a gliding smoothing algorithm over 10 measurement values (@500 Hz) was applied.

The first comparison of the measured values with the calculated figures of the static model showed an excellent correlation between the measured values and the predicted results from the 'non-cracked' concrete cross section model. This correlation is shown in Figure 4, where graphs representing measured and predicted values are presented together. This analysis provided the first solid evidence that the bridge is generally in good condition.

In addition, the measurement clearly showed the different reaction of the bridge to different types of truck. A direct correlation between the crack width and the weight of the truck could be identified, allowing counting and weight-classification of vehicles to be used directly in the fatigue assessment of the bridge's reinforcement and pre-stressing cables.

3.5 *Short time analysis of traffic load impact*

After completion of calibration, the monitoring system was adapted to measure the effects of traffic on the structure. Due to the enormous amount of data generated at the chosen frequency of 500 Hz, and the hardware limitations of the installed computer, this assessment was conducted for one week only. All data was stored locally on the hard drive of the computer and was transmitted by satellite to the server in the office. After processing of data all values were made accessible in user-friendly format via the internet. The results of this analysis were the same as those resulting from the calibration measurements, meaning that normal traffic conditions result in the same stresses and crack widths as observed during the calibration.

Therefore the measured data could be used for the fatigue assessment of the structure (Rafiq et al., 2005)—the number of peaks in the crack width measurements can be used to determine the number of trucks passing over the bridge.

3.6 *Long term analysis of temperature impact*

Finally the system was adapted, using lower measuring frequencies, to assess the impact of the temperature over a period of one year. The duration was chosen to ensure that the

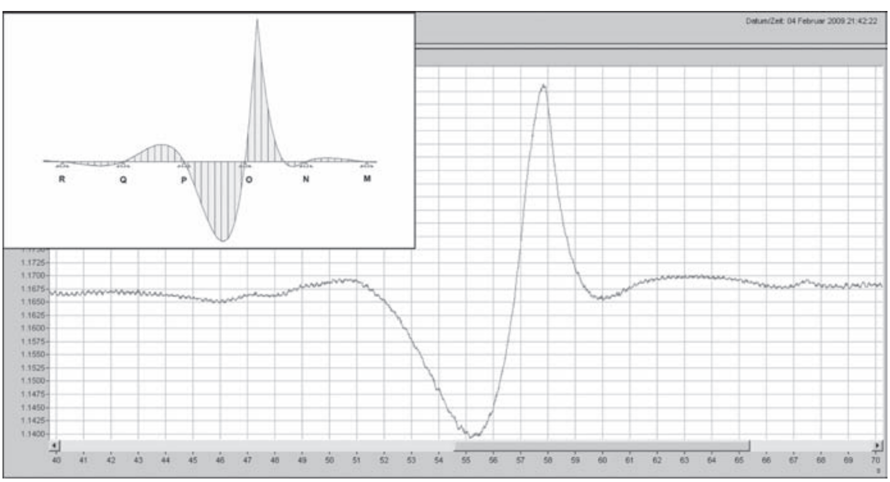

Figure 4. Crack width measurement during system calibration (main graph), demonstrating excellent correlation with calculated influence line of static model (inset graph).

whole range of temperatures during summer and winter times would be covered. The data processing was changed to suit the different needs of the adapted system.

Continuous measurements at highest frequency were no longer required due to the slow impact of temperature changes. The sensors still record at high frequency, but the on-site computer filters all values and only saves and transmits selected values at pre-defined intervals of five minutes, recording the average, minimum and maximum crack width values for each period. The system can be remotely controlled, to allow measurement frequency and data collection and transmission interval to be adapted as desired from the engineer's office.

The data is transmitted to the off-site server and can be accessed in real time via the internet, allowing the authorities and engineers to analyze the data from any computer in the world with internet connection. All data can be directly downloaded in Excel format for ease of analysis. As a result the bridge can be completely monitored from the office, reducing greatly the effort and expense associated with manual methods. The graphic presentation of detailed measurements by the Weyermannshaus Viaduct monitoring system is shown in Figure 5.

3.7 *Structural analysis based on the measured data*

The measurements such as those presented in Figure 5 indicate a clear correlation between temperature (of the structure and of the air) and crack widths (here shown with reference to the crack widths measured at location R5). Furthermore it could be established, by comparing crack movements during initial measurements with crack movements at 6 months later that no significant difference had developed, indicating that the cracking was not deteriorating due to increased cumulative loading. The measured values were used in the engineer's analysis of the impact of temperature on the bridge's condition. The hypothesis, which resulted from the computer model in advance, that the impact on bending moment from temperature is higher than that from traffic loading, was confirmed by the measurement results.

The static evaluation of the structure, using the results provided by the monitoring system could thus be used by the responsible bridge engineer to conclude:

1. The critical concrete section is not fully cracked, but showing a slightly reduced resistance.
2. The strength of the structure is still adequate for current and projected loading.
3. The fatigue assessment was positive, indicating that the pre-tensioning tendons should continue to serve their purpose for the remaining life of the bridge.

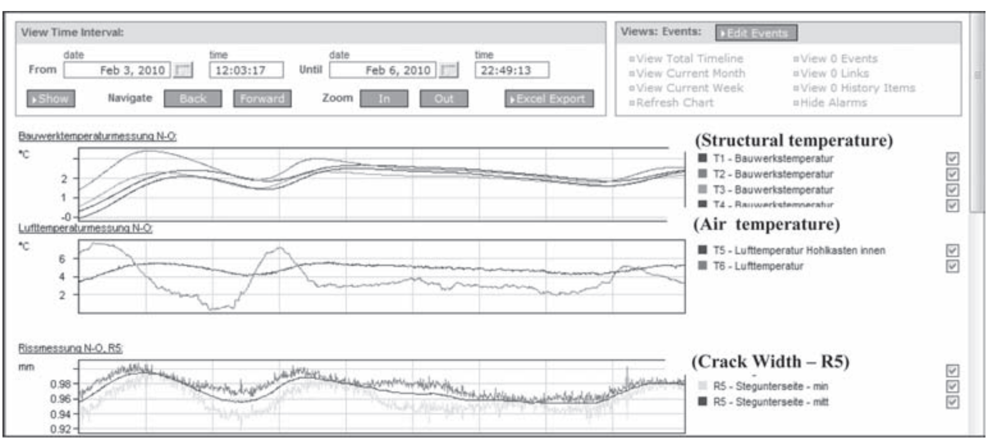

Figure 5. Presentation of detailed measurements in graphic form (3-day period).

4 THE RHINE FALLS—SWITZERLAND

4.1 *Situation and problem statement*

The Rhine Falls (Figure 6) in Schaffhausen, Switzerland is visited by hundreds of thousands of tourists every year. It is one of the region's most important tourist attractions and visitors marvel at the beautiful scenery from a terrace located close to the castle of Laufen.

Rock anchors installed to stabilise the rock wall below the castle showed unexpected force changes, leading to concerns that some sliding surfaces had developed. To ensure the ongoing safety of the terrace above, additional rock anchors with measuring devices were installed, with a Robo®Control system monitoring anchor force changes. This enables the responsible design engineer to draw conclusions about the wall's movement behaviour. The rock wall of particular interest (Figure 7) is below the castle of Laufen and about 20 m high, and was stabilised with 11 additional rock anchors. The installation conditions were challenging due to high exposure, noise and dampness.

4.2 *Design and installation of the system*

It was determined that a permanent Robo®Control "Basic" system should be connected to the newly installed rock anchors, to measure the loads arising in the anchors. The system should continually transmit all measured data to a web interface for analysis at any time, and additionally provide an alarm function to notify immediately of any sudden change in the forces acting on the anchors. As the system is independent of external measurement technology, flexible adaptations to suit commonly available load cells are possible (Casas et al., 2006).

It was decided during consultations with the bridge owner and engineer that records of forces at 30-minute intervals would in general provide an appropriate level of information. However the ability to obtain more frequent values is possible; the measuring frequency can be adapted according to the end user's requirements at any time.

4.3 *Description of the SHM system*

The system primarily comprises eleven load cells, which are directly connected to the installed rock anchors, and two meteorological devices to measure air temperature and humidity. A permanent power supply was locally available, but the client chose to have the system equipped with a buffer battery to overcome any potential power supply interruption of up to three hours. Data is transferred at defined intervals via the local GPRS network, with the system connecting itself to the internet and sending its data files per ftp to a central data server

Figure 6. The Rhine falls on the Swiss/German border.

Figure 7. Rock wall of main interest with public terrace on top, stabilized with 11 new rock anchors.

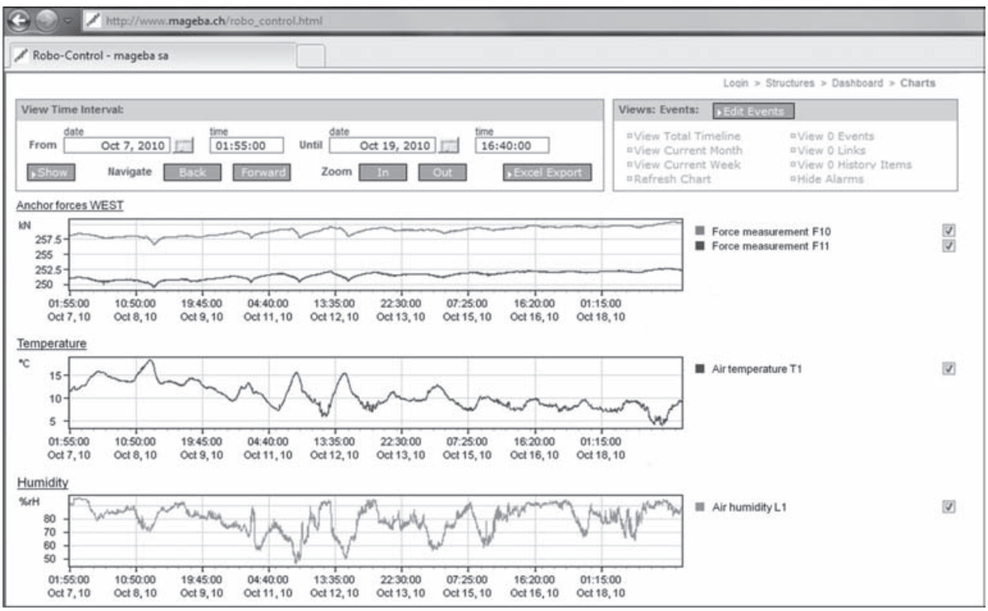

Figure 8. Graphical presentation of measured data relating to anchor forces F10 and F11.

before disconnecting again. Just one value is measured per hour for each variable, keeping the transmitted data volumes, and thus the transmission costs, very low. The system's central computer is housed in a high quality outdoor cabinet to withstand the harsh climatic conditions of the location, where very low temperatures in combination with high humidity can be expected to arise in winter. The system is also designed to facilitate possible expansion at a later date, with spare measurement channels.

A key feature of the installed monitoring system is the integrated alarm system which will automatically send alarm notification should any pre-defined force value limitation be exceeded. The alarm values were chosen in accordance with the designer's requirement that the force arising in any anchor should not vary from its initial design force by more than 15%. This allows the site owner to have confidence that any changes in the condition of the rock wall will be immediately recognized and alerted, enabling appropriate and immediate action to be taken to ensure the safety of the public.

4.4 *Monitoring of rock stability at Rhine Falls—observations to date*

It can be concluded that the rock wall has been well stabilised by the additional rock anchors, with only negligible movements observed. The measured data (see Figure 8, showing forces recorded at anchors K10 and K11) demonstrates this, with forces shown to vary by just +/–0.5%, and perfectly in line with the temperature changes recorded by sensor T1.

It can be concluded that the rock wall continues to be very stable, giving the local authority the confidence it needs to safely manage one of Switzerland's most frequented and most spectacular public terraces.

5 CONCLUSIONS

It can be seen from the examples presented above that automated structural health monitoring systems have a great deal to offer the engineers who are charged with the construction and maintenance of bridges and other structures. Questions that arise at any stage during the life of a structure, for example due to modifications to the structure or changes in its loading or condition, can be precisely analyzed using data efficiently provided by such a system. And where concerns remain following any such analysis, a long-term monitoring system can provide economical real-time confirmation, with alarm notification if necessary, that the structure continues to fulfill its function properly and safely. Monitoring systems can thus potentially enable a preferred solution to be implemented, saving alternative solutions that might result in higher costs and more disruption to usage—thus playing an important part in the efforts of the engineering community to maximize efficiency and minimize the impacts of construction work on society and the environment.

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Chapter 23

Strategies in tunnel design, construction, maintenance, inspection and operations

H. Capers, Jr. & L. Jiang
Arora and Associates, P.C., Lawrenceville, NJ, USA

K. Thompson
Arora and Associates, P.C., Auburn, CA, USA

ABSTRACT: Most highway facilities in the United States are governed by design, construction, maintenance, inspection, and operations codes and regulations of the American Association of State Highway and Transportation Officials (AASHTO) and the U.S. Federal Highway Administration (FHWA). However, to date highway tunnels in the U.S. do not have comparable national codes and regulations. Recent events such as the July 2006 ceiling collapse of the I-90 Central Artery Tunnel in Boston, Massachusetts, and the 2005 International Scan on tunnels, have called attention to the need for such national standards. The "Best Practices for Roadway Tunnel Design, Construction, Maintenance, Inspection, and Operations" domestic scan was a key activity initiated to assist in addressing this need. This paper provides a summary of the domestic scan 2009 findings and recommendations, as well as an up to date overview of efforts underway by the AASHTO Subcommittee on Bridges and Structures in advancing national standards and guidance for designing and constructing roadway tunnels.

1 INTRODUCTION

Most highway facility components in the United States are governed by design, construction, maintenance, inspection, and operations codes and regulations of the American Association of State Highway and Transportation Officials (AASHTO) and the U.S. Federal Highway Administration (FHWA). However, to date highway tunnels in the U.S. do not have comparable national codes and regulations. Recent events such as the July 2006 ceiling collapse of the I-90 Central Artery Tunnel in Boston, Massachusetts, and the 2005 International Scan on tunnels, have called attention to the need for such national standards.

The *Best Practices for Roadway Tunnel Design, Construction, Maintenance, Inspection, and Operations* domestic scan, conducted August-September 2009, is one of the activities initiated to assist in addressing the need for national tunnel standards and a national tunnel inventory.

2 US DOMESTIC SCAN PROGRAM

Continuing innovation in the practices of U.S. transportation agencies has brought substantial benefits to the nation. Examples of beneficial innovation range from new materials used in pavements and structures, to new ways of collecting and analyzing information about transportation system users and the environment in which the system operates, to new ways of funding the investments needed to improve public safety and efficiency of travel. Beneficial innovation occurs in any field when new ideas are disseminated and widely adopted by practitioners. Experience in many fields illustrates that expanding the extent of information exchange among practitioners and accelerating the rate of the exchange facilitate innovation.

Experience also shows that personal contact with new ideas and their application is a particularly valuable means for information exchange. U.S. engineering professionals have visited their colleagues in other countries and returned with information that they have subsequently communicated to their domestic colleagues and seen applied to improving domestic practice. The American Association of State Highway and Transportation Officials (AASHTO), the Federal Highway Administration (FHWA), and others have been active in technology transfers at the international level with their involvement in such activities as NCHRP Project 20-36 on "Highway Research and Technology—International Information Sharing."

These experiences have shown that the "scan" approach is a productive means for encouraging the spread of information and innovation. Many international program participants and observers have noted that new ideas are emerging in state and local transportation agencies around the United States, and that faster dissemination of many of these ideas could yield benefits similar to those associated with international information exchange. Domestic scans conducted by various FHWA offices as well as through the NCHRP illustrate the potential value of a domestic scan program. As such NCHRP Project 20-68 A, U.S. Domestic Scan Program was initiated in 2007, a six-year project intended to support scans on about 20 different transportation related topics. A complete description of the Program as well as all products resulting from scans is available on the TRB website (Capers et al., 2011).

A scan entails four key steps. First, knowledgeable people identify novel practices in their field of interest. Second, these people assess the likelihood that these new ideas might beneficially be applied in other settings. Third, new practices that offer the most promise are selected and field visits are made to observe the practices, identify pertinent development and application issues, and assess appropriate technology transfer opportunities and methods. Finally, the results of the initial steps are documented for use by those who participated and for others to apply.

Effective scans both supplement and make use of other mechanisms for information exchange such as publications in trade and professional journals, conferences, and peer-to-peer forums. A scan program focuses on face-to-face discussion of current experience, providing opportunities for a uniquely rich exchange of information that is difficult or impossible to replicate through written materials, telephone conversations, and e-mail correspondence. The informal discussions among the group of visitors participating in the scan contribute to the extraction of useful information from the individual members' observations. Executing an effective scan program requires sound understanding of the topic areas to be considered, insightful selection of topics and new ideas to be observed, careful selection of participants who can provide useful insights from their observations, and thoughtful documentation and dissemination of each scan's results. Managing the domestic scan program additionally requires that resources be conserved by not duplicating the information exchange activities of others.

The domestic scan program is broad, considering any innovative practices of high-performing transportation agencies that could be beneficially adopted by other interested agencies. Each scan might span a one to two week period, and entail visits to two to six sites, possibly geographically dispersed. The program includes annual cycles of topic selection, scans, and documentation.

The purpose of each scan and of the program as a whole is to facilitate information sharing and technology exchange among the states and other transportation agencies, and identify actionable items of common interest. While scans have been shown to be an effective means for encouraging innovation, the overall program will include activities to explore alternative methods of identifying emerging new practices and disseminating information about these practices to other practitioners.

AASHTO and NCHRP identify scan topics, based on suggestions submitted by state DOTs and FHWA; multiple topic proposals may be combined into a single scan. Each scan is planned and conducted with a scan team chair (or co-chairs) and 8 to 10 scan-team members. A subject-matter expert, working with the scan-team chair and members, is responsible for (a) conducting a desk scan; (b) defining the appropriate duration of the scan, its technical structure, and other factors likely to influence planning of the scan; (c) preparing scan technical materials; and (d) preparing a report of the scan. AASHTO and NCHRP identify scan team chairs and members. The scan-program management team receives preliminary scan-topic descriptions from NCHRP; plans, executes and documents scans, including securing NCHRP approvals of interim and final products; and prepares an annual report of the domestic scan program's activities. The management team works with scan-team chairs to select subject-matter experts. The priority and timing of each scan depends generally on availability of supplemental funding and advice of the management team, as well as the panel's priorities and conditions specific to each topic.

