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THE CONCRETE BRIDGE MAGAZINE

SUMMER 2015

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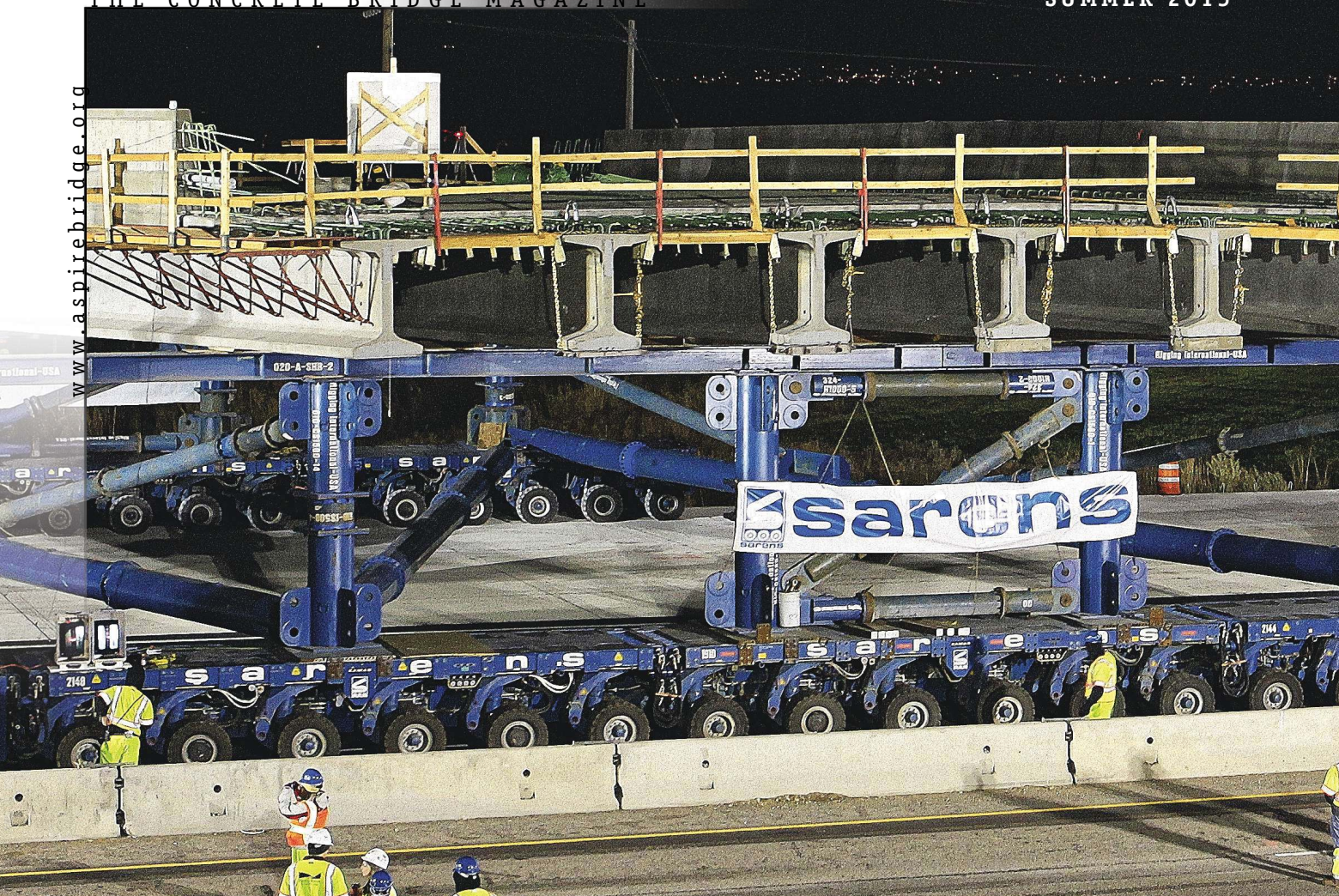

Proctor Lane Bridge
Utah County, Utah

HONOLULU RAIL TRANSIT PROJECT
Honolulu, Hawaii

I-84 BRIDGES OVER MARION AVENUE
Southington, Connecticut

U.S. HIGHWAY 12 BRIDGES AT MILEPOSTS 17.3, 19.6, AND 19.8
Broadwater County, Montana

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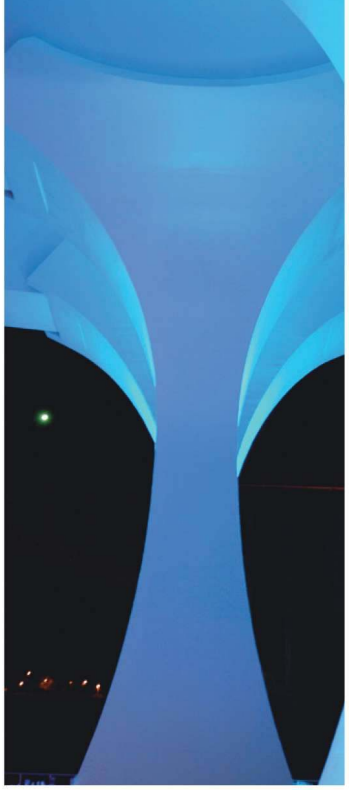

NEW I-35W BRIDGE, MN

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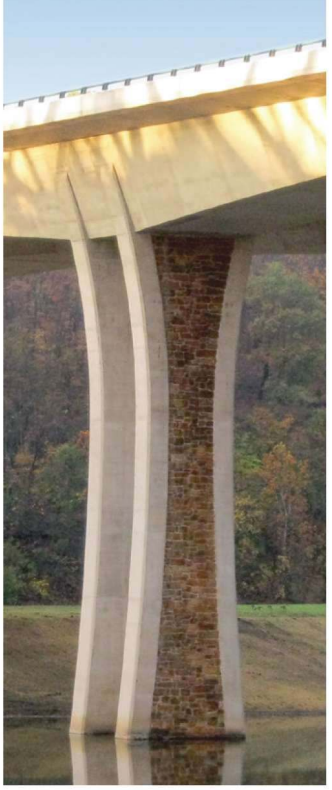