A nine-member team was assembled by the American Association of State Highway and Transportation Officials (AASHTO) to conduct this scan on *Best Practices for Roadway Tunnel Design, Construction, Maintenance, Inspection, and Operations*. Members of the team were recruited from a variety of agencies with interest in the subject; two representatives came from FHWA, five representatives came from State Departments of Transportation (DOT), and an academic representing the Transportation Research Board (TRB) Tunnels and Underground Structures Committee (AFF60) joined the team. The team was supplemented by a report facilitator from the private sector hired by the managing consultant, Arora & Associates, P.C.

The scan's focus was on highway tunnel design and construction standards practiced by state DOTs and other tunnel owners, maintenance and inspection practices, operations including safety as related to emergency response capability, specialized tunnel technologies, and inventory criteria used by tunnel owners. Included were consideration of fire suppression, traffic management, and incident detection and management. Also included were analysis, design, and construction repairs of existing tunnels.

The scan's focus was dictated by the NCHRP Project Panel to be an investigation of highway tunnel design and construction standards practiced by state DOT's and other tunnel owners, maintenance and inspection practices, operations including safety as related to emergency response capability, specialized tunnel technologies, and inventory criteria used by tunnel owners. After developing a more detailed research plan the scan team further defined the scope of study to include the investigation of fire suppression, traffic management, and incident detection and management and analysis, existing design criteria being used, construction methods being practiced and methods of repairs for existing tunnels.

Based on a literature search, review of various agencies' websites and personal knowledge of the scan team members potential host agencies were selected based on identified potential successful strategies, emerging technologies being employed or that had known potential lessons learned that would benefit the industry. Another criterion was that they had significant tunnels in their inventories. Hosts along the east coast included the Chesapeake Bay Bridge and Tunnel District, the Massachusetts Turnpike Authority, the Port Authority of New York and New Jersey, and the Virginia Department of Transportation (DOT). Hosts in the western U.S. included Caltrans, the Colorado DOT, the Washington State DOT, the Seattle DOT, and Sound Transit in Seattle. In addition to site visits with scan hosts, the team also conducted web conferences with representatives from the Alaska DOT, the District of Columbia DOT, and the Pennsylvania DOT.

3 SUMMARY OF INITIAL FINDINGS AND RECOMMENDATIONS

The scan team identified a number of highway tunnel initiatives or practices of interest that they felt should be considered by AASHTO and FHWA for further evaluation and possible immediate nationwide implementation. In their initial Summary Report on the scan, the team recommended that eight of these initiatives or practices, briefly described below, be considered for implementation (Ralls et al., 2010).

1. *Development of standards, guidance, and best practices for roadway tunnels*

Design criteria for new roadway tunnels need to be developed and should be developed considering:

- Performance-based construction specifications
- Design recommendations for extreme events (manmade and natural) and tunnel security and blast, lifeline, etc
- Design criteria for vertical, horizontal clearances, and sight distance
- Criteria for tunnel design life and future maintenance for structural, mechanical, electrical, and electronic systems
- Criteria for new tunnel load rating
- Seismic design criteria for one-level versus two-level design events
- Americans with Disabilities Act (ADA) requirements for emergency egress
- Placement and layout of the tunnel operations center

Similarly standards need to be developed governing the rehabilitation of existing tunnels should consider obsolescence, tunnel design life, high-performance materials, and existing geometry to maximize safety and system operation.

Tunnel systems are generally complex and expensive in terms of capital costs. The use of peer review teams and technical advisory panels with subject matter expertise should be considered in developing site-specific criteria. Risk management of complex systems is essential. Redundancy of systems is vital. Employing the use of a Supervisory Control and Data Acquisition (SCADA) system, a computer system that monitors and controls infrastructure or facility-based processes may be needed.

Standards need to be developed providing contract procurement guidelines for roadway tunnels to include design-bid-build, design-build, design-build-operate-finance, etc., considering to the extent applicable the Underground Construction Association's "Recommended Contract Practices for Underground Construction."

Design and construction standards and guidelines for tunnel construction methods need to be developed including guidance on the use of Tunnel Boring Machines versus conventional tunneling, design criteria including seismic design, and lifeline requirements. The guideline should also consider conventional tunneling methods include the Sequential Excavation Method (SEM), the New Austrian Tunneling Method (NATM), the Analysis of Controlled Deformations (ADECO), and cut-and-cover.

According to information made available by the National academies, some of the above topics are to be addressed in a proposed research project to develop LRFD design specifications and guidance for new and existing tunnels that was submitted to NCHRP in 2009 by the AASHTO Subcommittee on Bridges and Structures. As of this writing, a contract for that project is being initiated by the National Academies on behalf of AASHTO through the National Cooperative Highway Research Program (NCHRP).

2. *The team recommends that the owners develop an emergency response system plan unique to each of their facilities that takes into account human behavior, facility ventilation, and fire mitigation*

Owners should strongly consider having a fire ventilation study performed and a fire ventilation plan developed and adopted for each facility. The design of a tunnel to adequately address emergencies should take into account the realistic spread of fire and smoke in the tunnel including toxic gases and heat, and the effect of different types of ventilation systems on the fire, including fire suppression and deluge systems if so equipped.

In general, the scan team found and recommended that facility owners should improve their procedures to direct the public to safety. The fire plan should be consistent with users' instinctive response to a fire, and the operation of all tunnel fire response systems should be consistent with this behavior. Consider better signage, intelligible public address systems, etc., including recommendations for these from the 2005 "Underground Transportation Systems in Europe: Safety, Operations, and Emergency Response" (Ernst et al., 2006).

Further study and research is needed on how fire and smoke spreads in a tunnel and how people react in emergencies. Consider the research topics related to fire that were developed during the AASHTO workshop on tunnel safety and security research needs held November 2007 in Irvine, California.

3. *Develop and share inspection practices among tunnel owners*

The scan team found the best tunnel inspection programs they observed had been developed from bridge inspection programs. In many cases, bridge inspectors also performed the structural inspection of tunnels. Therefore, the team recommended that any tunnel inspection program being considered by the Federal Highway Administration be as similar as possible to bridge inspection programs.

For much the same reasons stated above the team recommended that those components of the tunnel that carry or affect traffic should be load rated in accordance with the *AASHTO Manual for Bridge Evaluation* to the extent possible, e.g., roadway slabs and floor systems that carry traffic. Further, different operational conditions should be considered in the analysis. Also, owners are advised that structural analyses should be performed on non-traffic-carrying components such as plenums, plenum walls, and hangers as their physical conditions change, as they are modified, or as the loads that they are to be subjected to change, such as air forces if fans are upgraded.

The team also identified the need to develop recommended practices for inspection frequencies, minimum coding requirements, and a federal coding manual. Current practice in place by owners visited was one-to-five years for structural inspections, and daily to yearly for mechanical and electrical inspections depending on the level of inspection. Maximum frequencies need to be set, and owners should be encouraged to develop actual frequencies based on a risk-based analysis of hazards due to condition, deterioration, and performance history.

The scan team felt strongly that inspection practices need to be shared among tunnel owners in five areas. First, inspection of submerged tunnels using sonar scans (host locations visited in Virginia and California have submerged tunnels). Second, tunnel inspection training needs to be developed taking into consideration all aspects of the tunnel structure and systems. Third, tools need to be developed to find voids behind tunnel linings. Fourth, coordinated closing of the tunnel overnight to do as much maintenance and inspection as possible. Fifth, share inspection manuals, e.g., the manuals of the Massachusetts Turnpike Authority and the Port Authority of New York and New Jersey.

4. *Consider inspection and maintenance operations during the design stage*

The scan team found that during the design phase, inviting all disciplines into the design results in a better product. The proper design of a tunnel should address future inspection and maintenance of all tunnel systems and equipment by providing for adequate, safe and unimpeded access to all components. Figure 1 shows limited accesses for inspection and maintenance.

This can be accomplished by bringing together all engineering disciplines that will have to be accommodated in the tunnel over its service life. While the scan team understands that tradeoffs must be made between access and a practical design, these tradeoffs could have cost and safety impacts for maintenance and inspection over the life of the tunnel. Owners should take care to weigh these tradeoffs to develop the best possible scenario to best accommodate competing needs.

Figure 1. Tunnel access for inspection and maintenance.

5. *Develop site-specific plans for the safe and efficient operation of roadway tunnels*

The scan team recommends that all owners develop a concise site-specific operations manual that includes tunnel incident response procedures and training; safe ventilation procedures; safe traffic control guidelines; and general maintenance procedures such as tunnel washing guidelines, fan and bearing maintenance, etc. The manual should include training guidelines and training schedules for all personnel. A separate incident response manual should be developed to outline procedures that will require various community, police, fire, and emergency services response in the event of catastrophic incidents. Perform periodic drills including table-top exercises with the appropriate agencies. The operating procedure should consider safety for the public and the owner personnel.

The scan team felt that it was in the best interest of tunnel owners to implement state-of-the-art video surveillance and communication systems. These systems provide numerous benefits, e.g., incident response, traffic management, and increased security. In Massachusetts, the "Operations Control Center" becomes a "Command Center" in times of extreme events/emergencies, etc., see Figure 2.

The scan team found a best practice of lane closure or changing traffic direction, e.g., pneumatically-activated lane delineators and zipper barriers that provide for reversible lanes and barriers through tunnels and tunnel approaches.

The scan team findings support owners restricting hazardous cargo through tunnels. In the event of no alternate route, a well-defined emergency response and fire ventilation plan should be in place. Restricted hours of tunnel operation for hazardous cargo are an option, e.g., hours from 3 a.m. to 5 a.m. under controlled conditions.

6. *A tunnel includes a long-term commitment to provide funding for preventive maintenance, upgrading of systems, and training and retention of operators*

The decision to build a tunnel is a long-term commitment on the part of the owner. The tunnels which include functional systems such as ventilation, fire suppression, and electrical/mechanical components are complex structures with more intensive needs for maintenance and operation than traditional transportation facilities. Owners are strongly encouraged to have a proactive plan, considering life-cycle costs, developed to address needs for preventive maintenance, upgrading of systems, and training and retention of operators. A target level of condition, system reliability, and performance should be established for the facility to

Figure 2. Operations Control Center.

guide operators and owners for current and future decisions which will require manpower or funding.

System components become obsolete and replacement parts difficult to find as equipment ages. In particular, electronic equipment such as computers, SCADA systems, and sensors become obsolete or are no longer supported by their original manufacturers sooner than mechanical equipment. Periodic upgrades are vital to keep all systems functioning reliably.

The team's findings resulted in a recommendation that a separate fund be dedicated to support an agency's tunnels for the entire service life. Agencies should work with local planning organizations to accomplish this task. The financial management plan for tunnels should not only include first costs for construction, but should also address future preservation and upgrading needs. The scan team found that without this dedicated fund, the funding for tunnel upgrades does not compete well with system-wide needs for traffic signals, pavement preservation, etc. As a minimum, the team highly recommended that funding to provide the ability to buy replacement parts be established or at least planned for when the tunnel is being built.

The team recommended that as a part of the previously mentioned standards, when agencies develop tunnel preservation guidelines to guide funding decisions, (e.g., for concrete repair and washing of walls). Training, retention, and a succession plan should be developed for tunnel operators. The scan team found best practices that fostered pride of ownership, a "home away from home" culture and can-do-anything attitude.

7. *Share existing technical knowledge within the industry to design a tunnel*

The team found that technical knowledge that exists within the industry was not commonly available to owners. This information is vital to tunnel owners to provide them with a range of practical tunnel design option. This would include using domestic and international tunnel scan information, past project designs, construction practices, emergency response best practices, and subject matter experts. Value engineering can improve technology transfer with limited owner experience in tunnel systems.

Design documents including calculations and as-built documents should be filed electronically and be easily retrievable by the controlling owner, with appropriate back-up copy, e.g., on microfilm.

In addition the team found that there where many success stories to be shared amongst tunnel owners themselves. Recognizing security concerns of tunnel owners, the scan team believes that actual details and best practices used in tunnels should be shared with prospective and existing tunnel owners without identifying the specific facilities where these details and practices are used. More effort needs to be put into making this happen by AASHTO, FHWA, and other industry leadership and associations.

In August of 2010, AASHTO published the first version of a roadway tunnel manual, the "Technical Manual for Design and Construction of Road Tunnels—Civil Elements, First Edition" (AASHTO, 2010) (Figure 3). This manual was originally published by FHWA, and later adopted by AASHTO Highway Subcommittee on Bridges and Structures (HSCOBS).

8. *Provide education and training in tunnel design and construction*

The scan team findings support training and development for owner agencies. Currently, there are few Civil Engineering programs in the U.S. that offer a graduate course in tunneling.

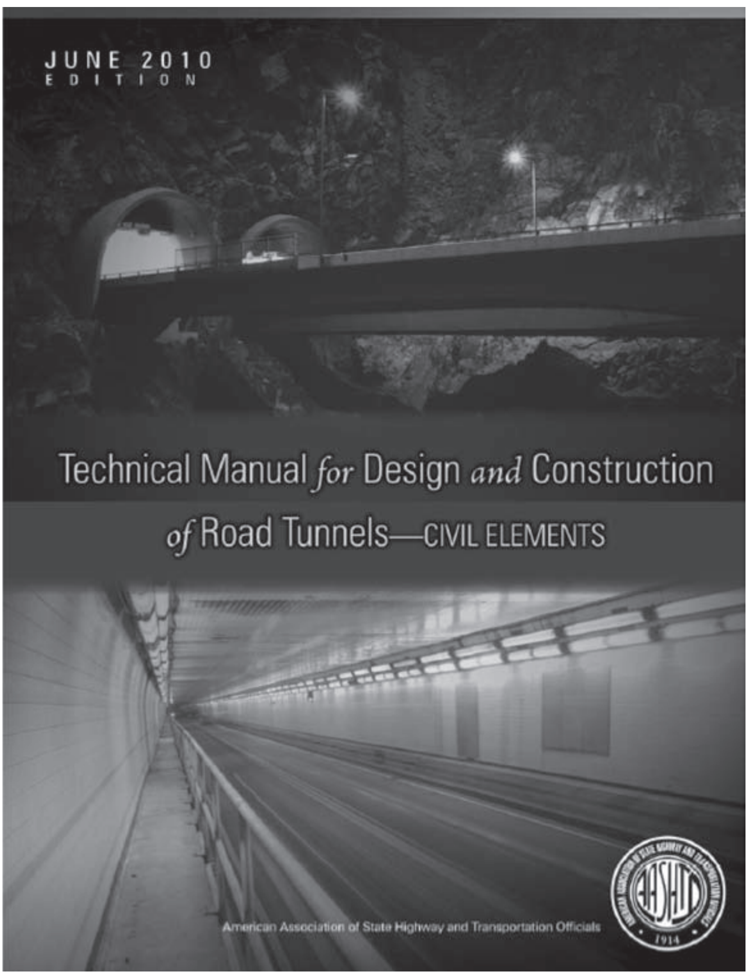

Figure 3. Technical Manual for Design and Construction of Road Tunnels—Civil Elements.

It is very likely that civil engineers are not exposed to tunneling. Many DOTs do not have tunnels in their transportation systems, others built their last tunnel 20–30 years ago and, therefore, the in-house expertise is either non-existent or out of date. The number and magnitude of tunneling projects is projected to increase dramatically in the next few years. The current offering of short courses allows engineers to acquire the nomenclature in tunneling, but not the working knowledge necessary to design, manage, review, and specify tunnel projects.

The scan team recommended that highway tunnel owners and FHWA provide their engineers with access to education and training on tunnels available through academia and industry. This involvement would also help direct academic research on tunneling. On-line courses and certificates on tunneling of international reputation would allow one to acquire up-to-date information and working knowledge in design and construction of tunnels.

4 PLANNED IMPLEMENTATION

The implementation of scan team recommendations will be a step in the process of developing national standards and guidance. Scan findings will also provide data for consideration in the development of a national tunnel inventory. These activities will assist the AASHTO Highway Subcommittee on Bridges and Structures (HSCOBS) Technical Committee for Tunnels (T-20) and FHWA in developing best practices for roadway tunnel design, construction, maintenance, inspection, and operations of existing and new tunnels.

The lead group for implementation of scan recommendations is anticipated to be T-20 in conjunction with FHWA and the TRB Tunnels and Underground Structures Committee, and working with the National Fire Protection Association (NFPA) and other tunnel organizations. The scan team will present its findings and recommendations to T-20 during the January 2010 TRB Annual Meeting. Initial scan team efforts also include distribution of the FHWA Tunnel Safety brochure that was developed following the 2005 international tunnels scan previously mentioned and providing additional information on the FHWA tunnels website. Other planned activities include coordination and development of research statements related to tunnel needs. The scan team also has plans to make technical presentations, webinars, and written papers in various publications and at national meetings and conferences sponsored by FHWA, AASHTO, and other organizations to disseminate information from the scan. A detailed discussion on planned activities is contained in the scan team's final report (Ralls et al., 2011).

5 CONCLUSIONS

The scan team identified a number of highway tunnel initiatives or practices of interest for nationwide implementation or for further evaluation for potential nationwide implementation. The implementation of the scan team's top eight recommendations will be a step in the process of developing national standards and guidance. Scan findings will also provide data for consideration in the development of a national tunnel inventory. These activities will assist the AASHTO Highway Subcommittee on Bridges and Structures (HSCOBS) Technical Committee for Tunnels (T-20) and FHWA in developing best practices for roadway tunnel design, construction, maintenance, inspection, and operations of existing and new tunnels. Many of the scan team's findings have already been incorporated into a large variety of initiatives being advanced through NCHRP projects and several FHWA efforts to develop national standards for highway tunnels on US highways.

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Chapter 24

Identifying and sharing best practices in bridge maintenance and management

M.M. Myers & H.A. Capers, Jr.
Arora and Associates, P.C., Lawrenceville, NJ, USA

ABSTRACT: Over the past two years, multiple studies have been conducted through the National Academies Cooperative Research Programs on the topic of determining best bridge management practices by various state departments of transportation. The studies discussed in this paper are NCHRP 20-24(37)E and Domestic Scan 07-05. Both of these studies focused on states' approaches to continuous bridge maintenance and concluded in recommendations for ways to incorporate the most effective bridge maintenance programs in practice around the country. Recommendations focused on proper use and allocation of resources and funding, preservation and preventative maintenance, advancing and improving effectiveness of inspection, and developing closer tracking of bridge condition and managing information. By sharing the results of these studies, it is hoped that other states will be motivated to modify and improve their practices and that applying successful practices will extend service life of bridges across the country.

1 INTRODUCTION

1.1 *Motivation*

Maintaining bridge inventory is an important and complex task for state department of transportation (DOT) agencies, as well as other bridge owners. It is a key aspect in the successful operation of transportation networks. Many states are in need of improvements in their means and methods of bridge management and condition assessment, while still operating within established budgets. By investigating states applying successful strategies to bridge maintenance, and sharing the results of these studies, it is hoped that other states will be motivated to modify and improve their practices and that applying such practices will extend service life of bridges across the country. Establishing guidelines for successful approaches to bridge maintenance is a time consuming and complex task. It involves working closely with state DOTs to obtain information on precisely which practices can and should be applied in other areas of the country.

Maintenance of a subset of bridges, such as the collection of bridges all owned and maintained by a specific state DOT, requires decision making in order to most effectively use resources and maintain a safe and functional transportation network. One important decision to be made is determination of the priority of needs for the bridges from minor maintenance to full replacements. Other decisions involve the use of new methods and materials, as well as the improved use of familiar means, to improve productivity during construction and reduce initial and life cycle costs for bridge projects. Organization and updated records can help an agency effectively maintain their bridges as well. By staying informed of the conditions of the bridge inventory, DOTs can address minor problems before they become severe and require costly means to repair or replace. Having a more formal, detailed set of guidelines for inspection would help greatly in this effort.

1.2 *Recent studies*

Over the past two years, multiple studies have been conducted through the National Academies Cooperative Research Programs on the topic of determining best bridge management practices by various state DOTs. The studies discussed in this paper are Domestic Scan 07-05 (Weykamp et al., 2009) and National Cooperative Highway Research Program (NCHRP) 20-24(37)E (Spy Pond et al., 2010). Both of these studies focused on state's approaches to continuous bridge maintenance and concluded in recommendations for all states for ways to incorporate the most effective bridge maintenance programs in practice around the country.

The subject of Domestic Scan 07-05 was to identify and catalog automated bridge maintenance decision support systems employed by state DOTs across the country. These systems help to reduce impacts to motorists on the transportation networks during maintenance of bridges and also enable DOTs to identify cost effective approaches to repair and replacement of bridges for both initial cost and life cycle cost. The scan team interviewed bridge engineers in various state agencies who were involved in the development of bridge maintenance programs to identify current effective systems and their costs and benefits. The team then compiled the information and concluded in recommendations for applications of the effective procedures in use as identified through the scan.

The American Association of State Highway and Transportation Officials (AASHTO) Standing Committee on Quality Performance Measures and Benchmarking Subcommittee began NCHRP Project 20-24(37)—Measuring Performance Among State DOTs in 2004. The subject of this project was to gather and compare information on transportation system management practices in multiple disciplines across various agencies. The goal was then to publish best practices from the data gathered from responsive agencies and discuss lessons learned through the research. The specific topics in the first few segments of this project were on-time, on-budget project delivery (Crosset et al., 2007), pavement smoothness (Spy Pond et al., 2008), and traffic safety (Spy Pond et al., 2009). The most recent installment of this project is the one focused on in this paper, addressing bridge condition. Similar to the scan discussed above, the subject of this project was to identify and compare successful efforts by transportation agencies in order to identify approaches to bridge maintenance that could be applied in other areas of the country to improve the condition of bridges everywhere. This study was supplemented by information from other research projects and reports, including AASHTO's "Bridging the Gap" (AASHTO 2008) regarding the overall condition of the nation's bridges, the Federal Highway Administration (FHWA) Long Term Bridge Performance program (Turner-Fairbank HRC 2011), and the American Society of Civil Engineers (ASCE) Structural Engineering Institute Workshop on Enhancing Bridge Performance (ASCE 2008).

2 SCAN 07-05 BEST PRACTICES IN BRIDGE MANAGEMENT DECISION-MAKING

2.1 *Procedure*

Domestic Scan 07-05 used various approaches to collecting information about the policies and procedures of various state and local government agencies on the topic of bridge management and maintenance. The scan team visited seven sites for meetings with employees of a total of 13 DOT agencies. These meetings and discussions provided the most detailed information for the study. Another aspect of information gathering came from reviewing current client documents such as manuals and guidelines which discuss states approaches to certain aspects of bridge management. These reviews provided information about the methods of eleven additional DOT agencies. Surveys were also sent to DOTs asking questions to expand upon what could be found in the documents. In the end, all of this information was compiled and summarized to describe the findings.

2.2 *Results*

The results of the scan comprised many data about the practices of specific state DOT agencies. The agencies focused on in the study were the following:

- California
- Delaware
- Florida
- Michigan
- New York
- Ohio
- Oregon
- Virginia
- Washington
- Wisconsin

The categories of these findings include the following:

- Bridge Maintenance—cleaning, minor and major repairs, component treatments and replacements, and rehabilitations.
- DOT Organization—central office, district offices, and field crews.
- DOT Inventory—most state-owned structures above or below highways.
- DOT Maintenance Crews—structural work (bridges, culverts, etc.) or general maintenance.
- Identification of Maintenance Needs—district offices receive needs for maintenance from the central office. Needs are addressed by priority, and some are sent to contract or enter procedures (bridge replacement, etc.).
- Priorities of Maintenance Needs—Inspectors and maintenance crews identify and prioritize maintenance needs. The needs are reviewed and assigned to crews, or to districts for contract or for programming through the central office.
- Performance Measures and Priority Indicators—Performance measures are values that show the condition of the bridges in the network and the bridge program's accomplishments. Priority indicators are values that give the category of maintenance and ranking for work of bridges.
- Maintenance Budget—funding for maintenance and contracts.
- Maintenance Planning and Programming—district-formed work plans for bridges based on crew and funding availability.
- Contracting Mechanisms—various types of contracts that dictate repairs or rehabilitations of bridges.
- Preventive Maintenance—routine cleaning, repairs, treatments, partial replacements, retrofits, etc.
- Federal Funds for Preventive Maintenance—a fraction of states have applied for and received permission from FHWA to use federal funds to perform preventative maintenance.
- Maintenance Tracking and Accomplishments—data systems store and track maintenance needs.
- Effectiveness of Maintenance—improving performance measures, quick and reliable maintenance, and maintaining good performance.
- Data Systems—data systems are used by DOTs to record data, track maintenance needs and work performed, and prioritize and analyze the information.
- Materials and Methods—new materials and methods are able to be tested by DOT crews.

Specifics of the findings within each of these categories can be found in the Scan Team Report, Scan 07-05, Best Practices in Bridge Management Decision-Making.

3 NCHRP 20-24(37)E MEASURING PERFORMANCE AMONG STATE DOTS, SHARING BEST PRACTICES COMPARATIVE ANALYSIS OF BRIDGE CONDITION

3.1 *Procedure*

In the first step of the study, selecting performance measures to compare the 34 participating states was the goal. Figure 1 depicts which states participated in the study. Possible performance measures were determined by reviewing the National Bridge Inspection (NBI) standards and literature on states current standards. States were then sent surveys in order to find out more about the current practices. After all of these results were reviewed and discussed, it was determined that NBI data would be used for comparison, even though some states felt that it had limited value. It was also decided that various other measures of bridge performance would be used, such as element-level data, which had greater value than the NBI data, but not as wide a range of information. These multiple measures were decided on in order to emphasize various aspects of bridge performance, since bridge performance is not based on simply one measure and certainly varies in its focus across different agencies. It was also determined that the bridges included in the study would be limited to most standard type, state-owned highway bridges in order to allow for fair comparisons between states. Preliminary results of an analysis of these measures was discussed with the Transportation Research Board (TRB) Bridge Preservation Joint Subcommittee and the Bridge Management Committee at the 2010 TRB Annual Meeting, and from the feedback, the final measures were set.

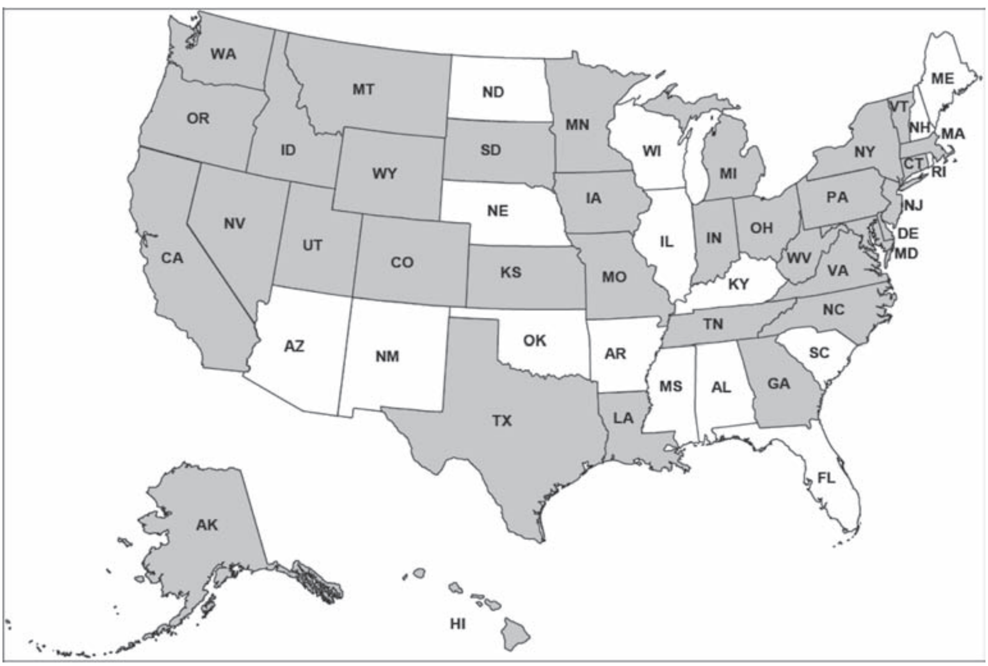

Figure 1. NCHRP 20–24(37)E final report—participating states (in gray).

The next step was to measure performance in the final categories, which included current performance as well as change in performance over the past 10 years in the following four categories:

– NBI Sufficiency Rating
– NBI Structural Deficiency Status
– NBI Posting Status
– An overall "Good" condition classification

These measures were weighted by bridge deck area. Another aspect of evaluating the performance measures was to group the states in order to compare states with similar features. The features by which the states were sorted were the following:

– Geographic region
– Traffic levels
– Bridge age distribution
– Level of bridge replacement activity
– Unit replacement costs

The geographic region category showed the most variety between groups. These regions can be seen in Figure 2 below.

After all categories were analyzed, charts were prepared showing relationships between the states performances for each measure. Then one state per geographic region was chosen which performed highly compared to the other states in its region. The respective bridge maintenance/management program manager for each of these top-performing states was interviewed via telephone to find out which practices contributed to their success in bridge management.

The last step in the analysis was to recommend bridge performance measurement techniques for future use in states all across the country based on the NBI data and the input from the states in the study.

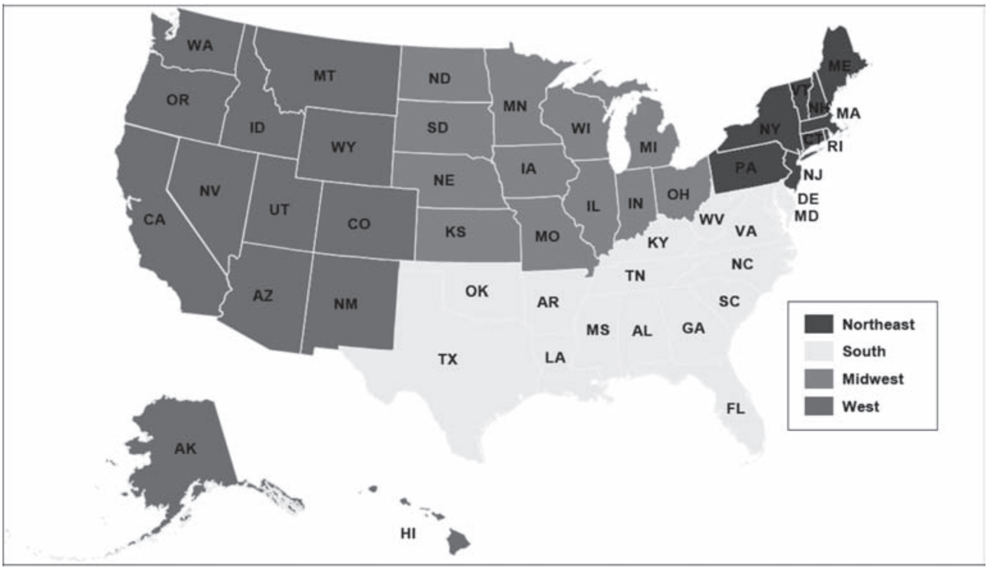

Figure 2. NCHRP 20-24(37)E final report—census regions used for geographic peer groups.

3.2 *Results*

The results of the interviews of the states included information regarding responsibilities and organization of departments or levels within the agency, the allocation of resources, performance measures, and design, construction, and maintenance practices. The states' bridge engineers were the representatives involved in the interviews, with other state contacts involved in the reviews of the results. The four top performing states that were chosen for the interviews were Kansas, Georgia, New York, and Utah. Following are brief summaries of the results from these states.

3.2.1 *Kansas*

The areas in which Kansas showed success were current Structurally Deficient Bridges and Bridges in "good" condition and changes in Bridges in "good" condition over the past 10 years. Bridge management is shared between three different departments in the state agency, including the Design Bureau performing inspections, organizing bridge preservation, and containing the bridge management activities, which is believed to speed up the process of responding to problems with bridges. Another activity which is believed to help maintain bridges in good condition is performing element-level inspections to collect data on bridge conditions, aside from collecting the standard NBI data.

As far as budget management is concerned, district offices identify the light maintenance activities that they can perform, and then make a list of the other work needed, which can be carried by one of five bridge budgets at the state level. The overall budget for major bridge projects is established by KDOT's capital plan. Aside from setting and managing budgets, KDOT has also aimed at reducing initial costs for bridge design and construction. Standard designs are provided to improve cost, quality, inspectability, and reparability of bridges, including haunched and post-tensioned slabs and prestressed concrete, as well as minimization of expansion joints, among other things. One final key in the success of KDOT's bridge management is good communication between various departments and levels. Regular meetings and discussions between inspection, maintenance, design, and management staff are held to determine what actions are necessary and possible.

3.2.2 *Georgia*

Like Kansas, Georgia also showed success in the areas of current Structurally Deficient Bridges and Bridges in "good" condition, ranking third in both categories. Georgia also shares bridge management tasks between multiple offices and departments. The budget for bridge funding is heavily based on federal Highway Bridge Rehabilitation and Replacement (HBRR) funds. Another common practice between Georgia and Kansas is the collection of element-level data during bridge inspections. Although NBI data is also collected and used as a measure of bridge condition, the element-level data tends to be used as a more reliable measure of the needs for maintenance for planning purposes. Bridge inspector recommendations tend to control the maintenance work that is performed on bridges in Georgia. The expertise at the district level is felt to lead to successful bridge management, though resources are more limited at this level.

Another similarity to Kansas is Georgia's use of standard designs that minimize costs. These standard designs have lead to mostly concrete bridges which require less maintenance than steel bridges. Simple spans have also been prevalent as a result of this approach, with spans being made continuous for live load over piers to minimize expansion joints. GDOT has also increased its spending on transportation significantly through capital programs which use state funds and bonds.

3.2.3 *New York*

New York State was chosen for slightly different reasons than Kansas and Georgia. Instead of mostly being chosen on current conditions, New York was chosen for its improvements over the past 10 years in three of the four categories (Structurally Deficient Bridges, Bridges with low Sufficiency Rating, and Bridges in "good" condition). One reason that NYSDOT states for its successful performance in bridge maintenance is that there are many staff allocated to bridge management throughout all districts within the state. Almost 13% of all maintenance personnel are connected to bridge maintenance. NYSDOT cites many other reasons for their success as well, including stability within their staff, scheduled maintenance activities, and increased investment in maintenance. And similarly to Georgia and Kansas, NYSDOT performs element-level inspections and has aimed at making bridges more inspectable.

New York boasts a well-established and effective bridge condition rating system, which helps in determining the needs and priorities across the state. Data systems are used to analyze and organize information obtained through inspections. Each district also has a team who manages local maintenance needs, and responds quickly when needed. NYSDOT cites a focus on maintenance, as opposed to replacement, as a primary reason for the success of the state's bridge management program.

3.2.4 *Utah*

The final top-performing state interviewed for the study was Utah. Like New York, Utah was chosen primarily for its improvements over the last 10 years. Utah ranked in the top three for all four categories for improvements and ranked second for current condition in the Bridges in "good" condition category. Like many other states, Utah shares the responsibility for bridge maintenance and management amongst many departments and levels. Also like the other top-performing states, Utah collects element-level data on its bridges, aside from collecting NBI data. UDOT has an established method of budgeting as well, which helps for managing bridge project needs and allocating resources efficiently. Bridges are classified based on vulnerability and functionality descriptors and prioritized accordingly for funding.

Similar to other states, UDOT has a policy of focusing on new or inventive methods, such as design/build and accelerated bridge construction, to speed contracting and construction and lower costs of replacing bridges, which also leads to more inspectable and maintainable standard designs.

3.2.5 *Summary*

From the interviews with these four states, three major themes emerged as contributors to successful bridge management:

- Emphasize bridge investment, beyond funds available through the HBRR program.
- Focus on bridge preservation, especially since it is largely infeasible to replace all deteriorating bridges.
- Construct maintainable bridges that are easy to inspect and repair through standard designs.

For more information on these themes, and full descriptions of the top-performing states' practices, see the NCHRP 20-24(37)E Final Report.

4 COMMON CONCLUSIONS AND GENERAL RECOMMENDATIONS

After reviewing the results of these two studies, multiple general themes arose. Recommendations can be made within these categories for future action in order to improve bridge management techniques throughout the nation. The general themes or categories can be summarized as inspection procedures, data organization and tracking, funding and budgeting, and bridge preservation and maintenance.