I-76 ALLEGHENY RIVER BRIDGE, PA

OPENING 2017
SARAH MILDRED LONG BRIDGE, ME
RENDERING

OPENING 2016
I-91 BRATTLEBORO BRIDGE, VT
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HONOLULU RAIL TRANSIT PROJECT
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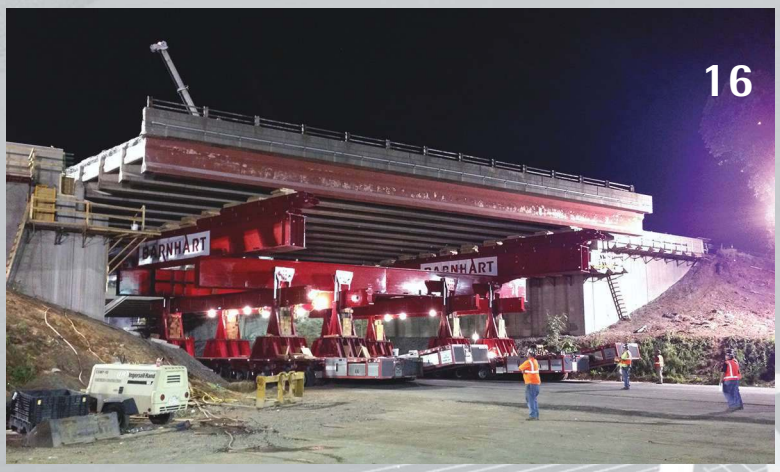

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Turning Internal Champions into External Stars

William Nickas, *Editor-in-Chief*

Our profession has an underlying set of rules and when followed they lead to consistent results. These traditional practices are being updated with new delivery schemes and complex new materials. The goal of high-performance assets is to go beyond what has been in the past: to deliver structures that will have low initial cost, low environmental impact, and extremely low reoccurring operating and rehabilitation costs over the long term.

Concrete structures are repeatable, rugged, and durable. Past practice has identified decks, bearings, and joints as the most vulnerable to environmental damage. In seismic areas, where integral superstructure and substructure details are most common, moisture and chlorides are no longer reaching substructure caps. These details have put a stop to a vulnerability of the past. Using two-way prestressing in decks keeps the concrete tensile stresses low and decks are lasting longer.

Dr. Bayrak wrote in his Winter 2015 *ASPIRE*™ article about some important traits of the profession and the need for strong engineering fundamentals. These key principals can be applied to all levels of our heavy civil construction industry.

The instability of long-term transportation funding has created unproductive angst and talented people have left all segments of our construction industry. Project managers, engineers, superintendents, crew chiefs, equipment operators, and the list goes on, are critical links in the delivery of new bridges and roads. When you find employees that have the skills and basics tenacity, please invest and grow those people by exposing them to new concepts and allowing them to further investigate recent innovations. Lee Iacocca said, "In times of great stress or adversity, it's always best to keep busy, to plow your anger and your energy into something positive."

Just run a benchmark test and identify one or two internal person(s) to investigate a recent change that has occurred for one aspect of concrete bridge delivery in your region. (There are plenty of topics within the Design-Build procurement delivery method or prefabricated bridge elements and systems [PBES] arena.) Ask that person to develop the idea as a parallel to a traditional solution and watch the excitement build. Developing internal mavericks will get your firm noticed. Two years ago in the Spring 2013 *ASPIRE*, I wrote of "Creating a Lasting Separation from your Competitor." Today, mid-size firms are disappearing because of consolidation. Whether a mega firm or a smaller boutique firm, both need to foster a new spirit of innovation that allows identification of a new internal champion and brings new vision to your customers. With this increased value, you will be able to win work and command higher fees with bigger margins.

Another way to engage employees is by allowing them to advocate sustainability efforts within your company or for your projects. In the next issue of *ASPIRE*, we will provide an update on sustainability activities in the bridge industry that may interest an internal champion in your firm.

This year, *ASPIRE* launched a few new series to help you refresh your whole team on detailed, specific areas of concrete bridge information. Take a few minutes to use the new online version to search for a topic in all past issues and look at how others have utilized concrete to achieve a desired outcome.

Retaining talented employees is a challenge for many in our arena, but mentoring those employees may inspire their creativity, which could Turn an Internal Champion into an External Star.

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Self-propelled modular transporters allowed two spans of precast concrete girders for the Proctor Lane Bridge, Utah County, Utah, to be moved into place during a 6.5-hour overnight closure. Photo: Jacobs.

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Publisher
Precast/Prestressed Concrete Institute
Robert Risser, President

Postmaster: Send address changes to *ASPIRE*, 200 W. Adams St., Suite 2100, Chicago, IL 60606. Standard postage paid at Chicago, IL, and additional mailing offices.

ASPIRE (Vol. 9, No. 3), ISSN 1935-2093 is published quarterly by the Precast/Prestressed Concrete Institute.

If you have a project to be considered for *ASPIRE*, send information to *ASPIRE*
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PRESTRESSED CONCRETE BRIDGES

Route 70 over Manasquan River in New Jersey (Photo courtesy of Arora Associates) Alternate design structure utilizes precast caissons, piers, pier caps, and prestressed concrete beams. The bridge was opened to traffic two years ahead of as-designed schedule.

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Brett H. Pielstick is a senior vice president/principal at Eisman & Russo Inc. in Jacksonville, Fla. He is author of the 2007 and 2012 ASBI Durability Survey Report of Segmental Bridge and is chair of ASBI Bridge Condition Advisory Committee.

CONCRETE CALENDAR 2015/2016

For links to websites, email addresses, or telephone numbers for these events, go to www.aspirebridge.org and select "EVENTS."

August 2-7, 2015
AASHTO Subcommittee on Materials Annual Meeting
Marriott Pittsburgh City Center
Pittsburgh, Pa.

September 9-11, 2015
2015 Western Bridge Engineers' Seminar
The Peppermill Hotel
Reno, Nev.

September 15-17, 2015
Fall 2015 National Concrete Consortium Meeting
Hyatt Regency Milwaukee
Milwaukee, Wisc.

October 7-9, 2015
2015 PTI Committee Days
Harrah's New Orleans Hotel
New Orleans, La.

October 15-18, 2015
PCI 2015 Fall Committee Days and Membership Conference
Louisville Convention Center
Louisville, Ky.

November 2-3, 2015
ASBI 27th Annual Convention
Omni Dallas Hotel
Dallas, Tex.

November 8-12, 2015
ACI Fall Convention
Sheraton Denver Downtown Hotel
Denver, Colo.

December 7-8, 2015
2015 National Accelerated Bridge Construction Conference
Hyatt Regency Hotel
Miami, Fla.

January 10-14, 2016
Transportation Research Board 95th Annual Meeting
Walter E. Washington
Convention Center
Washington, D.C.

February 1-5, 2016
World of Concrete 2016
Las Vegas Convention Center
Las Vegas, Nev.

March 1–5, 2016
2016 PCI Convention and National Bridge Conference at the Precast Show
Gaylord Opryland Resort
and Convention Center
Nashville, Tenn.

April 11-12, 2016
ASBI 2016 Grouting Certification Training
J. J. Pickle Research Campus
Austin, Tex.

April 17-21, 2016
ACI Spring Convention
Hyatt & Frontier Airlines Center
Milwaukee. Wis.

April 20-22, 2016
2016 DBIA Design-Build in Transportation Conference
Charlotte Convention Center
Charlotte, N.C.

April 24-27, 2016
2016 PTI Convention
Long Beach, Calif.

New PCI President

The Precast/Prestressed Concrete Institute (PCI) has named Robert J. Risser its new president. Risser has more than 20 years of executive association management experience and, prior to joining PCI, was the president and CEO of the Concrete Reinforcing Steel Institute (CRSI) in Schaumburg, Ill.

Bob serves the ACI Foundation as a member of the board of directors for the Strategic Development Council and the executive committee of the Concrete Research Council. Risser received his master's and bachelor's degrees in civil engineering from the University of Illinois and is a registered professional engineer.

ENGINEERING RELATIONSHIPS

Jacobs relies on strong relationships, repeat clients, and value-focused design decisions as it delivers innovative and efficient bridges in an evolving bridge market

by Craig A. Shutt

Since its founding in 1947, Jacobs has risen to become one of the world's largest and most diverse providers of technical professional and construction services. It also has become a specialist in total project delivery for major, award-winning bridge projects. Ever evolving, Jacobs has recently developed a value-oriented analytic tool, named JacobsValue+, that allows clients to verify savings the firm has produced, which in 2014 totaled a record $5.3 billion.

"We have built our reputation on being relationship-based, which is one of the core values of our company," explains National Bridge Principal Marcos P. Loizias. "We focus on forging strong, long-term relationships, and it's a major contributor to our success in becoming one of the world's largest engineering firms." He notes that more than 90% of the firm's work comes from repeat customers.

"Clients rely on us to be strategic and practical," he says. "In designing and building bridges—and in delivering all the different types of projects we do in a diverse range of markets—our customers want us to help them save money while being innovative, environmentally sensitive, and understanding of community concerns."

JacobsValue+

One of the key ways Jacobs proves its value comes through its JacobsValue+ program, in which the company documents the savings that it provides through ideas, innovations, value-engineering, and constructability reviews of designs as they are being developed. "The clients confirm these savings through a formal verification process to support our claims," Loizias explains.

Many of the changes that drive these savings result from engaging subject-matter experts and specialists from the firm's resources around the globe. "While local expertise is sufficient for many projects, our access to uniquely specialized expertise at Jacobs provides our clients with the best technical knowledge, experience, and methods available," Loizias says. "Our design and technical groups regularly confer with other value-engineering experts, who help us brainstorm new ideas."

Creating value-engineered, cost-effective solutions has become more prevalent as many clients began adopting alternative delivery forms,

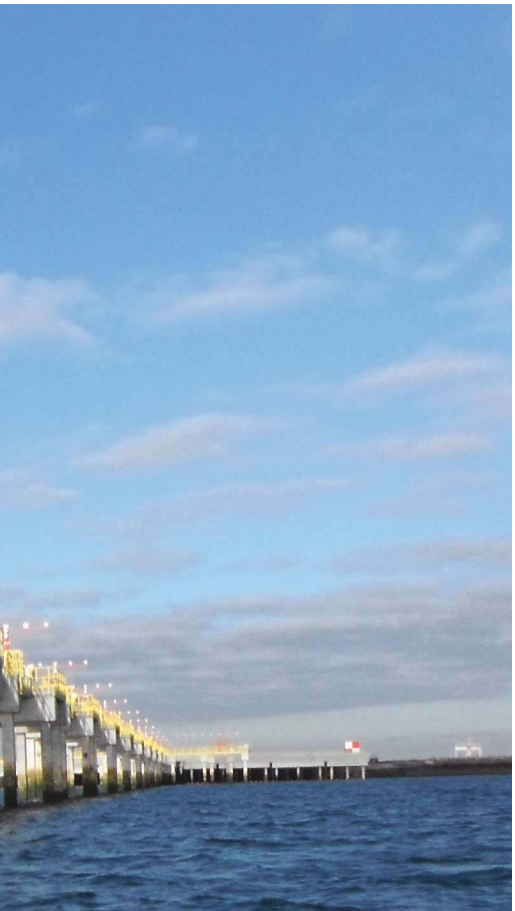
As part of the improvements at Boston-Logan International Airport, a 1700-ft-long concrete pier was constructed to support the approach lights. All Photos: Jacobs.

especially public-private partnerships (P3) and design-build options. "Currently, more than 70% of our bridge work is done on design-build and P3 projects, primarily supporting contractors as the lead engineer and design manager," notes Loizias. "Doing so much design-build work with contractors allows means and methods to be incorporated as cost-effective ideas early into the design process. I see this bringing more innovation. It allows us to think outside of the box."