4.1 *Inspection procedures*

Within the category of inspection procedures, the number one recommendation seems to repeatedly be to implement element-level data collection procedures. In order to be as useful as possible, these procedures should be standardized so that they can be comparable within and across states. This can be achieved partly by adopting consistent interpretations of a set National Bridge Elements, proposed to be developed by AASHTO. As a more general recommendation for improved inspection techniques, it would be beneficial to establish performance measures that would be consistent across all states in order to be able to compare conditions. As part of this, it is recommended that these performance measures be based on deck area, instead of a bridge count. Also, using a good-fair-poor categorization to track performance is believed to be the most useful and simply descriptive way to report the maintenance success of bridges within an inventory. These performance measures should also be appropriate to be used to establish funding levels and ensure comparison across all states. As a final recommendation for improved inspection techniques, especially within the idea of establishing new procedures, the support of inspector training and cross training between maintenance and inspection personnel is key to good quality assurance.

4.2 *Data organization and tracking*

As a continuation of the above ideas, tracking and organizing the data that comes from these inspections is an important part of bridge management. By storing work data and cost data in a comprehensive database, agencies can not only improve cost estimations (which in turn helps with budgeting and planning), but also can keep track of accomplishments and needs that are still unmet. This kind of organization can help to create a feedback loop in order to assure that needs are being organized and that no redundancy is being realized within maintenance requests and project planning, as this can reduce productivity and waste funds. Another way to improve organization within state DOT agencies is to publish data systems that are being used and proving useful throughout the nation. Table 1 lists some of these systems and the states that are using them. See the Scan Team Report for more information on these systems. By using such data systems, modeling and forecasting of bridge conditions can become more accurate. Accurate data tracking can also help future bridge management by allowing tracking of not only current bridge conditions, but also changes in bridge conditions over a period of time.

4.3 *Funding and budgeting*

As mentioned above, improved organization of data can help improve budgeting and funding plans, which is another category of recommendations concluded upon within these studies. Using performance measures, as described in the discussions above, to set funding for bridge maintenance and preservation will help to improve the efficiency of use of resources. It is also recommended to determine the needs within an agency's bridge inventory for the future and establish multi-year programs to treat any deficiencies, using owner-specific objectives to develop needs-based funding allocations. Another way to manage budgets and aim for the most cost effective methods is to use a wide range of alternatives for design and contracting, with the goal of reducing initial costs for new bridges and bridge rehabilitation or replacement projects.

4.4 *Bridge preservation and maintenance*

The last general category of recommendations focuses on preventative maintenance for bridges. It is recommended that agencies aim toward keeping bridges in good condition and not necessarily focus on bridges in the worst conditions first. By only focusing on the worst bridges, bridges that are on the edge of being in poor condition, but are still considered good will not remain in good condition. By addressing these bridges on the edge, an agency's inventory can remain in good condition with less costly repairs. This can be achieved by developing work programs for preservation and scheduling routine maintenance to avoid deterioration before it even begins. As an example, Figure 3 shows cases of preventative

Table 1. Data systems from scan 07–05 team report.

DOT	System
California	SMART BIRIS LP 2000 Pontis IMMS TSN
Delaware	MAXIMO Pontis
Michigan	Oracle TMS MARS MBIS Possible Projects BCFS
New York	MAMIS Bridge Program Worksheet Needs Assessment Tool Bridge Needs Assessment Model
Ohio	BMS Ellis TMS BMRI
Oregon	Pontis MMS
Virginia	Site manager Pontis CrossWalk Optimizer
Washington	Bridgit TRAINS BEISt SI MPET BRL
Wisconsin	HIS

maintenance that are currently conducted on a regular basis in a number of states. Also, by targeting lower-level management, or higher level management with extensive field knowledge and maintenance experience, these programs can be managed efficiently and effectively. It is also recommended that agencies coordinate their work plans through all levels and departments to assign tasks where they can be most effectively carried out (such as within district offices, versus central offices, etc.).

4.5 *Communication*

This discussion lists many various recommendations for effective bridge management within state DOT agencies. However, one key aspect of an efficiently run program, no matter what the topic or goal, is open and clear communication. By preparing a plan to exchange information on the condition of bridges within a state, or with the country, and monitor the techniques and methods being implemented effectively, while discussing most useful

DOT	Preventive maintenance
California	▪ Crack sealing, deck overlays, replace joint seals, painting ▪ Request for federal HBP funds is in review
Delaware	▪ Deck repairs, treatments and overlays, painting, cleaning, pile jackets, scour countermeasures, bearing replacement, and seismic retrofits ▪ Uses federal HBP funds
Florida	▪ Bridge deck cleaning, cathode protection systems; movable bridges: lubrication, adjustment, and general upkeep of mechanical and electrical systems ▪ Does not use federal HBP funds
Michigan	▪ Deck overlays, joint replacement, and painting ▪ Uses federal HBP funds
New York	▪ Remove brush; maintain stream channels; maintain bank protection and walls; clean substructure; seal substructure; lubricate bearings; repair bearings; clean superstructure and deck; repair joints; remove wearing surface; place wearing surface; place membrane; seal deck, curb, sidewalk, and fascia; fill cracks and joints; clean drainage system; spot painting; paint bridges; maintain electrical and mechanical equipment ▪ Uses federal HBP funds
Ohio	▪ Deck patching; overhead loose material removal; scour corrections; resetting bearings; deck repair and replacement; abutment repair; concrete sealing; replacement of deck edges; box culvert installation; approach slab repair and replacement; drainage repair, including scupper extensions, pile encasements, bridge and deck cleaning, resetting/repair bearing devices, scour protection/ channel alignment ▪ Does not use federal HBP funds
Oregon	▪ Washing steel bridges, spot painting, and deck sealing ▪ Does not use federal HBP funds
Virginia	▪ Seal/replace leaking joints, deck overlays, painting spot and zone, cathodic protection systems, electrochemical chloride extraction, scour countermeasures, removal of large debris, fatigue retrofit, concrete deck repairs during overlay, substructure repairs during cathodic protection or electrochemical chloride extraction , sealants for concrete by coating or membrane, bridge cleaning or washing ▪ Uses federal HBP funds
Washington	▪ Bridge cleaning, drain clean/repair, painting ▪ Federal HBP funds used for preservation, not preventive maintenance
Wisconsin	▪ Washing bridge decks, sealing bridge decks, spot painting ▪ Does not use federal HBP funds

Figure 3. Table 14.15 from scan 07-05 team report—preventative maintenance actions.

allocations of resources, bridge conditions everywhere can improve and be more successfully maintained.

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7 Historic bridges

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Chapter 25

New bascule bridge for historic New Bern, North Carolina

E. Kelly
HNTB Corporation, New York, NY, USA

ABSTRACT: The existing swing span that crosses the Trent River had reached its useful life and the North Carolina Department of Transportation (NCDOT) contracted HNTB to provide the expertise to help them with the development of replacement alternatives and the final design for the new movable structure. The outcome of the completion of the Categorical Exclusion document and stakeholder outreach found that the most suitable alternate would be a double leaf trunnion bascule bridge that is situated in picturesque, historic New Bern. The replacement structure needed to maintain the historic character of its surroundings and provide some connection with the previous structure as well. This paper will go into the examination and explanation of the details and materials used for the structure, various components, and more specifically the control house. The discussion will show how their selection helped to meet the project requirements. The control house will be emphasized since this is the most prominent portion of the structure.

1 PROJECT BACKGROUND

1.1 *Existing bridge*

The Alfred Cunningham Bridge, NCDOT Bridge 60 in Craven County, was designed in approximately 1953 and built in 1955. It was 1763 feet long and consisted of a 350 foot long New Bern approach, a 220 foot swing span, and a 1190 foot long James City approach, See Figures 1 and 2. The approaches consisted of 35 foot long spans comprised of a reinforced concrete deck supported on rolled steel beams with a substructure composed of reinforced concrete pier cap bents on octagonal precast piles. The swing span was a center bearing through truss span with a concrete filled grating deck and reinforced concrete safety walks supported on a reinforced concrete circular pivot pier founded on octagonal precast piles. The original control house was previously removed from the swing span and a new house was built behind the rest pier to the side of the James City approach. The bridge typical section consisted of a 28 foot clear roadway width (two 12 foot lanes with 2 foot shoulders) and two 3 foot wide sidewalks elevated 10 inches above the roadway surface.

The swing span provided two 78 foot navigational channels, between the fenders with unlimited vertical clearance when the span was in the open position. An approximate clearance of 14 feet was provided with the swing span in the closed position.

The bridge was in a deteriorated condition and had been classified by NCDOT as structurally deficient and functionally obsolete. The portals of the swing span did not provide sufficient clearance over the roadway which resulted in impact damage. Based upon the rating system of 1 (failed) to 9 (good), rating of some of the critical members of the bridge was as follows:

Deck Expansion Joints: Grade 4 (poor)
Longitudinal Beams: Grade 4 (poor)
Precast Piles: Grade 4 (poor)
Paint: Grade 3 (poor)

Figure 1. Elevation of existing swing span of bridge.

Figure 2. View of roadway on existing swing span of bridge.

Besides its deteriorated condition, the bridge also had inadequate sidewalk width, inadequate railing height and restricted vertical clearance.

The NCDOT gave the bridge a sufficiency rating of 8.0 out of 100 in its 2002 Inspection Report. (NCDOT, 2002) This is a scale based upon a weighted formula relating four unique factors: structural adequacy and safety (55%); serviceability and functional obsolescence (30%); essentiality for public use (15%); and special reductions (13% maximum). The Inspection Report indicated that the estimated remaining life of the structure without major maintenance or rehabilitation was 10 years. Two years earlier it had been estimated to be 8 years.

1.2 *New structure design constraints*

NCDOT determined it was time to replace the existing structure and tasked HNTB Corporation to prepare the environmental studies, conduct stakeholder outreach, develop the alternatives for the bridge replacement and ultimately design the preferred alternative.

1.2.1 *Bridge type*

During the National Environmental Policy Act (NEPA) process (NEPA 1970), 7 alternatives were investigated. These alternatives were 1) permanently remove the existing bridge; 2) rehabilitate existing bridge; 3) replace existing bridge with bascule type movable bridge; 4) replace existing bridge with a lift type movable bridge; 5) replace existing bridge with a tunnel; 6) replace existing bridge with a high level fixed bridge on existing alignment and 7) replace existing bridge with a high level fixed bridge on new alignment. A "No Build" alternative was also considered, but deemed unfeasible since the existing bridge only had 10 more years of useful life. Ultimately the bascule bridge on existing alignment was selected for implementation. This alternative was selected due to its ability to meet the purpose & need, strong agency and community support, minimal environmental impacts, competitive cost and unlimited vertical clearance at the navigational channel.

1.2.2 *Project commitments*

In the development of the environmental documents, Categorical Exclusion, a series of project commitments were made to minimize the environmental impacts (Categorical Exclusion, 2006).

These included:

- Replacement of the structure on the same alignment to minimize impact to the community. This required the bridge to be shutdown for the construction duration, approximately 3 years, but this was determined to be a temporary impact to businesses in the downtown, bicyclists and motorists.
- The use of a vibratory hammer during pile installation to reduce the noise. A vibration monitoring and enforcement program was to be implemented during construction to minimize the amount of vibration effects that the surroundings and buildings, some of which are historic, would be subjected.
- There will be no dredging in the Trent River.
- Existing 45 mph speed limit will be reduced to 35 mph.
- To allow stormwater from the new bridge to discharge directly into the Trent River, the impervious surface of the proposed bridge could not exceed the impervious surface of the existing bridge. To do this, the new cross section of the roadway consisted of two 11 foot lanes with a 4 foot shoulder along the northbound lane and a 2 foot gutter along the southbound lane. A raised sidewalk of 5 feet 6 inches in width on the bridge will be provided adjacent to the southbound lane. This made the out to out of the new bridge 36 feet- 1 inch while the existing bridge out to out dimension was 36 feet- 4 inch (See Figure 3).
- The bridge railing is to be a Texas Classic with arched cutouts. There will be no encroachment into Union Point Park in New Bern. The bridge retaining wall along E. Front Street

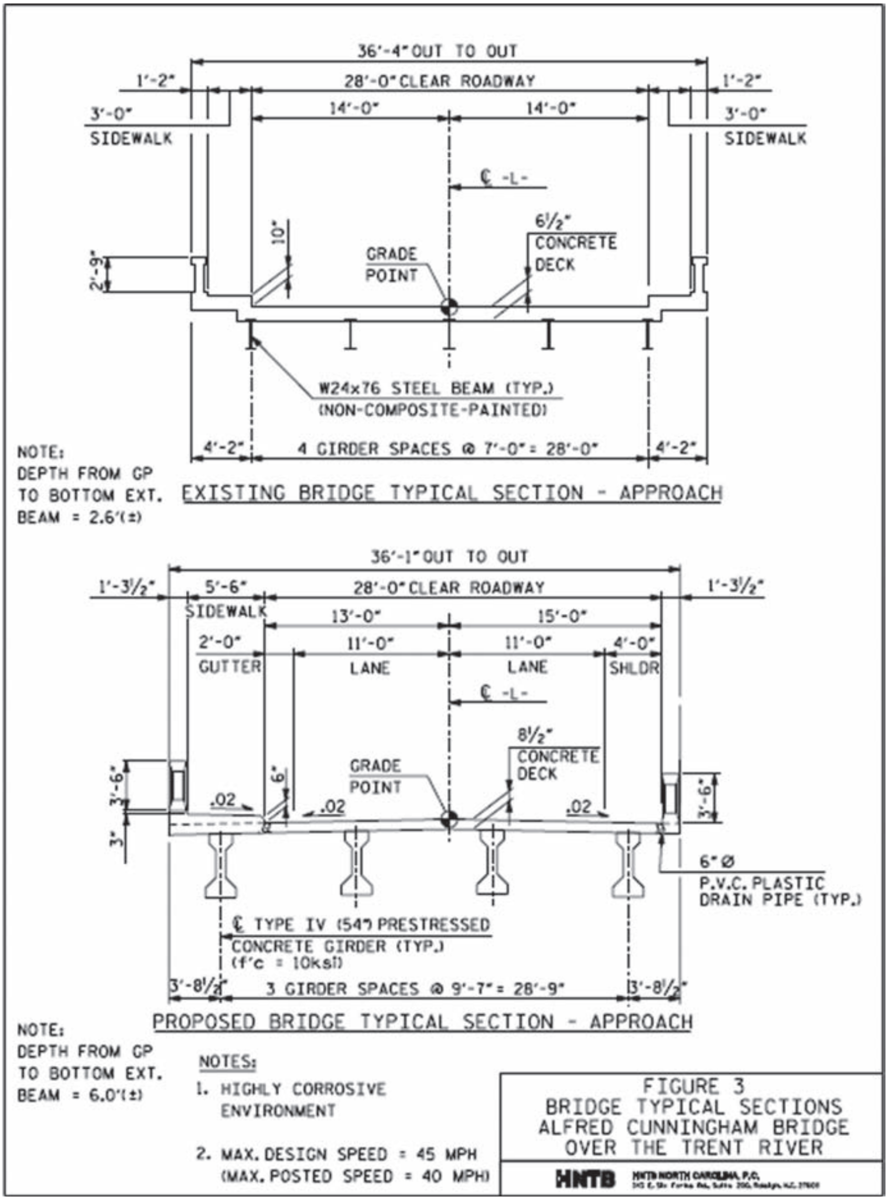

Figure 3. Existing and proposed bridge cross sections (Categorical Exclusion, 2006).

will be covered with brick to match the New Bern Riverfront Convention Center. The color, material and design of the pedestrian railing on top of the retaining wall shall match the existing pedestrian railing used in Union Park. The areas around the retaining wall will also be landscaped.

1.2.3 *Project location*

The bridge was eligible for listing on the National Register of Historic Places. The northern end of the bridge is located in the City of New Bern and is adjacent to the New Bern Historic

District, which features over 150 historic landmarks. Due to the bridge's eligibility for the National Register and its proximity to the New Bern Historic District, NCDOT consulted with the Advisory Council on Historic Preservation, NC-HPO and the New Bern Historic Preservation Commission (HPC) throughout the project to ensure documentation of the existing bridge as well as the appearance and details for the new bridge addressed their concerns. The main concern was the appearance of the new bridge. For this reason, two Bridge Aesthetics Forums (BAF) were held in New Bern to assist NCDOT with establishing the architectural treatment of the bridge. The BAF was composed of local planners, architects, landscape architects, artists, historians and others having special knowledge or skills related to New Bern's architecture and history. Particular attention was paid to bridge type, mass, scale, materials, colors as well as overall treatment of the bridge railing and pedestrian railing, the bridge operator's house, sidewalk design and detailing of the retaining wall at the north end of the bridge. The proposed architectural treatment of the new bridge was then presented in a joint public meeting to receive public comment.

2 NEW STRUCTURE

2.1 *Structure arrangement*

As previously mentioned the new structure placed on the existing alignment was determined to be a bascule bridge with concrete approaches on either side. The movable portion of the bridge is a double leaf trunnion bascule. It is 147 feet from centerline trunnion to centerline trunnion and provides a 90 foot clear navigation channel that is on an 8 degree skew from perpendicular to the bridge, See Figure 4. In the closed position the bridge provides 14 feet 11 inches of vertical clearance and when the bascule leaves are raised to their 71 degree normal open, the span provides unlimited vertical clearance in the channel.

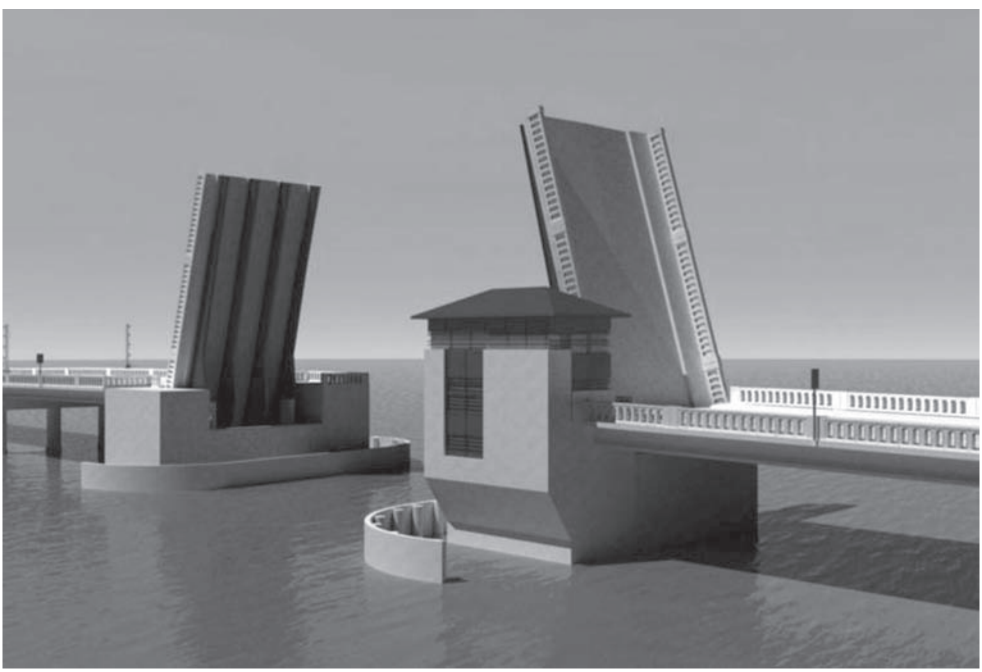

Figure 4. Rendering of proposed new structure.