Design-Build Grows

Jacobs is currently involved with multiple national design-build projects, including serving as the lead designer for the two Ohio River Bridges connecting Indiana and Kentucky. The project consists of the $860-million Downtown Crossing, which is being done on a design-build basis, and the $760-million East End Crossing, which is a P3 project. Both projects feature signature cable-stayed bridges over the Ohio River. For the Downtown Crossing, 47 bridges feature prestressed concrete girder designs. The East End Crossing includes 11 new prestressed concrete girder bridges and five rehabilitated concrete structures.

Jacobs is also working with joint-venture partner HDR to deliver final design of the $2.3-billion I-4 Ultimate P3 project in Florida, in which 95 bridges will use precast, prestressed concrete Florida I-beams and U-beams.



At 4.1 miles, the $71.6-million St. George Island Bridge Replacement is one of the largest design-build projects undertaken by FDOT. It features high-capacity, 54-in.-diameter precast concrete cylinder piles with precast concrete caps. The design provided greater corrosion resistance, sped up construction, carried greater load, and provided the longest post-tensioned spliced girder structural unit at the time.

Jacobs has thrived on design-build projects, working both for the contractor and as the owner's advisor. In 2003, it handled design, construction oversight, environmental impact statements (EIS), permitting, and other areas of the $71.6-million St. George Island Bridge Replacement Project for the Florida Department of Transportation (FDOT). One of the largest design-build projects undertaken by FDOT at the time, the bridge is the third longest in the state at 4.1 miles.

The new bridge, which is more efficient and safer than the original, featured new high-capacity, 54-in.-diameter precast concrete cylinder piles with precast concrete caps. The design provided greater corrosion resistance, sped up construction, carried greater load, and included the longest post-tensioned, spliced-girder structural unit at the time (comprised of five spans varying in length from 208 to 258 ft for a total length of channel unit of 1180 ft). The total length of the prestressed concrete bridge from abutment to abutment is 21,615 ft. The other 160 spans consist of 125- and 140-ft-long spans in four-span continuous structural units. Among the awards the design won was the Harry H. Edwards Award for Innovation from the Precast/Prestressed Concrete Institute.

Durability Needs Grow

The need for durability has continued to increase in the time since that design was completed. "Especially on design-build and P3 projects, there is a much more stringent focus on durability today," explains Loizias. "The goal is to design for a 100-year service life in many cases, which requires a thorough corrosion-protection plan. Our overall approach combines a high-quality design with strategies to achieve high durability and elimination of non-durable details, the use of highly durable materials, a thorough quality-control program during construction, and a detailed inspection and maintenance manual."

These needs often lead owners and engineers to concrete components. "Concrete offers more durability and less maintenance, creating an economical design in many cases. We can adjust the concrete to meet specific requirements to achieve reduced permeability and greater corrosion resistance. Concrete components of the bridge are designed to achieve the specified service life with a target confidence level of 90%. Typically, we provide several options of mixture proportions for the contractor to choose from."

Speed Increases

Owners also are looking for faster schedules, a requirement that plays to Jacobs' strengths. "We have been using accelerated bridge construction [ABC] methods since the early

1960s," says Loizias, pointing to work on the Chesapeake Bay Bridge and Tunnel Crossing in Norfolk, Va. Built in 1964, it included 12 miles of prestressed concrete low-level trestles with full-span, precast, prestressed concrete girder/deck modules and precast concrete pier caps. Pier caps were supported on three 54-in.-diameter precast concrete cylindrical piles designed to withstand 20-ft-high waves and 10-ft-high storm surges. Proving the cost effectiveness, ease of construction, and low maintenance of this design, 35 years later similar trestle structures were used on the Parallel Crossing, which opened in 1999.

Loizias points out that owners are emphasizing shorter construction-schedule times, as well as maintenance of traffic lanes, staged construction, and increased constructability, to mitigate the economic impacts of congestion and inconvenience to the traveling public during construction.

Many of these challenges are addressed by ABC techniques. "The use of ABC is being considered in more and more projects, including smaller, conventional projects," he says. "We believe the use of precast components has a lot of potential. They can accelerate construction on-site economically. Precasting results in a better product, but often the decision to use precast or not is driven by the availability of local suppliers."

Jacobs incorporated ABC into the I-15 Corridor Expansion (CORE) design-build project in Utah County, Utah, the largest road-construction project ever undertaken by the Utah Department of Transportation. The $1.1-billion project included 24 miles of reconstruction impacting 55 bridges on 29 new and modified interchanges. At the new three-span Proctor Lane Bridge, two 126-ft-long spans consisting of BT58 precast, prestressed concrete girders were moved into position using self-propelled modular transporters (SPMTs) requiring a single 6.5-hour overnight closure. It marked the first time two spans were moved with SPMTs in a single-night closure.

For the three-span Proctor Lane Bridge on Utah's $1.1-billion, I-15 Corridor Expansion project, two 126-ft spans of precast concrete girders were moved into position using self-propelled modular transporters.

"From our experience, the use of prefabrication and precasting offers numerous benefits beyond ABC," notes Loizias. "It also improves the quality of bridge elements, since they are constructed in a plant-controlled environment using high-quality materials and standardized production processes. This improved quality leads to extended bridge service life and reduced life-cycle costs. Other advantages include reduced on-site construction time, which offers key benefits with regards to traffic impacts, safety, environmental, and weather impacts."

New Concrete Designs

Many of the design challenges arising today can be met with new concrete designs, Loizias says. "Precast concrete beams are extending their length, pushing from 150 ft a few years ago to 300 ft or more," he says. "That allows us to eliminate piers in the navigational channels and work from above to minimize disruption to the environment."