2.2 *Context sensitive structure*

2.2.1 *Control house*

The new bridge had to be designed to not only accent and highlight the features of the historic City of New Bern, but also to take some of the original characteristics from the older existing structure. During the development of concepts for the replacement structure certain specific areas were concentrated on to make the new bridge an important part of the surrounding community and still retain some details from the original bridge.

The most prominent feature of the new bridge, other than the bascule span is its control house and for this reason special emphasis was placed on the architectural treatment of the building. The control house is a three story structure that provides a critical function for the bridge. The top floor is the operator level. From here the traffic is stopped, communication with mariners is conducted and all operations of the bridge opening are performed. The roadway level is the entrance to the second level that houses the electrical equipment as wells as the bathroom facility for the operator. The third level below the roadway is where the generator, used as an alternate power source, is housed in case of power interruption.

When developing the details for the control house, HNTB's architects continued to look at the historic nature of downtown New Bern, but it was not the intent to make the control house appear historic or have construction details of the buildings from that period. Rather the appearance of the control house would utilize highlights from these building.

Many of the historic structures as well as the new convention center located adjacent to the Union Point Park are constructed of red brick (See Figure 5). This held an important link that the architects felt the control house should utilize. Since brick is not practical for construction on a bridge, the new control house, with its modern features, had the details and colors selected to highlight and match the red brick. The control house built mostly of concrete had the concrete made of the architectural white concrete and a terracota rainscreen wall system along with the building louvers made in red brick color. The combination of the red on white complimented each color. The adjacent convention center features a green patina roof and the architects utilized this by making the control house room constructed of

Figure 5. General view of New Bern.

a standing seam copper hip roof with deep overhangs for sun control. This roof will also turn patina green with time and weather. The modern features of the control house include glass all around to allow 360 degree viewing at the operator level (See Figures 6 & 7).

As previously mentioned, the control house along with other details of the structure including the bridge railing and the retaining wall supporting the downtown approach road were presented to the various stakeholders during two Bridge Aesthetics Forums to allow discussion explanation and receive input on the materials, appearance and details for the bridge.

2.2.2 *Bridge railing*

To ensure some continuity of the original structure was incorporated into the new structure a concrete bridge railing reminiscent of the original one was included in the new bridge, See Figures 8 & 9. The new railing had to meet minimum requirements for crash testing as well as have the features that were present on the original railing. The new railing is a Texas Classic arched railing (FHWA, 2005). To ensure continuity this railing had to exist not only on the approach roadways, but it also had to be utilized on the movable portion of the bridge.

Two types of pier arrangement for the approach spans were also investigated. The first used precast square concrete piles similar to the existing bridge. The second alternate was to use drilled shafts. The drilled shafts alternate was selected.

2.3 *Unique structure characteristics*

2.3.1 *Bascule deck*

The deck of the bridge utilizes an 8 inch reinforced lightweight concrete deck. This was selected for its improved ride-ability, improved durability as well as its better composite action than other deck types. By using this type deck, the designers were also able to eliminate the lateral bracing of the movable span.

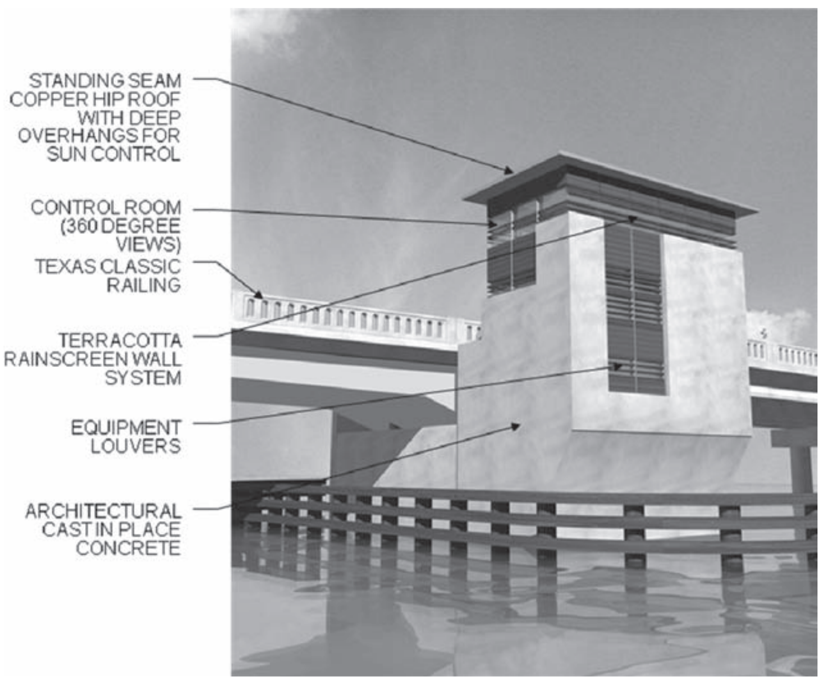

Figure 6. Details of proposed bridge control house (HNTB, 2007).

Figure 7. Completed control house.

Figure 8. Original bridge railing.

The roadway joint for the separation of the movable span to the fixed span is located at the rear of the bascule span. By doing this it reduces the complex details in the roadway that this joint generally produces as well as provides better protection of the machinery and trunnions below. By introducing the joint to the rear of the bascule span, it also eliminates the need for a flanking span or span specifically required to interface with the bascule counterweight. This further simplifies the details and cost associated with the structure. This also provides simpler details for the transition to the approach spans by having the approach span terminate onto the rear bascule pier wall. Since vehicular traffic rides on the counterweight, the use of tail locks was introduced to eliminate the potential of having vehicular traffic live load opening the bridge.

The unsymmetrical roadway cross section always causes concern for the engineer regarding design loads, but with a movable bridge it also makes for a difficult balancing condition and a more complex counterweight with different areas of unit weights for the concrete. In an effort

Figure 9. New bridge railing.

to offset the magnitude of transverse imbalance, the designers used lightweight concrete for the railing and the sidewalk on the west side while using normal weight concrete on the east side railing.

2.3.2 *Bascule piers*

The bascule piers are a four walled box configuration. This was done to allow the structure to be closed for protection from the elements as well as pigeons. It also allowed the counterweight to be able to dip lower than the water level when the span is open. As previously mentioned, the appearance of the structure along with its size and mass were important aesthetic considerations. Trying to minimize the size of these piers was important in keeping the proportioning of the substructure and its relation to the superstructure. The proportioning of the piers to the rest of the structure had to be considered so that they were not out of context. To maintain a clean appearance of the bascule piers, minimal treatment was done to the pier faces. This allowed the other aspects of the structure to be accented.

The sidewalk is located on the west side of the bridge. Since the bridge is adjacent to a park and it was desired to integrate the bridge into the surrounding area, the opposite pier from the control house has an overlook. This overlook provides pedestrians a point to stop and observe the city, the Union Point Park, the water and nearby marinas. This added a feature to the bridge that makes it a destination to visit and observe the surrounding areas. This overlook also allowed the south pier to have similar proportioning to the north bascule pier. This overlook also served to provide access to the inside of the south pier.

An added convenience for maintenance personnel and the bridge operators are two parking areas, one behind each bascule pier where workers can safely park their vehicles close to the movable portion of the bridge and not reduce the lane width or close a lane. This had been provided on the previous bridge and was requested on this structure (See Figure 10).

2.3.3 *Structure proportions*

The movable portion of this double leaf bascule bridge has a long and slender arrangement. To keep this proportioning as well as maximize the vertical clearance in the closed position, welded plate steel for the bascule girders was done. This allowed the designer the ability to detail the girder to the requirements of the design and optimize the geometry and material. As previously mentioned, the bascule piers were meant to be minimized for proportioning, but also for cost. The size of the back (or rear) portion of the bascule span determined not only the length of the pier longitudinally, but also its depth. This is due to the rotation of the span during an opening. Since the deck, sidewalk and railings on the bascule span were

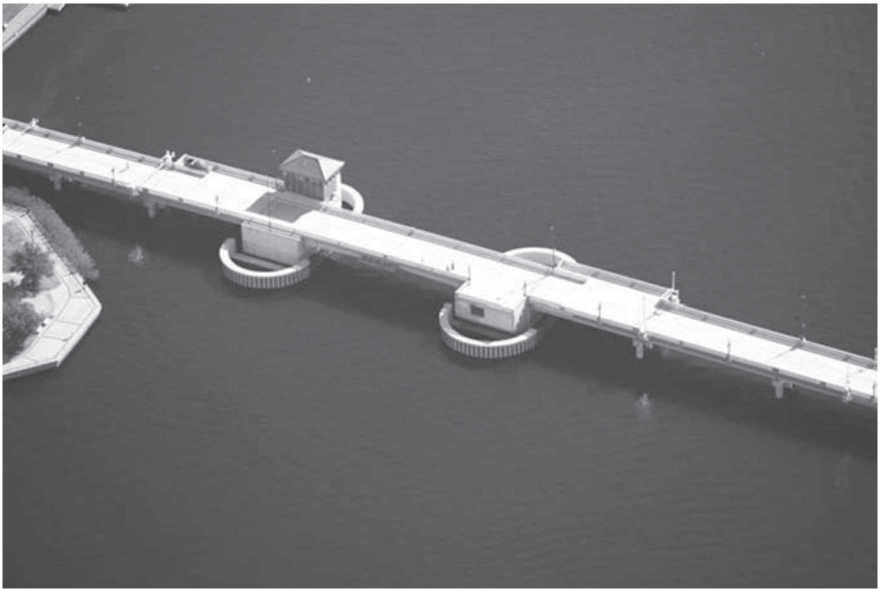

Figure 10. Aerial view showing outlook and two parking areas.

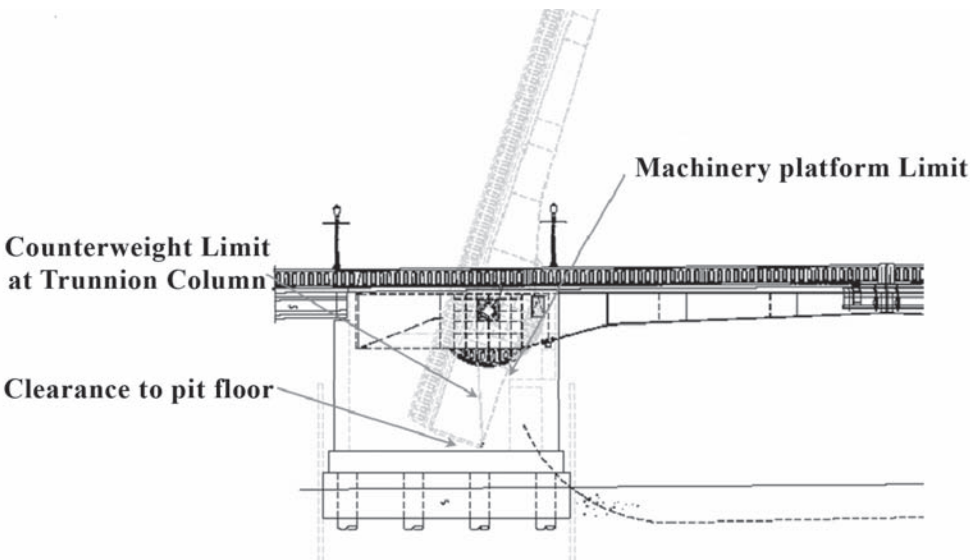

Figure 11. Tight clearances in bascule pier.

all made of concrete, wherever possible elsewhere, weight optimization was performed. The use of a practical, but heavy unit weight concrete for the counterweight was done to try and minimize the volume of concrete and the rear end of the bascule span. Coordination of the dimensions and details between the fixed and movable portion of the bridge to ensure sufficient clearances was required. This included the base of the bascule pier, the trunnion columns as wells as the platform that supports the operating machinery, See Figure 11. A view of the finished structure is shown in Figure 12.

Figure 12. Finished structure.

3 CONCLUSIONS

Replacing an older bridge with a new structure in an historic district of a town can be a daunting task. It takes the right people who are, suitably prepared, ready to listen, understand, explain and be able to incorporate agency, historian and public concerns into the project. Being able to conceptualize, set proportions, select specific details, overcome unique challenges and explain the concept are critical for all parties to understand and reach consensus for the structure to be built successfully. Understanding the importance of each component and how it can affect the outcome is critical when designing any new movable structure. This paper only highlights a small portion of the effort, discussions, explanations, understanding, coordination and agreements necessary to produce a unique aesthetically pleasing, context sensitive structure situated in a historic city.

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Chapter 26

Site and sight: Examining the forms of Hudson River Bridges

P. Draper
Manhattan College, Bronx, New York, NY, USA

E. Segal
Simpson Gumpertz & Heger, New York, NY, USA

A. O'Connor
Manhattan College, Bronx, New York, NY, USA

ABSTRACT: The Hudson River is home to a variety of bridge types that demonstrate the development of structural engineering. This project catalogues many of the bridges crossing the Hudson River to determine what they can teach us about the history of structural engineering, structural forms for bridges, and the relationships bridges have to their environments and to the other bridges nearby. An area of particular focus is the Tappan Zee Bridge and its planned replacement. Looking at the form of bridges in relation to their site also proves to have educational benefits by giving students a sense of the social and symbolic contexts of structures, in addition to technical aspects.

1 INTRODUCTION

The Hudson River is one of the most historically rich rivers in the country and defines one of the most important regions of development in the nation's history. It is also home to a variety of bridge types that reflect the development of structural engineering. For this project we catalogued many of the existing bridges crossing the Hudson River to determine what they can teach us about the history of structural engineering, structural forms for bridges, and the relationships bridges have to their environments and to the other bridges nearby.

A recent book describes the history of many of the bridges on the Hudson (Wolf 2010). This study is not intended to cover territory already described in that work or in others (Steinman and Watson 1957, Petroski 1995, Rastorfer 2000). Instead we aim to show how the forms of the Hudson River Bridges vary with respect to geography, chronology, span, and site. Form selection seen through these lenses suggests that how a bridge relates to its specific environment is not always a significant consideration.

An area of particular focus is the Tappan Zee Bridge. This bridge and the plan for its replacement have been in the news a great deal lately. But nearly all of the public and political discussion has focused on the cost of replacing the bridge and not on how the bridge might make a positive addition to its surroundings. A consideration of the relationship between the location of the replacement bridge to its form may contribute to a new look at how we should go about replacing the bridge. Notable academics and bridge designers have identified the relationship of a bridge's form to its environment as one important criterion by which we can critique bridge aesthetics. By describing the opinions of these experts we intend to show the significance of this aspect of a bridge's design. To further demonstrate the influence of a bridge's integration into its environment on its overall aesthetics, it is necessary to compare two bridges in which everything but the environment is held constant. A case study comparison of two nearly identical bridges by Robert Maillart reveals this influence and provides the means to extrapolate to future comparisons between Hudson River Bridges where more than the environment varies.

Looking at the form of bridges in relation to their locations proves to have educational benefits, as well. A catalogue of local bridges provides examples for students to better understand how structures are designed, analyzed, constructed, and maintained. Visits to built works can be used to augment traditional methods of teaching structural design and analysis. In this way we can also give students a sense of the social and symbolic contexts of structures, in addition to the technical.

2 THE HUDSON RIVER BRIDGES

2.1 *Geographic progression*

The Hudson River flows for more than three hundred miles along eastern New York from New York City up to the Adirondacks. Here we have focused on the major portion of the Hudson from New York City to where the Mohawk River feeds into it north of Troy, as this defines the main area of industrial growth along the Hudson.

Table 1 shows the geographic order of bridges crossing the Hudson. We see a general variety of types of forms, with nearly all major bridge types represented save for arches and cable stayed. The bridges also include a variety of spans, from smaller spans on the order of 60 m for the bridges in Troy, where the Hudson is narrower, to the 1,100 m span of the George Washington Bridge.

2.2 *Chronological progression*

Table 2 shows the chronological progression of bridges on the Hudson. Many bridges on the Hudson have been destroyed or replaced over the years. This table shows only existing bridges.

Table 1. Geographic order of bridges crossing the Hudson River.

		Bridge	Year	Type
1	N	Troy-Waterford	1909	Truss
2		112th Street	1996	Beam
3		Collar City	1980	Beam
4		Green Island	1981	Vertical lift
5		Congress street	1971	Beam
6		Troy-Menands	1933	Truss
7		Patroon Island	1968	Truss (deck)
8		Livingston Avenue	1909	Truss (swing)
9		Dunn Memorial	1969	Beam
10		Castleton	1959	Truss (cantilever)
11		Alfred H. Smith Memorial	1924	Truss
12		Rip Van Winkle	1935	Truss (cantilever)
13		Kingston-Rhinecliff	1957	Truss (continuous)
14		Poughkeepsie Railroad/ Walkway Over the Hudson	1888	Truss (cantilever)
15		Mid-Hudson	1930	Suspension
16		Newburgh-Beacon #1	1963	Truss
17		Newburgh-Beacon #2	1980	Truss
18		Bear Mountain	1924	Suspension
19		Tappan Zee	1955	Truss (cantilever)
20	S	George Washington	1931	Suspension

Table 2. Chronological order of existing bridges crossing the Hudson River.