Balanced-cantilever methods and overhead gantries or travelers are gaining popularity as environmental issues become more prominent, he notes. An example is the Route 364 Creve Coeur Lake Memorial Park Bridge between St. Louis and St. Charles counties in Missouri, constructed over an environmentally sensitive lake.

The dual five-lane, side-by-side segmental structures feature cast-in-place, post-tensioned box girders with sloping, variable-depth webs. The 2675-ft-long structure consists of nine spans constructed by the balanced-cantilever method with form travelers. This approach was used to construct the bridge entirely from above and minimize environmental impacts. The project won a variety of awards, including the American Segmental

The Route 364 Creve Coeur Lake Memorial Park Bridge between St. Louis and St. Charles counties in Missouri, constructed over an environmentally sensitive lake, features cast-in-place, post-tensioned box girders with sloping, variable-depth webs. The nine-span project was constructed by the balanced-cantilever method with form travelers and was the first segmental bridge built in the state.

An all-precast concrete system was specified for the Boston-Logan Airport design, including piles, pile caps, and New England Extreme Tee beams to support the 141,000-ft² engineered-materials arresting system deck.

Bridge Institute's 2005 Bridge Award of Excellence.

Jacobs' designers are also seeing great potential in new girder designs, such as the New England Extreme Tee (NEXT) developed in conjunction with the Precast/Prestressed Concrete Institute's New England regional arm. The firm made extensive use of the girders in its design for the 600-ft-long Runway Safety Area at Boston-Logan International Airport, which included an engineered-materials arresting system (EMAS) placed on a 303-ft-wide, pile-supported deck that extended 470 ft into Boston Harbor.

The Massachusetts Port Authority required the deck to handle the structural design-load capacity of a fully loaded Boeing 747 aircraft, provide a minimum 75-year service life, and be completed "on an extremely aggressive schedule with limited construction windows," he explains. That led the design-build team to specify an all-precast concrete system, including piles, pile caps, and NEXT beams to support the approximately 141,000 ft² EMAS deck. It was one of the largest uses of the NEXT beams.

Segmental concrete construction also offers great potential, he notes. "Segmental girders, especially with high-performance concrete, give us much more flexibility today." An example is the Route 36 Highlands Bridge over the Shrewsbury River in Sandy Hook, N.J. The twin 1610-ft-long precast concrete segmental box-girder bridges were built with the balanced-cantilever method maintaining traffic uninterrupted throughout construction.

The bridges were constructed in stages to replace the existing historic double-leaf bascule bridge. The substructure for each box-girder structure featured single-column, precast concrete, post-tensioned, hollow-box-section piers, along with prestressed concrete piles and precast concrete cofferdam cells. Each bridge was constructed in only 15 months. "Precasting the bridge from the footings up provided not only for rapid construction but also for long-term performance and durability." See *ASPIRE*™ Summer 2010 for more details.

Future Options

Successes like these have states looking to designers and contractors for innovative approaches. That's especially true with changes to concrete techniques and new materials. "Trends that we've witnessed include an increased use of higher strength concrete mixes in precast elements, to 8 to 10 ksi, self-consolidating concrete, high-slump concrete, and occasionally lightweight concrete, especially for bridge decks on existing trusses to reduce weight and increase load capacity," says Loizias. "As states become more familiar with lightweight concrete, I expect there will be greater use in precast girders, deck slabs, and segmental bridges."

Extradosed bridges are also gaining attention. These bridges feature post-tensioning outside the box section serving much like the cables in a cable-stayed bridge but at shallower inclination and with shorter towers. Several have been built in North America already. "The efficiency of concrete extradosed bridges is expanding to fill the gap in the 500- to 700-ft range where steel trusses or arches are more cost effective than cable-stayed bridges," Loizias says. "Extradosed bridges offer another way to use concrete as a solution for creating economical designs. I see more of them being used in the future."

Jacobs' Past Fuels the Future

Since opening in 1947, Jacobs has evolved from a one-man, engineering-consultant firm to a publicly traded Fortune 500 company and the second largest design firm in the world in 2015, according to *Engineering News Record.*

Its primary markets are aerospace and defense; automotive and industrial; buildings; chemicals; food, beverage, forest and consumer products; infrastructure; mining and minerals; mission-critical and high-tech facilities; nuclear; oil and gas; pharmaceuticals and biotechnology; power and utilities; refining and petrochemicals; telecommunications; transportation; and water and wastewater.

Headquartered in Pasadena, Calif., the firm has 66,000 employees in 250 offices located in 35 countries around the world. Its 2014 revenue was $12.7 billion.

As new designs develop, Jacobs will remain on the cutting edge, Loizias says. "We have been at the forefront of many technical innovations and have the expertise necessary to design all types of bridges. Working closely with contractors in a diverse number of design projects allows us to further enhance construction methods and integrate them into our designs."

'Concrete as a whole continues to be the material of choice for most bridges.'

Those designs will no doubt include new concrete techniques, he adds. "Concrete as a whole continues to be the material of choice for most bridges. Precasting methods, segmental construction, ABC methods, and advances in concrete mixture proportions and higher strengths will result in more economical and durable structures with low maintenance."

For additional photographs or information on this or other projects, visit www.aspirebridge.org and open Current Issue.

Overview of the 2012 ASBI Durability Survey

by Brett H. Pielstick, Eisman & Russo Inc.

In the fourth edition of the American Segmental Bridge Institute's (ASBI's) *Durability Survey of Segmental Bridges*,[1] released in 2012, the performance of 363 segmental bridges is compared to that of 373,670 bridges built within the same period. The results were favorable, as discussed in this summary.

The study began by pulling the National Bridge Inventory (NBI) data from 2011, which included 604,426 bridge condition ratings throughout the United States. Since durability is a time-dependent function and segmental bridges first debuted in the early 1970s, the 2011 NBI data set was filtered to eliminate all bridges built prior to 1970. The new data set contained 373,670 bridges. To increase the accuracy of the segmental bridge data, the NBI data set was supplemented by inspection data received directly from participating states. Thus, condition ratings were provided for segmental bridges up to 2 years beyond that reported within the filtered NBI data set.

The modified NBI data set was then broken down into two bridge types as defined in the Federal Highways Administration's publication *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*.[2] Based on this breakdown, steel bridges built since 1970 were about 26% of all bridge types. Concrete bridge types make up 71% of the bridges built since 1970, with all other types making up the remaining 3%. Segmental bridges were then identified, separated from the data set for comparison, and supplemented and corrected with data from the ASBI master inventory list. The master list identified over 450 segmental bridges, of which 363 had condition ratings available within the 2011 NBI data set—which represents 0.11% of bridges in the data set (Fig. 1).

With the 2011 NBI data set broken down, an analysis was run on the filtered 2011 data set to identify structurally deficient and functionally obsolete bridges across all bridge types. This revealed that 24% of all bridges in the set were classified as structurally deficient or functionally obsolete, with 11% of bridges in the data set classified as structurally deficient and 13% classified as functionally obsolete. The next task was to identify the percentage of each bridge type represented in the overall 11% structurally deficient number. Steel bridges accounted for 43% of the structurally deficient bridges while, within this same data set, concrete structures (71% of the total bridges built) accounted for only 39% of the deficient bridges (Fig. 2). Timber bridges, while representing only 2.8% of the total bridges built, represent 17.5% of the structurally deficient bridges.

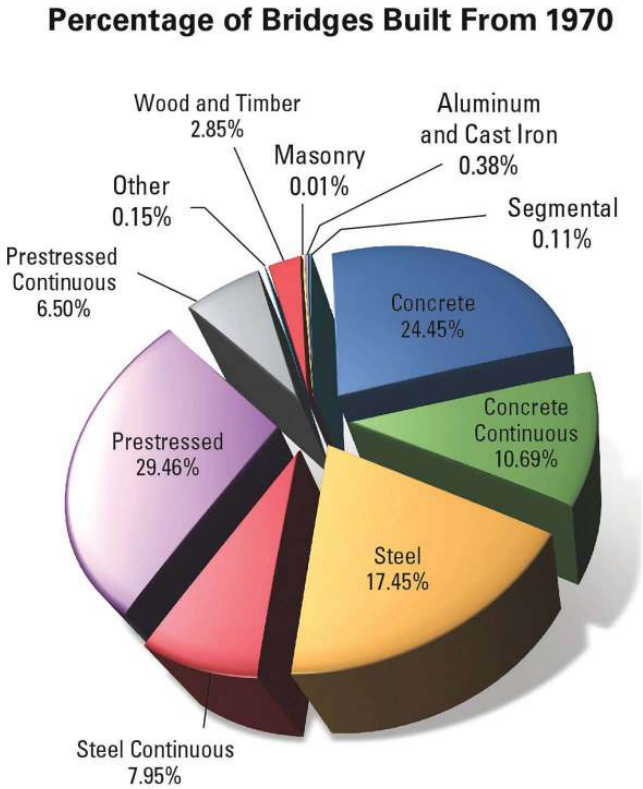


Figure 1. Percentage of bridges by type that were built since 1970.

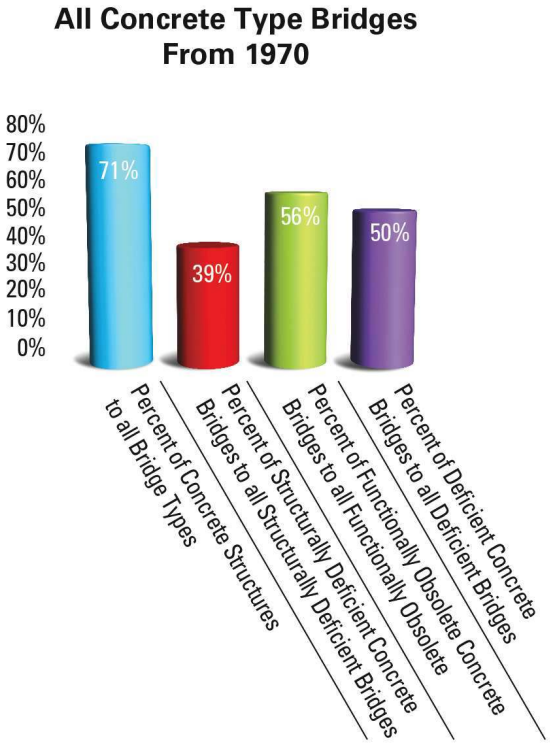


Figure 2. Concrete bridges by type that were built since 1970.

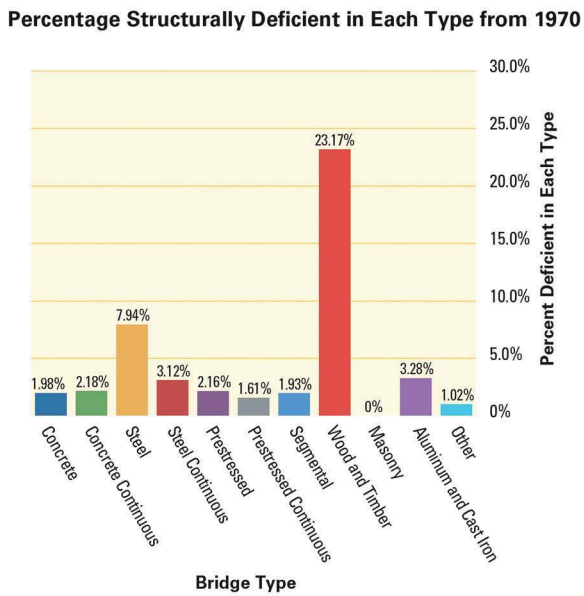


Figure 3. Percentage of bridges that were rated as deficient within each bridge type that were built since 1970.

These results clearly show that, while representing almost three quarters of the bridges built in the representative time frame, concrete bridges only represent slightly more than a third of the structurally deficient structures. Alternatively, steel structures represent a little more than a quarter of the bridges built in this period and represent nearly 45% of the structurally deficient structures. Segmental bridges, while representing only 0.11% of structures, represented merely 0.06% of structurally deficient bridges. The study further identifies comparable ratios for the functionally obsolete and overall deficient structures, with similar results showing superior performance among the concrete structures.