	Bridge	Year	Type
1	Poughkeepsie Railroad/ Walkway Over the Hudson	1888	Truss (cantilever)
2	Troy-Waterford	1909	Truss
3	Livingston Avenue	1909	Truss (swing)
4	Alfred H. Smith Memorial	1924	Truss
5	Bear Mountain	1924	Suspension
6	Mid-Hudson	1930	Suspension
7	George Washington	1931	Suspension
8	Troy-Menands	1933	Truss
9	Rip Van Winkle	1935	Truss (cantilever)
10	Tappan Zee	1955	Truss (cantilever)
11	Kingston-Rhinecliff	1957	Truss (continuous)
12	Castleton	1959	Truss (cantilever)
13	Newburgh-Beacon #1	1963	Truss
14	Patroon Island	1968	Truss (deck)
15	Dunn Memorial	1969	Beam
16	Congress Street	1971	Beam
17	Collar City	1980	Beam
18	Newburgh-Beacon #2	1980	Truss
19	Green Island	1981	Vertical lift
20	112th Street	1996	Beam

Here we see a correlation between the era of the bridge and its form. The earliest bridges up until the 1920s were trusses, as we might expect in accordance with the historical development of bridge engineering in this country. The 1920s saw the construction of three suspension bridges, followed by a return to trusses. The late 1960s show a move toward beam bridges.

The eighteenth and nineteenth bridges in the table each represent special cases. The Newburgh-Beacon Bridge #2 of 1980 was designed as a twin to the Newburgh-Beacon Bridge #1 of 1963 (Wolf 2010). And the Green Island Bridge is a vertical lift bridge for its original railway loading, but is also of the beam type.

Thus we see in essence four eras of bridge types for the crossings of the Hudson: truss, suspension, truss again, and beam. This suggests a tendency to follow historical trends when selecting a bridge type.

2.3 *Span length*

The relationship between the form of the bridge and its span length (as well as construction considerations) has been well documented (e.g., Taly 1998). Table 3 shows the type and span length of selected bridges on the Hudson. The information for some of the other bridges has not yet been confirmed, but many of the beam bridges are of visibly shorter span and length (i.e. narrower sections of the Hudson). Span length along with the cost of design and construction plays a large role in the choice of an appropriate type of bridge (Troitsky 1994).

2.4 *Site*

The Hudson River provides a diverse group of settings for its crossings. The river defines one of the most important regions in the historical and economic development of the state and nation and the natural beauty both of the river and the surrounding valley has been widely celebrated in literature and the visual arts.

Table 3. Type and span length of selected Hudson River bridges (Janberg 2011).

	Bridge	Year	Type	Span length, m
1	Troy-Waterford	1909	Truss	59
2	Poughkeepsie Railroad/ Walkway Over the Hudson	1888	Truss (cantilever)	167
3	Rip Van Winkle	1935	Truss (cantilever)	244
4	Kingston-Rhinecliff	1957	Truss (continuous)	244
5	Newburgh-Beacon #1	1963	Truss	305
6	Newburgh-Beacon #2	1980	Truss	305
7	Tappan Zee	1955	Truss (cantilever)	370
8	Mid-Hudson	1930	Suspension	457
9	Bear Mountain	1924	Suspension	498
10	George Washington	1931	Suspension	1067

Table 4. Approximate combined population of localities connected by Hudson River bridges (Wolfram 2011).

	Bridge	Year	Type	Population of localities
1	Rip Van Winkle	1935	Truss (cantilever)	8,000
2	Tappan Zee	1955	Truss (cantilever)	10,000
3	Castleton	1959	Truss (cantilever)	10,000
4	Kingston-Rhinecliff	1957	Truss (continuous)	20,000
5	Poughkeepsie Railroad/ Walkway Over the Hudson	1888	Truss (cantilever)	30,000
6	Mid-Hudson	1930	Suspension	40,000
7	Newburgh-Beacon #1	1963	Truss	40,000
8	Newburgh-Beacon #2	1980	Truss	40,000
9	Collar City	1980	Beam	50,000
10	Bear Mountain	1924	Suspension	50,000
11	Troy-Waterford	1909	Truss	50,000
12	Alfred H. Smith	1924	Truss	50,000
13	Troy-Menands	1933	Truss	50,000
14	Green Island	1981	Vertical lift	50,000
15	Congress Street	1971	Beam	60,000
16	112th Street	1996	Beam	60,000
17	Dunn Memorial	1969	Beam	100,000
18	Patroon Island	1968	Truss (deck)	100,000
19	Livingtson Avenue	1909	Truss (swing)	100,000
20	George Washington	1931	Suspension	8,000,000

An engineer might therefore seek to design a distinctive bridge that either complements an urban environment such as Albany or New York City or is in harmony with a grand rural setting.

Table 4 shows the type of bridge in relation to the approximate combined population of the two communities each connects. This is somewhat open to interpretation, as many of the bridges connect smaller towns that are considered parts of either the New York City metropolitan area (population of 19,000,000) or Albany metro area (850,000). But it nevertheless seems to suggest that for most areas, with multiple bridge types across varying populations, that the size of the localities to be connected does not heavily influence the choice of form.

One possible exception to this is the George Washington Bridge, in New York City. Othmar Ammann, the designer of the bridge, certainly chose a suspension design as one of the few effective options based on the massive span of the structure. But he also expressed an interest in the relationship between the form and the aesthetics of the bridge and designed the bridge

Figure 1. Tappan Zee Bridge.

with an eye towards attractiveness (Billington 1977). He could have chosen the form to have his bridge complement not only its urban location, but also the other existing suspension bridges of New York City.

3 THE TAPPAN ZEE BRIDGE

In looking at the collected bridges of the Hudson River, the Tappan Zee Bridge presents an interesting individual case study (Figure 1). It is the first bridge north of New York City and connects two historic towns, Nyack and Tarrytown, considered part of the New York City metropolitan area. Despite its location and heavy use, it is not a celebrated bridge like the George Washington. In his book on the history of bridge design, the bridge designer David Steinman (with co-author Sara Watson) stated that engineers of the New York State Thruway Authority referred to it as "one of the ugliest bridges in the East" (Steinman & Watson 1957).

Its current state of decay has led engineers and political leaders to decide to replace it. The appealing location, from both standpoints of natural environment and built environment, suggests an opportunity to replace the Tappan Zee with a more attractive form. As recently as 2009, an article optimistically stated that for the replacement, "transportation officials decided to think big" with plans for a bridge that would also carry rail lines (Kocieniewski 2009). Since that time economic concerns have gradually eroded the wish list to the point that the state is even considering privatization of the project just to get something built (Grossman 2011). Lately almost all of the public discussion of the replacement has focused on the cost of the replacement bridge instead of how it might (or might not) complement its location or the other bridges of the Hudson.

4 RELATIONSHIP OF BRIDGE AESTHETICS TO SITE

The importance of engineers designing bridges with an aesthetic intent, including how a bridge relates to its location, has been explained in a number of sources. Elizabeth Mock suggested that bridges be subjected to the same standards of architectural evaluation as buildings (1949). David Billington instead describes structural engineering as a separate creative discipline from architecture and explains that the best works of structural engineering, or structural art, are those that fully integrate efficiency, economy, and elegance (1983).

Frederick Gottemoeller outlines specific prescriptive factors for bridge aesthetics, and includes suggestions on how engineers might go about improving their aesthetic design skills for all bridge designs (1998).

Others have suggested that the aesthetic qualities of bridges may be considered optional depending on cost. The Swiss bridge designer Christian Menn has described the relationship of aesthetics and economy, with aesthetics the center goal on a target for designers that also includes increasing rings of economy, serviceability, and, ultimately, the largest ring of safety (Billington 2003).

The German bridge engineer, Fritz Leonhardt, issued a series of "Guidelines for Aesthetic Structures," that includes their "Integration into the Environment" (Leonhardt 1996). Similarly, Menn in his article, "Aesthetics in Bridge Design," states that both "integration of the bridge into its surroundings" and "design of the bridge as a structure itself" are fundamental to creating an aesthetically pleasing design. In Menn's opinion, careful consideration of road lines, topography, and proportion of a structure and its components relative to its environment is necessary to create an elegant design (Menn 1985).

Comparing Robert Maillart's bridges in Salginatobel (1930) and Rossgraben (1932) illustrates the influence of site on aesthetics (Figures 2 and 3). Both are single-lane, three-hinged, hollow box arches with simple cross walls and maximum depth at their quarterpoints. Rossgraben has a slightly greater span-to-rise ratio (8.6 vs. 6.9), but otherwise, the "design of the bridge as a structure itself" is the same. While a clear evolution of form is evident when comparing Maillart's earliest to final three-hinged hollow arches, many of his intermediate bridges are similar. Jörg Schlaich in his article, "The Bridges of Robert Maillart," states:

> There is an uneasy feeling about one's inability to understand Maillart's much praised individualistic topographic relationship in view of the fact that his three-hinged arches differ from each other only marginally, contrary to their particular environments. Is the Salginatobel Bridge's major asset its environment, as opposed to the lesser known—though almost equal and cleaner—Rossgraben Bridge? Must Maillart be confronted with his own holistic postulate? (Schlaich 1993)

Schlaich suggests that Maillart did not always conceive of a bridge form specifically for a site, but rather used the same form repeatedly without considering its surroundings. While neither the Salgtinatobel nor the Rossgraben is out of place in its environment, the former does benefit from its dramatic site. At the Salginatobel Bridge the ravine acts structurally as the abutments. This functional utilization of the site results in a better visual integration of the form into the environment. Consequently, this bridge has been recognized as a World Monument by The American Society of Civil Engineers and is arguably Maillart's best known work.

Maillart's bridges demonstrate how at times, even aesthetically sensitive engineers rely on "standard" forms and in doing so do not take complete advantage of a particular site.

Figure 2. Salginatobel Bridge.

Figure 3. Rossgraben Bridge.

Standard truss and beam bridges are used at a greater frequency across an even larger array of sites with less consideration for their surroundings.

5 CONCLUSIONS

The Hudson River is home to a variety of bridge types, with many of the major forms (save for arch and cable stayed) spread along the length of the river. Yet aside from the George Washington, few are celebrated as great works of structure. In tabulating the dates and spans of the collected bridges of the Hudson, the data seem to suggest that as engineers we tend to choose a type of bridge based primarily on factors like span length, the predominant form of that time, and cost more so than how the form of the bridge might complement its environment or the other bridges nearby.

The Tappan Zee presents an interesting case study, for its past, present, and future. It was designed as one of several indistinct large truss bridges in the central portion of the Hudson. Now considered past its functional lifespan, it has been deemed necessary for replacement. There has been a fair amount of debate over the requirements of the replacement bridge and certainly about its cost, but there has been little public discussion about how the appearance of the replacement bridge will contribute to (or detract from) its location. This perhaps reflects the implicit perception of aesthetic qualities as add-ons to a bridge, an unnecessary luxury that cannot be afforded during times of privation. Costly recent bridges such as the Turtle Bay Sundial Bridge in Redding, California, and a replacement span for the San Francisco-Oakland Bay Bridge suggest that we as engineers have not done enough to disprove this assumption. Yet historical examples show us that this need not be the case. Othmar Ammann's design of the George Washington Bridge, for instance, was selected not just for its dramatic form, but also for its economical design.

In "On the Conceptual Design of Structures—An Introduction," Jörg Schlaich described his proposal to sue the German government or national railroad for dotting the countryside with unsightly bridges (Schlaich 1996). A colleague dissuaded him on the basis that the public has little opinion on the matter. Schlaich is passionate about bridge aesthetics, but the layperson may not be. This, Schlaich is quick to point out, is a failure on the part of the profession. The current literature on aesthetics and exemplary structures is not sufficient. We need to improve how we educate the public. The University is a logical place to begin. By introducing both engineering and non-engineering students to the complex social and symbolic issues that accompany the technical challenges of structural design and construction, the profession can create ambassadors to the greater public.

The act of visiting an elegant bridge or one that complements its surroundings extends beyond the classroom. While photographs and small-scale models can give a sense of how a structure relates to its environment, viewing a full-scale bridge elicits a visceral response. One immediately has a sense of the social and symbolic aspects of the structure in addition to the technical. Visits to visually interesting structures provide students with a broader understanding of structural design, analysis, construction, and context.

The Hudson River region has a rich variety of natural and built environments that is ripe for these visits. Further study is required to understand how effectively the two environments in the Hudson River region interact. The sentiments of the Swiss shell builder, Heinz Isler, illustrate the internal conflict that all members of the construction industry face: "At first I was against building because I had seen how it could destroy nature. I was of two minds: on the one hand, I didn't want to be part of this destruction; on the other hand, I was fascinated by doing things very elegantly and very lightly" (Isler 2008). Ultimately, the natural and built environments need to coexist. The best bridges are those that while efficient and economical are also visually interesting both on their own and in the context of the surrounding structures and landscape.

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Chapter 27

The St. Louis Bridge, the Brooklyn Bridge, and the feud between Eads and Roebling

K. Gandhi
Gandhi Engineering, Inc., New York, NY, USA

ABSTRACT: The St. Louis Bridge, built by Captain James Eads in 1874, and the Brooklyn Bridge, completed by Colonel Washington Roebling in 1883, shared innovative techniques and established precedents for designing long-span bridges in the years ahead. Despite the St. Louis Bridge being a steel arch bridge which differed from the wire suspension construction of the Brooklyn Bridge, both of these structures were considered the largest bridge of the time, and continue to hold historic significance. Despite the achievements of both Eads and Roebling, their individual successes were overshadowed by a dispute between the two men. The root of this disagreement was the design of caissons used for the foundation, as each engineer claimed to be the first to design the caisson for his bridge foundation. This paper looks at the St. Louis Bridge, the Brooklyn Bridge, and the dispute which came to define these two giants of Civil Engineering.

1 INTRODUCTION

The St. Louis Bridge, built by Captain James Eads in 1874, and the Brooklyn Bridge, completed by Colonel Washington Roebling in 1883, shared innovative techniques and established precedents for designing long-span bridges in the years ahead. Both used methods and designs that challenged engineering preconceptions; pneumatic caissons were used to reach the riverbed to secure the bridge foundation and daring long-span designs defied previous building standards.

2 THE ST. LOUIS BRIDGE

The idea of a suspension bridge across the Mississippi River in St. Louis was first proposed by Charles Ellet in 1839 to the Mayor of St. Louis. Ellet's bridge had a main span of 1200 feet, side spans of 900 feet, and a deck 27 feet wide with a 19 foot roadway and two 4 foot wide sidewalks for a sum less than $600,000. His proposal was accepted, but the Mayor and City Council refused to proceed with the extravagantly wild and unsafe scheme (Steinman 1957). A second attempt to build a bridge in St. Louis failed in 1855 when the supporters of the bridge did not raise the estimated $1.5 million needed to build the bridge to carry railways and pedestrians.

John A. Roebling, after completing his railway suspension bridge with a clear 800 foot span over the Niagara Gorge in 1855, proposed a suspension bridge in 1866. His scheme (Figure 1) was made public by his son, Washington, in 1869 in a book "Long Span Railway Bridges" written by his father and published posthumously (Roebling 1869). Roebling's proposal was not accepted possibly due to concerns about the safety of suspension bridges.

The history of the St. Louis Bridge, as given by Colonel W. Milnor Roberts, involved two rival companies that were each formed to build the St. Louis Bridge (Roberts 1872–3).

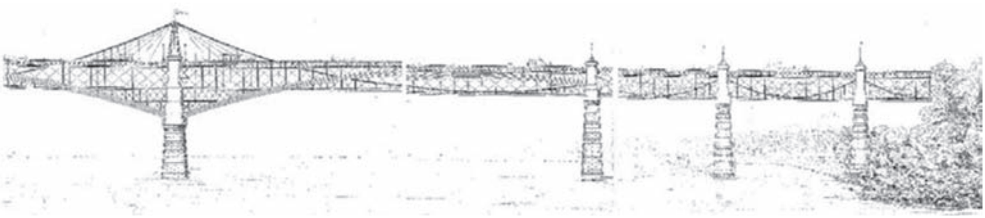

Figure 1. Roebling's proposal for building a suspension bridge in St. Louis (Roebling 1869).

Figure 2. The St. Louis Bridge built by James Eads (Williams 1877).

One company was created in 1864 in St. Louis and the other in Illinois in 1867. In March 1868 an act was passed to consolidate the two companies, and in July 1868, the act of consolidation was recognized and sanctioned by the United States Congress. All the powers of the two companies were vested in a new company, the Illinois and St. Louis Bridge Company.

The act of the U.S. Congress recognizing the first company in 1866, before the consolidation of the two companies, stipulated that the bridge could not be a suspension bridge or a draw bridge, but must be constructed with continuous or unbroken spans, and subject to three additional conditions. One of these conditions was that the bridge must have at least one span 500 feet in clear or two spans of 350 feet with a minimum vertical clearance of 50 feet above the highest tide elevation recorded on the Mississippi River at St. Louis over the navigable channels.

There were two competing groups interested in building the St. Louis Bridge. One group, backed by 27 engineers, recommended two equal spans of 368 feet, preferring it over the three spans of 500 feet designed and proposed by Eads. The group of engineers had serious doubts about the feasibility of building a 500 foot arch because no one had built an arch using wrought iron, and no one had considered using steel which was very expensive in comparison to iron at this time. In May of 1868, Eads answered satisfactorily and conclusively to all of the objections raised by the first group and prevailed. In the final design, Eads selected 520 feet in length for the center span and 502 feet for the side spans. The bridge built by Eads is shown below in Figure 2.

3 THE BROOKLYN BRIDGE

The first notion of a bridge across the East River in New York City was published by Thomas Pope, an architect and landscape gardener, in 1811. He called his patented bridge the "Flying Pendent Lever Bridge." He built a model approximately 50 feet long on a scale of 3/8th of an inch to one foot to "illustrate a bridge suitable to span the East River at New York"

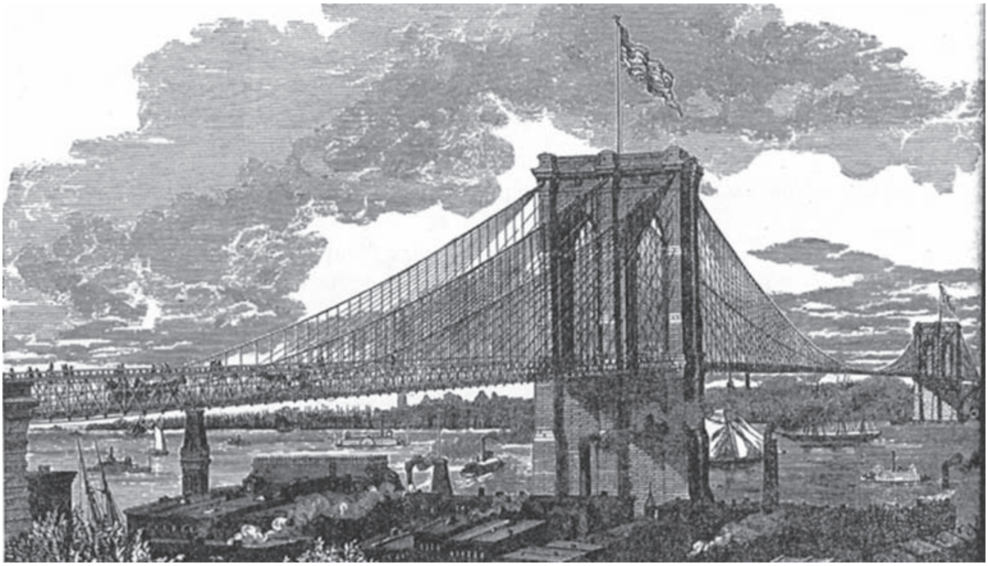

Figure 3. The Brooklyn Bridge (Scientific American 1877).