The report further broke down each bridge type as structurally deficient, functionally obsolete, and overall deficient. Figure 3 graphically depicts the percentage of each bridge type that is structurally deficient. To better illustrate this, the figure shows that for the concrete bridge classification built from 1970, 1.98% are classified as structurally deficient. From this chart it can also be seen that 23.17% of wood and timber bridges built from 1970 are structurally deficient. Again, in this analysis, all concrete structure categories outperformed their steel counterparts.

Segmental Bridge Performance

Looking specifically at segmental bridge performance, the report breaks down the 40 years of segmental bridge condition ratings into 5-year increments categorizing the performance averages in this period for superstructure, substructures, and deck (Fig. 4). With this information plotted, trend lines were applied to each of the components. From the slope of the trend lines, a projected service life of over 100 years was extrapolated based on past performances of these components.

In conclusion, the 2012 Durability Report shows that 98% of the segmental bridges have a condition rating of 6—"Satisfactory"—or better, which is consistent with the previous durability studies performed in 1994, 1999, and 2007. Based on their past performance, the service life of segmentally constructed bridges can be extrapolated at over 100 years utilizing slopes of the trend lines taken from the 5-year average declination of the superstructure, substructure, and deck condition ratings. Segmental bridges have also outperformed their bridge "peers" (defined as "bridges built with equal span range, capacity, age, and environmental condition") over the last 40 years. Concrete segmentally constructed bridges are the original accelerated bridge construction method, providing a quick and durable solution along with a best value bridging solution in many applications. Finally, as a byproduct, this report shows the superior performance of concrete structures in the area of durability based on the condition assessments found within the filtered NBI 2011 data set.

Reference

1. Pielstick, Brett H. and Leo Offredi. 2012. *ASBI Durability Survey of Segmental Bridges*. Fourth Edition, American Segmental Bridge Institute (ASBI), Buda, Tex.
2. Federal Highway Administration (FHWA). 1995. *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges.* Report No. FHWA-PD-96-01., FHWA, Washington, D.C.

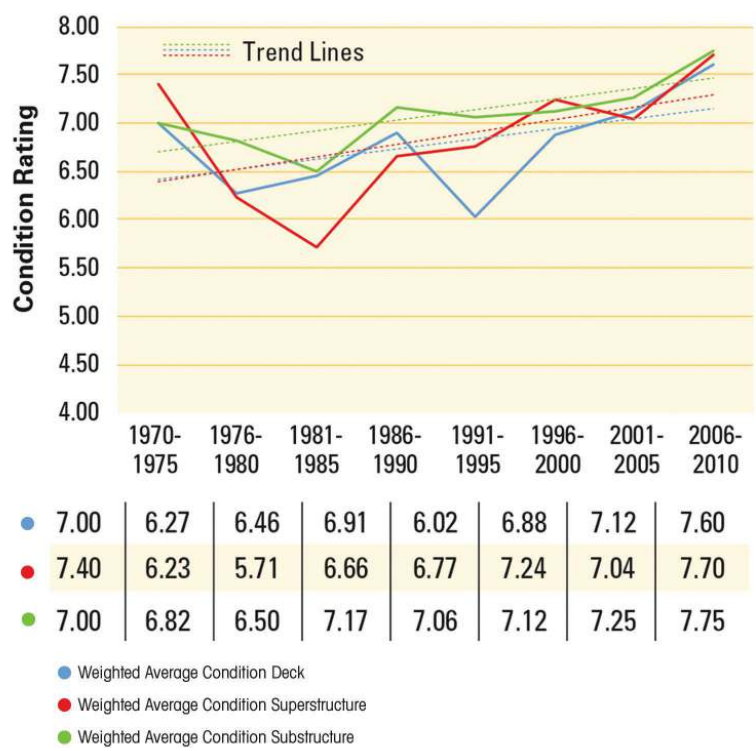


Figure 4. Component condition ratings of segmental bridges with trend lines.

Honolulu Rail Transit Project (Segments 1 and 2)

Improving the quality of life in a highly congested urban area

by Jose Rodriguez, FIGG

Rendering of typical urban section of Honolulu Rail Transit Project (Segments 1 and 2). All Renderings and Photos: FIGG.

The Honolulu Rail Transit Project is a 20-mile elevated rail line on the island of O'ahu that will connect west O'ahu with downtown Honolulu and the Ala Moana Center via the Honolulu International Airport. The system features state-of-the-art, electric, steel-wheel trains that will travel the entire route through 21 stations in 42 minutes. Trains will travel on steel rails and will be powered by a third rail. The $5.16 billion project will provide approximately 10,000 direct and indirect jobs per year.

Honolulu Authority for Rapid Transportation (HART) developed the project to improve mobility, enhance reliability, and address the island's increasing congestion. By 2030, the rail system will handle approximately 120,000 trips per weekday, reducing roadway traffic by about 40,000 vehicles. This will reduce overall commuting times and costs and enhance the quality of life for the residents of O'ahu. The western-most 10 miles of the project's guideway system are being completed through two design-build contracts.

Project Alignment

O'ahu's mountainous landscape and congested roadway system in this project segment make it difficult to build a new guideway. The project requires erection of the guideway on the shoulder or in the limited-width median of the existing 2- or 3-lane roadway for 7.5 miles. The route includes intersecting roads and left-turn pocket lanes in the median. The guideway alignment consists of various straight and curved sections with a minimum radius of 1100 ft for precast concrete sections and 800 ft for cast-in-place concrete sections. The maximum vertical profile grade is 5.5%. The contractor is using a plinthless design for the rail support, connecting the rails directly to the top of the guideway deck with fasteners and requiring superelevation in curved alignments.