(Pope 1811). The length of the single span was 1800 feet and the height of the abutment was 223 feet. No one attempted to build the bridge proposed by Pope.

After John Roebling successfully completed the Niagara railway suspension bridge with a clear span of 800 feet, he began contemplating a bigger assignment. He wrote a final report to the owners of the bridge—without mentioning the East River or Brooklyn Bridge—and stated that the future may hold "railway bridges suspended of a 2000 foot span, which will admit of the passage of trains at the highest speed" (Roebling 1855).

John Roebling was appointed Chief Engineer of the New York Bridge Company on May 23, 1867 and submitted his first report on the proposed East River Bridge to the Bridge Company on September 1, 1867 (Roebling 1867).

In 1869 a board of consulting engineers was appointed to review the plans to build the Brooklyn Bridge proposed by John Roebling. Its members were all well-known engineers of the period including Horatio Allen, W.J. McAlpine, J. Dutton Steel, Benjamin H. Latrobe, John Serrel, J.P. Kirkwood, and J.W. Adams. They unanimously agreed that "there is no insurmountable obstacle to building a suspension bridge of a 1,600 span and even much greater" (Engineering 1869a).

Work started on the Brooklyn Bridge in 1869 and it was opened by President Chester A. Arthur on May 24, 1883. The bridge completed by Roebling is shown in Figure 3.

4 CAPTAIN JAMES EADS

James Eads, the builder of the St. Louis Bridge, was born in Lawrenceburg, Indiana on May 23, 1820. At age 13, his family moved to St. Louis, and from then on he supported himself by first selling apples on the street, working in a store, and then, beginning in 1839, as a clerk on a Mississippi steamboat. He had very little formal education, but had a keen interest in all things mechanical. His knowledge was entirely gained by self-study and practice (Engineering News 1887). On this steamboat, he learned that after five years the submerged cargo of a sunken ship belongs to the retriever; and with this information, Eads discovered the lucrative profession of raising sunken riverboats (Jackson 2001).

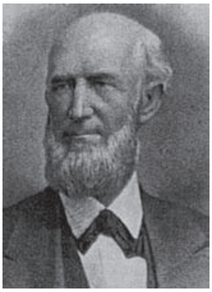

Figure 4. Captain James Eads (Woodward 1881).

In 1842 he formed a company to raise sunken riverboats. As the business expanded, Eads acquired a fleet of boats and was given the title "captain" which followed him for the rest of his life. He used to say that there was not a stretch of 50 miles from St. Louis to New Orleans, in which he had not stood on the bottom of the river under his diving bell (Railroad Gazette 1887).

During the Civil War, Eads was called upon by President Lincoln to share his knowledge of the Union Army against Confederate attacks on the Mississippi river. This made him a friend of General Ulysses S. Grant who later became President of the United States.

In 1867, Eads was engaged in the planning of the St. Louis Bridge. His design of a three-span arch bridge was chosen over a truss bridge proposed by a rival group. Despite a lack of experience in bridge building, his creative genius, background in raising capital for businesses, and his elegant bridge design won the favor of influential supporters.

To calm the fears of skeptics, Eads (Figure 4) appointed a staff of experienced assistants with impressive experience in building bridges. Three of these engineers included, Col. Henry Flad, Mr. Charles Pfeifer, and Col. W. Milnor Roberts (Eads 1871). Colonel Roberts was selected to take charge of the project in the absence of Eads, as well as provide technical and administrative support. Pfeifer analyzed the various bridge components and calculated stresses. Colonel Flad, besides being on the front line of construction, was responsible for the cantilever erection scheme for the arches. There were two other engineers, Walter Katte and Theodore Cooper, who also became prominent in the bridge engineering field. Katte was the construction engineer for the bridge erector, Keystone Bridge Co., and Cooper was the inspector who made sure that all materials and workmanship were of the highest quality and complied with the construction specifications.

In the summer of 1868, Eads was ordered by his physician to take a trip to Europe to recover from his failing health. During his trip he took every opportunity to visit important bridge construction sites with deep foundations. He returned from Europe fully recovered on May 1, 1869. In his report to the Board of Directors, in September 1869, Eads discussed witnessing the sinking of masonry piers by the "plenum pneumatic process," and claimed that he could improve this process and sink the piers of the St. Louis Bridge safely, expeditiously, and more economically than the method he had originally proposed (Eads 1868).

5 COLONEL WASHINGTON A. ROEBLING

Washington Roebling's upbringing and education contrasted significantly from that of James Eads. Unlike Eads, Roebling had a formal education including prep school at the Trenton Academy and a degree in civil engineering from Rensselaer Polytechnic Institute (RPI) in Troy, NY. His thesis at RPI was "Design for a Suspension Aqueduct" to carry the Poestenkill Creek in Troy, NY (Roebling 1857). The weight and quantity estimates of the bridge components were so detailed that they leave no doubt to the reader's mind about Roebling's familiarity with the construction of a suspension bridge. After finishing college in 1857,

Figure 5. Washington A. Roebling (Schuyler 1931).

Roebling worked for his father, John Roebling, at his mill making wire ropes (Schuyler 1931). This experience at his father's company allowed Washington Roebling to become familiar with steel ropes which—one day—would be responsible for suspending the Brooklyn Bridge above the East River.

Roebling's next major career move was in 1861 when he enlisted in the Union Army. He intended to design suspension bridges for the army, but largely spent his time writing a publication on military suspension bridges (Mulholland 2006). When he retired from the army as a Colonel in 1865, Washington Roebling married Emily, his commander's sister, whom he had met while she was visiting her brother at the army camp (Sayenga 2009). Shortly after his marriage, Roebling began to work with his father in Cincinnati to construct a suspension bridge over the Ohio River to connect Cincinnati and Covington, Kentucky (Schuyler 1931).

After finishing the Cincinnati and Covington Bridge, Washington Roebling, accompanied by his wife, went to Europe to study pneumatic caissons being used to set bridge foundations (Sayenga 2009). In 1868 he returned brimming with new ideas after seeing a French engineer successfully use pneumatic caissons to sink the piers of a bridge over the Allier River at Vichy. After the time spent studying caissons in Europe, Washington Roebling began to work with his father, John Roebling, who had recently begun the Brooklyn Bridge project and participated in the alignment surveys. In 1869, shortly after Washington Roebling joined his father, John Roebling sustained a painful injury to his foot. The injury became infected and, less than a month after his injury, John Roebling died in June 1869 from tetanus (Mulholland 2006). With his father gone, Washington Roebling, at 32 years old, was appointed by the Board of Directors to be Chief Engineer in charge of building the Brooklyn Bridge in August 1869 (McCullough 1983). *Engineering* (1869c) observed that although the inhabitants of Brooklyn and New York City "sustained a great loss in the untimely death of Mr. John A. Roebling, they are fortunate in that he has left behind him a son possessing the genius of his father, as well as the benefits of his great experience". Similar to the talented engineers working with Eads, Roebling (Figure 5) also had well-respected assistant engineers on his team. Notable engineers included Charles C. Martin, Francis Collingwood, Samuel Probasco, Col. William H. Paine, and George W. McNulty.

Given the stark differences between the upbringing and career paths of Eads and Roebling, it is easy to imagine that a disagreement between the two men could be partially professional, as well as tinged with a sense of superiority on the part of each man. Eads was likely to consider himself more experienced in understanding the nature of working with rivers, whereas Roebling likely felt superior in understanding the science behind bridge building.

6 SIMILARITIES BETWEEN THE BRIDGES AND THEIR BUILDERS

With the invention of the Bessemer Process in Europe, the cost of steel had decreased considerably and it was possible to build bridges on a grander scale with structural capabilities previously unimagined. In the past steel was unaffordable as a construction material notwithstanding its superior strength compared to wrought iron. Both Eads and Roebling used steel for their bridges. Eads used steel for the tubular arch members which constituted a small percent of the total weight of the bridge. Roebling constructed a large part of the Brooklyn Bridge from steel and the cables are exclusively made of steel.

Both Eads and Roebling spent a great deal of time and effort proportioning the key elements of their bridges to make them aesthetically pleasing; Eads in using tubular members and determining the relationship between the length of the center spans (520 feet) and the end spans (502 feet); and Roebling in his towers.

They both also devoted considerable thought and energy to protecting the most vital bridge components: Eads by using six enclosed staves to protect the tubular arch against the atmospheric influence and Roebling by covering the galvanized wires of the compacted cables inside a continuous wire wrapping (Gayler 1909).

Both Eads and Roebling instituted stringent quality control programs. Colonel Henry Flad, Chief Assistant Engineer to Eads, developed a testing machine which was capable of detecting a change in the length of a specimen equal to two hundred thousandth of an inch (Eads 1870). The same machine was calibrated to apply a weight up to 100 tons. Specimens of steel, iron, woods of various kinds, granite, brick limestone, concrete, cement, models of tubes, trusses, etcetera were tested in trial runs. Eads had defined stress limits for compression, tension, and yield in pounds per square inch, and modulus of elasticity between 26,000 ksi and 30,000 ksi. The specifications clearly stated that "failure to stand any one of such tests will be sufficient cause for the rejection of the piece" (Eads 1871).

Roebling developed similar specifications for his steel cable wire (Roebling 1876). When bids (or tenders) were invited for the supply of 3,400 tons of steel wire to form the main cables of the Brooklyn Bridge, the bidders were required to submit samples of their wires, and these wires were tested by Roebling's two assistant engineers, Charles C. Martin and Colonel William H. Paine (Engineering 1877).

Despite the St. Louis Bridge being a steel arch bridge which differed from the wire suspension construction of the Brooklyn Bridge, both of these structures were considered to be a crowning achievement. In addition, the two bridges shared the objective of supporting train traffic, as well as pedestrians. The Brooklyn Bridge continued to transport elevated trains and streetcars until 1948 when the rail lines were removed in favor of additional road space for cars. This reconstruction work was completed in 1953. The St. Louis Bridge was modernized in 2003 in order to support light rail cars and increased vehicular traffic on the bridge.

Although the St. Louis Bridge and the Brooklyn Bridge were engineering successes of historic measures, financially they followed different paths. The St. Louis Bridge had problems meeting its obligations to note and bond holders as early as January 1876; it was sold on December 20, 1878 under decrees of foreclosure on the first and second mortgage (Railroad Gazette 1876, 1878). The Brooklyn Bridge, on the contrary, was a great success due to income generated by daily commuters traveling between Brooklyn and New York.

In 1872 Eads and Roebling, two well-known and highly respected civil engineers, became associated with a well-publicized quarrel. The root of this disagreement was the design of the caissons used in the construction process, as each engineer claimed to be the first person to have designed the caisson structure used to lay the bridge foundation. The argument between Eads and Roebling, a back-and-forth series of articles written by the engineers and published in journals, survives today given the highly public forum in which this dispute occurred.

7 EADS' DESIGN OF PNEUMATIC CAISSONS

Until 1859, only two bridges were built in the United States on pneumatic pile foundations, and the airlocks used in sinking the piles of these bridges was the invention of Alexander

Holstrom (Smith 1874). Europe was ahead of the U.S. in developing techniques for deep underwater foundations until the construction of the St. Louis Bridge in 1868. The two piers of the St. Louis Bridge were required to be founded on rock at a depth of more than 80 feet to avoid their failures from scouring during high floods in the Mississippi. Colonel Roberts (Roberts 1872–3) noted that during the sinking of the piers it was recorded that "a rise of only about 16 feet on top of the ordinary ten feet flow above extreme low water, has scoured out more than 40 feet of sand; the rock bed upon which the caissons finally rested exhibited a smooth, water-worn surface."

The caissons created by Eads were made of wrought iron (Figure 6). The east pier was 82 feet long by 60 feet wide and the west pier was 82 feet by 48 feet. The air chambers which were located at the bottom were 9 feet high. The caissons designed were very similar except the east piers had seven air locks while the west piers had five air locks.

The air locks were circular vertical chambers with diameters varying between 5 and 5½ feet and the height between 6 and 12 feet. The air locks had two doors; the first opening into the open air-shaft and the second leading into the air chamber. This system was designed in such a way that only one door would open at any time.

The air pressure in the air chamber was much higher than the atmospheric pressure in the open air shaft because it contained compressed air. Workers enter the air lock by entering through the first door between the air shaft and the air lock. After closing this door, the air pressure in the air lock is gradually increased by pumping compressed air into the chamber until the air pressure is equal to that in the air chamber. At this point the person opens the second door of the air lock and enters the air chamber where the construction activities are taking place, closing the second behind himself. Again, the pressure in the air lock is reduced to that of the atmospheric pressure. For those leaving the air chamber, the process is reversed and the air lock is pressurized first.

On the east pier, the maximum depth at which the caisson was founded was 110 feet, 6 inches below the surface of the water. With a pressure of 48 pounds per square inch, men were allowed to work one hour at a time, up to three times per day. Almost all of the excavation was removal of sand from the river bottom while the caissons were lowered by adding masonry on the top. One of the most simple and yet most novel, effective, and ingenious devices used for

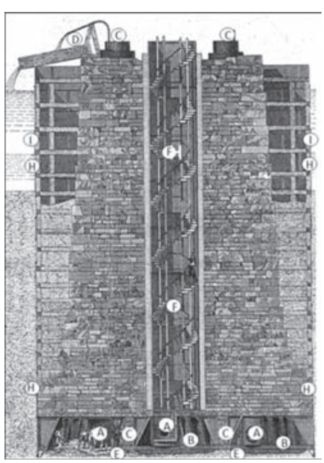

Figure 6. The caisson designed by James Eads (Engineering 1871).

A Air lock B Air Chambers C Timber girders D Discharge of sand pump
E Sand pumps F Main entrance shaft G Side shaft H Iron casing I Caisson bracing

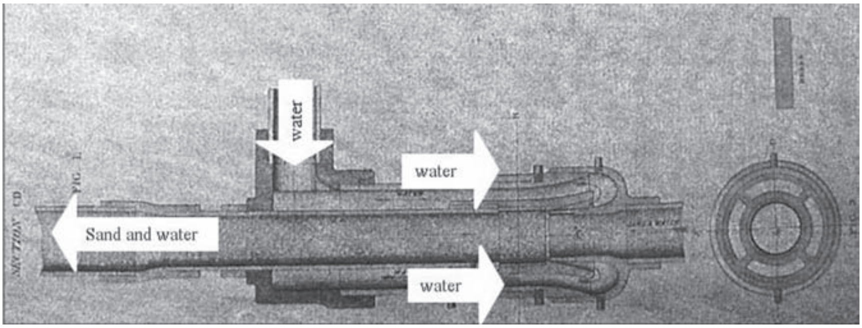

Figure 7. Details of the pump created by James Eads (Woodward 1881).

the removal of sand was the sand pump invented by Eads (Figure 7). There were no moving parts and, therefore, no danger of clogging the pump. The operation of the pump is described by Roberts as follows:

> *A stream of water is forced down through one pipe and caused to discharge near the sand into another annular jet in an upward direction. The jet creates a vacuum below it by which the sand is drawn into the second pipe or pump, the lower end of which is in the sand and water, and the force of the jet drives the sand on upward to the horizontal discharging pipe above the surface of the river as soon as it passes through the annular opening in the jet.*

For the east pier the sand pumps work perfectly when the discharge pipe was 120 feet in vertical height above the jet. It required a gang of six or seven men to shovel the sand and manage one sand pump; two sand pumps were powered with steam and a pump.

With the construction of the deep caissons and the first bridge over the Mississippi, Eads established his supremacy in solving the most difficult engineering problems of his time. He was the first one to place the air locks within the air chamber at the bottom of open air shafts and he invented a sand pump which considerably reduced the construction time necessary within the caissons. Other firsts in building the deep foundations for the St. Louis Bridge are listed by Woodward (1881).

8 ROEBLING'S DESIGN OF PNEUMATIC CAISSONS

Brooklyn had an air of festivities the day the first caisson (Figures 8a and 8b) was released from Greenpoint, Brooklyn in May 1870 (Roebling 1870a). The caisson was pulled five miles down the East River by six tugboats in two stages—to account for tides—over two days. Eventually it came to rest next to the Fulton Ferry docking slip (Roebling 1870b). Due to the fact that the work was being done out of sight, the caisson and the workers became a topic of intense interest for many New Yorkers with reporters and artists regularly visiting the work area and publishing reports. Construction of the caissons of the Brooklyn Bridge is covered in greater detail by Collingwood (1872–3, 1874, 1875).

Similar to Eads, Roebling intended for the base of the Brooklyn Bridge foundation to lie directly on the bedrock underwater. Washington Roebling, who had spent a great deal of time in Europe studying the structure of caissons, was to step in and become the engineer in charge of building the bridge. The dimensions of the caissons varied between the Brooklyn and Manhattan locations, but were approximately 170 feet long by 102 feet wide, or about half the length of a city block (Roebling 1872a). This would be the first attempt to sink a structure this large into the ground. The structure of the caissons built by Roebling were

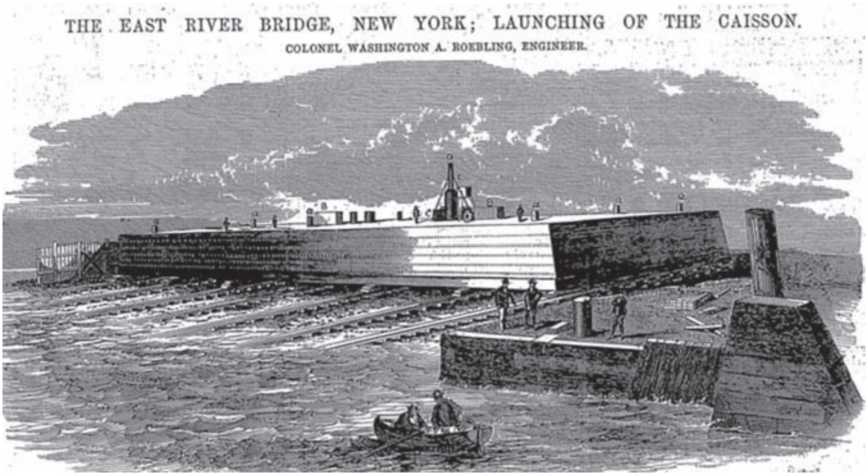

Figure 8a. The Brooklyn caisson before being launched (Engineering 1870).

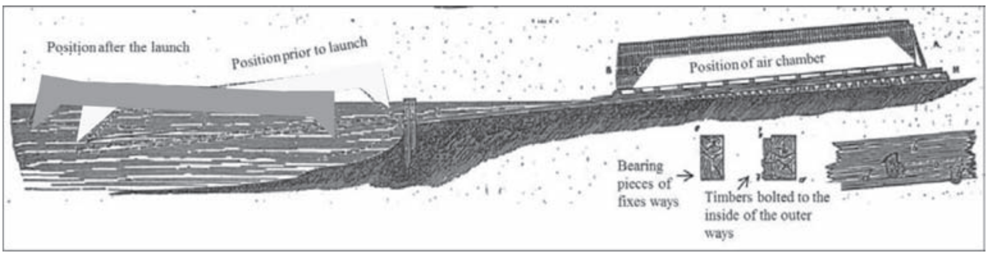

Figure 8b. Launching of the Brooklyn caisson (Engineering 1870).

similar to those designed by Eads; Roebling's caissons were built of yellow pine, caulked to prevent air leakage, and contained immense air-locks to monitor the air pressure in the work chamber (Roebling 1873a). Unlike the system devised by Eads, the caissons in New York used a sand syphon created by Roebling's team to remove the excavated materials (Collingwood 1872–3).

The objective of clearing away earth submerged below water was straightforward, but the workers began to face severe health problems as the caissons sank deeper into the ground. The workers were unaccustomed to working under such air pressure and suffered from decompression sickness due to a build-up of nitrogen in their bodies. It was eventually discovered that workers should not quickly emerge after working; instead, the safest method was to slowly climb the stairs when finished working. As the caissons continued to sink deeper and be succumbed to greater pressure the workers began to tire quickly and work shifts were decreased to keep the workers in good health (Roebling 1872b, 1872c).

Washington Roebling also suffered from multiple attacks of the bends throughout the construction of the Brooklyn Bridge; his health problems eventually exacerbated to the point in which he supervised the bridge construction via a telescope from his home (Salvadori 1980). Yet, despite the setbacks, the work progressed with bedrock reached in New York in May of 1872 (Roebling 1872b). The depth of the bedrock was significantly shallower in New York than in St. Louis. On the Brooklyn side of the East River, workers hit bedrock at 44.5 feet and in New York bedrock was reached at 78.5 feet (Collingwood 1872–3).

9 EADS' LETTER IN *ENGINEERING*

The disagreement between James Eads and Washington Roebling was linked to ownership in the design of caissons; the problem began with Roebling's publication in 1872 of a 92-page booklet titled "Pneumatic Tower Foundations of the East River Suspension Bridge" (Roebling 1872a). This booklet explained the construction and purpose of the caissons used for the foundation of the East River Bridge. In the second paragraph of the preface of his booklet Roebling acknowledged that despite his two pneumatic caissons (located in Brooklyn and New York) being the largest in size, the St. Louis Bridge caissons went deeper than those used in building the Brooklyn Bridge. Roebling's account of the construction of the two caissons was largely impersonal and did not blatantly claim ownership of the idea of placing the air-lock at the bottom of the air shaft rather than above water level. A paragraph on page 71 of his booklet became the reason for the feud between Eads and Roebling, as well as a $100,000 lawsuit filed by Eads against Roebling for the infringement of his patent. The paragraph reads as follows:

> *The idea of placing the air-lock at the bottom of the air-shaft, below the water level, in place of above it, in masonry caissons, is not new, having been proposed in England as long ago as 1831 by Lord Cochran, and again by Wm. Bush in 1841, and still later in 1850 by G. Pfaunmuller, of Mayence. It, nevertheless, remained for Captain Eads, in his St. Louis caissons, to make the first practical application of the same on a really large scale in this country.*

To a casual reader, this paragraph credited Eads for converting the European idea into a reality for the first time in the United States, Yet, as Eads read these words, he believed the statement implied that he had copied the caisson design—specifically placing the air-lock at the base of the air shaft—from predecessors and was not attributed credit for his innovations. Eads found this statement to be tremendously insulting.

After Washington Roebling published his booklet, James Eads responded with a letter to the editor of *Engineering* on April 16, 1873 which was published on May 16, 1873 (Eads 1873a). In this letter, he asserted that the previous engineers referenced by Roebling did not use the same air-lock system as Eads' design with the air-lock placed below water level. Eads also claimed his open-air staircase shaft system was an original design for which he deserved credit.

Eads taunted and tormented Roebling about his lack of understanding about patents held by previous inventors, Lord Cochrane and William Bush, whose names Roebling had mentioned in his paragraph to which Eads objected. When Eads was—seemingly—giving credit to Roebling, his tone was dripping with sarcasm. An example follows:

> *Colonel Roebling's generosity to me in one point has made him unjust to himself, and has probably delayed his acknowledgment to Herr Pfaunmuller. The credit he gives me for making the first practical application of Pfaunmuller's idea "on a really large scale in this country" belongs exclusively to himself.*

The implication, according to Eads, was that Roebling had not only copied Pfaunmuller's ideas, but Eads' ideas, as well. Eads ends his letter by saying that Roebling's failure for the last three years to credit him with the design of the pneumatic caissons which Roebling appropriated, and his silence to give Eads the credit for his patent, had compelled him to correct the record. Nowhere in his letter did Eads mention that he had filed a $100,000 lawsuit against Roebling for infringement of his patent.

10 WASHINGTON ROEBLING'S REPLY

When Eads' letter was published in *Engineering*, Roebling was traveling in Europe. His letter dated at Wiesbaden, Germany was June 12, 1873 and was published in *Engineering* on June 27, 1873 (Roebling 1873c). After reading Eads' letter in *Engineering*, Roebling observed that Eads' "skill in blowing his own trumpet is only surpassed by his art in writing abusive and unjust articles about other people." Roebling also claimed that the description of the pneumatic caissons from his booklet gave credit to Eads' originality with a nod to the influence of predecessors in Europe. He explained the nature of the dispute between himself and Eads as follows:

> *The principal point of dispute, however, between us is as follows: Captain Eads virtually makes the broad claim that any device which has been used in a pneumatic cylinder can be made the subject of a new patent when applied to a "masonry caisson" and in that spirit has had several patents granted. I choose to differ with him on that point, and before paying the round sum with which he proposes to tax the engineering world for the next fifteen years, I have preferred to leave the matter to the decision of the court of law, where it is now being tried.*

Roebling's main argument against Eads' patents was that neither did Eads invent the airlock, nor did he invent any device related to the airlock. Eads' main innovation was positioning the air lock at the base of the air shaft rather than at the top; Roebling did not believe this warranted a patent.

Roebling's skepticism about granting patents by different countries without thorough research and investigation was justified. In 1869 he learned that patents were granted in England for wire rope fastenings designed by Charles Lundborg in Sweden (Figure 9). He responded by showing wire rope fastenings developed and used by his father's business more than 30 years ago (Figure 10).

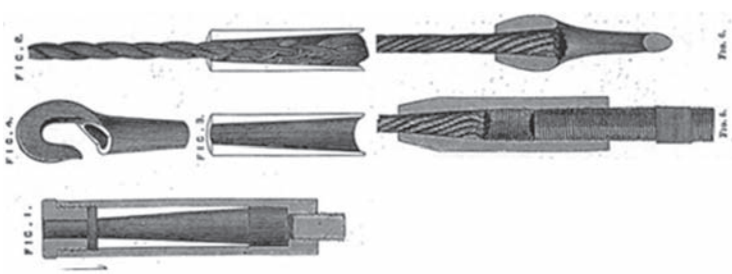

Figure 9. Wire rope fastenings designed in Sweden by Charles Lundborg and patented in England by J.E. Holmes in 1869 (Engineering 1869b).

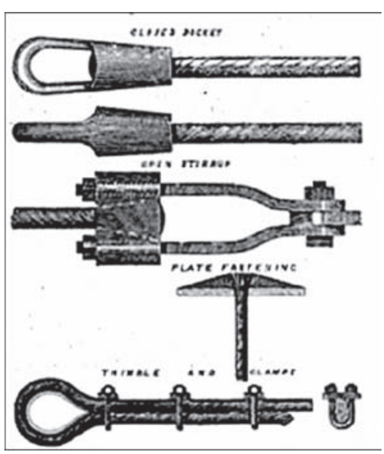

Figure 10. Wire rope fastenings used by John A. Roebling since 1839 without a patent (Engineering 1869c).

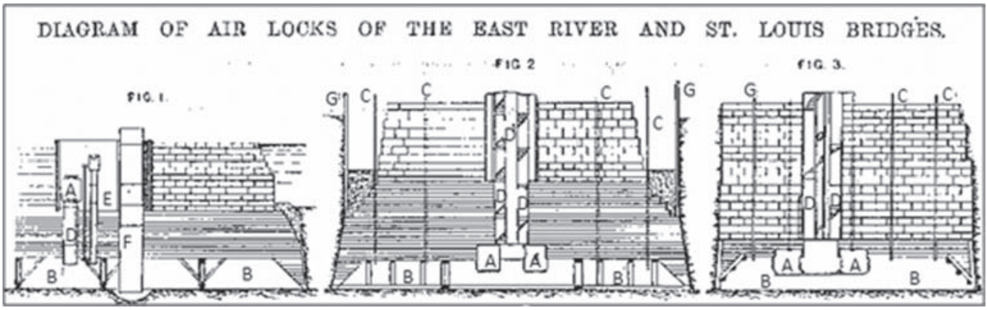

Figure 11. A—Air locks, B—Air chamber, C—Sand pipes, D—Air shaft, E—Supply shaft, F—Water shaft, G—Cofferdam (Eads 1873b).

If Roebling had concluded his letter in *Engineering* at this point, the statement would have been less offensive. However, he went on to accuse Eads of stealing his ideas and insulted him by calling him names. Roebling sarcastically commented that "… where would you go to find an easier material to sink [a caisson] through [a river] than at St. Louis, or a more difficult one than in the East River" (Roebling 1873b). This response did not help Roebling to establish a good relationship with the public and continued to escalate his dispute with Eads.

11 REJOINDER FROM EADS

The published response from Washington Roebling encouraged James Eads to—again—respond to the claims in *Engineering*. In September 1873 his response was published showing three cross-sections of caissons (Eads 1873b). The first was Roebling's original design, the second was Eads' design, and the third showed Roebling's modified design (Figure 11). If the drawings are correct, the structure designed by Roebling was clearly influenced by the ideas of Eads.

Although the exact construction dates are somewhat unclear, it is agreed that James Eads began constructing his caisson prior to Washington Roebling. These letters revealed that Roebling had visited Eads in St. Louis in 1870 to view the caissons and later Eads had visited the work site of Roebling to see the progress of the Brooklyn caisson. Eads' response to Roebling also mentioned a series of five features in the St. Louis caisson that he believed were unique by being used in combination together. The remainder of Eads' letter devolves into a series of cantankerous criticisms over trivial matters.

12 THE U.S. ARMY ENGINEERS' INVESTIGATIONS AND FINDINGS

In 1873, while the St. Louis Bridge was under construction, several business men linked to the steamboat companies filed a formal protest against the bridge to the Secretary of War. He convened a "Board of Engineer Officers" (the Board) to examine the construction of the bridge and report if the bridge would prove a serious obstruction to navigation, and if so, to determine the means of modifying the construction.

The Board consisted of Colonel J.H. Simpson who had his headquarters in the city of St. Louis, Major Governeur K. Warren (brother-in-law of Washington Roebling), Major G. Weitzel, Major William M. Merrill, and Major Charles R. Suter.

After two days of hearings, the Board issued a report on September 11, 1873 addressed to General A.A. Humphreys, Chief Engineer of the U.S. Army, concluding that the "bridge, as at present designed, will prove a very serious obstruction to the navigation of the Mississippi

River." To remedy this problem, the Board recommended building a 120 foot wide canal behind the east abutment with a draw bridge over the canal to allow the easy passage of large boats (Simpson, et al., 1873).

General Humphreys concurred with the findings of the Board, and submitted the report to William Belknap, the Secretary of War, with a recommendation dated October 6, 1873 suggesting that it be submitted to Congress during the next session for further action. The Secretary of War approved the recommendation of General Humphreys on October 10, 1873 and authorized him to forward the report to the bridge company. General Humpreys complied and submitted the report on October 15, 1873.

13 EADS' RESPONSE TO THE U.S. ARMY ENGINEERS' FINDINGS

Eads and the officers of the Bridge Company were concerned that the U.S. Army Engineers' report would affect their credit rating and their ability to raise capital to complete the bridge (Woodward 1881). Eads vigorously defended the bridge by responding to each and every objection raised in the report, concluding that the bridge was built according to the dimensions listed in the charter of the Bridge Company and approved by the U.S. Congress (Woodward 1881).

An anonymous letter published in the *Railroad Gazette* criticized the Army Engineers' report and provided the cost estimate prepared by Colonel Henry Flad (the assistant engineer to Eads) to build the canal and the draw bridge (1873). This cost estimate was \$3,065,497, however the letter commented that the cost would increase to \$10 million if the work was done by the government.

The Board of Engineer Officers reconvened in January 1874 to review the response from Eads, the survey, and the cost estimates for the canal and drawbridge. The Board issued a Supplementary Report dated January 31, 1874. It estimated the total cost to implement the recommendations to be \$1,172,436.12. The Supplementary Report reaffirmed its original decision that "if no acceptable modifications can be made (to the bridge) then it (the bridge) should be entirely reconstructed" (Woodward 1881).

General Warren wrote a separate opinion expressing his frustrations against Eads and his supporters for framing a charter to build a new bridge which explicitly excluded the construction of a suspension bridge. As stated before, John Roebling had submitted his proposal in 1865 to build a suspension bridge across the Mississippi River at St. Louis which was rejected. In part, his report stated:

> *I am not indifferent to the interest of those who have lavished their money in this undertaking* [of building the St. Louis Bridge]; *but a greater public interest should not be destroyed unnecessarily for their sake. I am convinced that a bridge suited to this great want, at an expense much less than has already been made, almost if not entirely unobstructing* [sic] *navigation, could years ago have been completed, upon design well known and tried in this country, had not the authors of the present monsters stood in the way* (Woodward 1881).

Although Washington Roebling had nothing to do with the opinion expressed by his brother-in-law, the public in general and engineers in particular, considered this as retaliation against Eads and related to the feud between Eads and Roebling.

Eads was able to defy the recommendations of the U.S. Army Engineers due to his relationship with President Ulysses S. Grant. After consulting with Eads, Grant issued an opinion stating that "Congress had authorized the building of the bridge … so only Congress could decide to pull it down" (McCullough 1983). Once the bridge was built, it looked magnificent and people appreciated it. Congress did not vote to pull down the bridge; and the whole matter of it being an obstruction to navigation was soon forgotten.

Since 1873, Roebling had been crippled with the "bends", and was not in any position to direct his work force on the construction site. Instead, he needed to conserve his energy and concentration to focus not only on the construction, but also the financial, administrative, and political problems facing the project.

The years 1875 and 1876 were very hectic, as far as activities for the Brooklyn Bridge were concerned. The removal of the existing buildings on the site of the anchorage had started on May 7, 1875 and excavation had reached a depth of twenty feet. As of February 1876, the New York tower had reached the height of 208 feet and left 30 feet yet to be completed. The anchorage on the Brooklyn side was practically complete and the New York anchorage was in an advanced condition. Since August 1875, 27,000 CY of the masonry of the anchorage was completed. The last of the masonry of the anchorages was completed in the beginning of September 1876. The towers and anchorages were completed and the construction of the bridge proper was about to begin, including the spinning of the cables between the two anchorages passing over the two towers. Each cable was going to have 19 strands and each strand was going to contain 330 wires.

The idea of defending himself against the lawsuit by Eads was mentally taxing. Had Roebling been as physically healthy and active as Eads, he would have likely defended Eads' claim in a court of law. Rather than defend himself, he decided to make an out-of-court settlement for $5000 (Engineering News 1876). At the time, Roebling was in poor health, had a crushing workload, and the dispute had been unresolved for five years. It was prudent for Roebling to make a settlement of $5000, or five cents on the dollar for the amount which he was originally sued, to Eads. For Eads this was a moral victory, and an acknowledgment from Roebling that he had copied Eads' design of pneumatic caissons by placing the airlocks at the bottom of the air shafts. The five year old feud between the two giants of civil engineering was finally settled for $5000, after both Eads and Roebling had spent five years and a great deal of money attempting to defend their reputations.

14 CONCLUSIONS

Given the harsh upbringing of Eads—who had to fend for himself since he was thirteen—and the comparatively comfortable childhood of Washington Roebling, it became evident that Roebling was not well-equipped to handle the ugly politics of a public dispute. Eads, on the other hand, had a history of working in the business world. This experience provided him with a real-world savvy in handling the dispute with skill. Eads understood how to make his point while winning over public opinion; he emerged from the fight as the winner. Roebling, on the other hand, lost his temper in a public forum and shortly after gave up the argument against Eads' claims. Years later, as evident in the quote below (last sentence), Roebling reflected on the harsh repercussions of the dispute due to his poor handling of the situation.

More than the financial loss of $5000, Roebling's hurt was psychological and one of isolation as a result of the feud with Eads. He discussed this incident later in life as follows:

> *I adopted this suggestion of [air-locks from] Mr. Sickels [which were used] in the N. Y. Caisson—This led to a fierce attack and lawsuit for $100,000 damages on part of Cpt. Eads of St. Louis to whom a patent for this idea had been granted in the meantime … The whole Engineering world in the U.S. sided with Cpt. Eads, most unjustly so, because the facts are exactly as here stated. These feelings were actuated entirely by jealousy—the Brooklyn Bridge was by far the largest and most prominent Engineering work of its day—Throughout its construction the attitude of the Engineering fraternity towards me was one of hostility, envy and hatred (Mulholland 2006).*

Unfortunately, Roebling never recovered from this isolation for the remainder of his life.

ACKNOWLEDGEMENTS

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