Structural Solutions

To solve the challenges of material delivery, mobility, and erection of the structure that are posed by the project's congested, urban location, the guideway was designed to be built from above using precast concrete box girder segments. The contractor is precasting the segments at a site approximately 5 miles away while also building the substructure elements on site. The project consists of 438 total spans, 430 of which are made of precast concrete and range in length from 68 to 151 ft. The remaining eight are made of

profile

HONOLULU RAIL TRANSIT PROJECT / HONOLULU, HAWAII

BRIDGE DESIGN ENGINEERS: Segmental Superstructure: FIGG Bridge Engineers, Tallahassee, Fla., and Substructure: HNTB, Lake Mary, Fla.

PRIME CONTRACTOR: Kiewit Infrastructure West Co., Vancouver, Wash.

POST-TENSIONING SUPPLIER: Schwager Davis Co., San Jose, Calif.

OTHER MATERIAL SUPPLIERS: Formwork: EFCO Corp., Des Moines, Iowa, and Bearings: D.S. Brown Company, North Baltimore, Ohio

PRECASTER: Kiewit Infrastructure West Co., Vancouver, Wash.

CONSTRUCTION DRAWING PREPARATION: FIGG Bridge Engineers, Tallahassee, Fla.

cast-in-place concrete with span lengths ranging from 204 to 342 ft.

The substructure consists of typical 30-ft-tall single columns that are 6 to 7 ft in diameter and have flared capitals. The columns are supported by single drilled shafts that are 7 to 8 ft in diameter. There are some locations that require offset piers or straddle bents to clear ground-level obstructions. There are also hammerhead piers at station locations. Both the drilled shafts and columns use a typical 4-ksi compressive strength concrete. Because of design considerations, some of the reinforcement in the substructure consists of Grade 75 steel, while the majority used Grade 60.

At each station location, artistic diagrams are added to columns using formliners. These celebrate the local culture by providing a visual representation of the history of the community. Each diagram is a pictorial description of the cultural history of the surrounding community.

The typical guideway superstructure consists of a precast concrete trapezoidal box girder section that is 30 ft wide and 7 ft 2 in. deep. Some areas require a section that is 17 ft 3 in. wide and 7 ft 2 in. deep. The cast-in-place sections are also 30 ft wide but vary in depth from 17 ft at the piers to 7 ft 2 in. at the ends. The top slab deck varies in thickness from 8 to 15 in.

The 11-ft length of the precast concrete segments utilizes the maximum legal transporting limits on the roads, eliminating almost 10% of the precast concrete segments that would have been required were typical 10-ft lengths used. The precast concrete segments are made of 6.5-ksi compressive strength concrete that allowed for quick stripping of the forms so that production was maximized. Epoxy-coated reinforcing steel was utilized in portions of the superstructure.

Precast concrete segments, 30 ft wide and weighing 36 tons, were cast in a facility that was approximately 5 miles away, then delivered to the site for erection on underslung girders.

The precast concrete spans are longitudinally post-tensioned typically using 3 or 4 external tendons that consist of twelve to twenty-seven 0.6-in.-diameter strands. The tendons are anchored in the expansion joint segments and all deviate in the span at about the third points of the span. The cast-in-place spans are longitudinally post-tensioned using internal tendons with twelve 0.6-in.-diameter strands. The top slab is transversely post-tensioned using tendons with two and four 0.6-in.-diameter strands.

Formliners were used on station columns to create a pictorial celebration of local neighborhood folklore.

HONOLULU AUTHORITY FOR RAPID TRANSPORTATION (HART), OWNER

ERECTION GIRDER DESIGN: Somerset Engineering Group, Bellingham, Wash.

GEOTECHNICAL CONSULTANT: Shannon & Wilson Inc., Seattle, Wash.

GENERAL ENGINEERING CONSULTANT: Ch2M Hill, Englewood, Colo.

CE&I WEST: PGH Wong, San Francisco, Calif.

BRIDGE DESCRIPTION: Ten-mile-long, mainly span-by-span precast concrete segmental box-girder superstructure with external post-tensioning

STRUCTURAL COMPONENTS: Segmental box girder superstructure, cast-in-place columns, and drilled shafts

BRIDGE CONSTRUCTION COST: $850 million

All tendons are inside air-tight tested plastic ducts that are grouted using prepackaged thixotropic material to provide a durable system.

Construction Challenges

The eastern-most 7.5 miles of the first two project segments are required to be erected in the middle of roadways that either have a small median or no median at all. To ensure all roadway lanes are open during peak hours, the roadways were widened to the outside to provide a work zone in the median that allowed for the drilling of shafts and constructing the substructure.

To maximize safety and mobility while rapidly constructing the project, the 5238 segments were precast while the road widening and substructure construction were in progress. The segments are erected using the span-by-span method with underslung girders supported on temporary pier brackets that rest on the top of the columns and are leap-frogged forward as the girders are launched. The structural system uses simple spans of match-cast segments that only require epoxy at the joints.

The structure is designed to support a crane to eliminate any traffic disruption due to this equipment use. The segments are typically delivered on the ground at night, lifted by the deck-mounted crane, and placed onto the underslung erection girders. Then, the external post-tensioning is installed and stressed, the temporary girders are launched to the next span, and the crane moved forward to begin the erection of the next span. This operation is typically a 1- to 2-day cycle per span.

The longer spans over the highway are being built in balanced cantilever using cast-in-place concrete placement on form travelers. Most of these spans take advantage of integral twin-wall piers to eliminate temporary falsework. This overhead construction allows uninterrupted traffic flow below.

Project Status

As of April 2015, the precast yard was using at least 13 casting cells, which are being turned every day. The concrete mixture proportions allow for stripping of the forms every day to cycle the segments as quickly as possible. Currently, about half of the segments have been cast and are either erected or in storage at the casting yard. Approximately 110 spans (25%) have been erected by the span-by-span erection method. Construction of the balanced cantilever spans has recently begun, with three pier tables built and the first few typical segments cast on one pier.

These first two project segments, totaling 10 miles, are scheduled to be completed and in operation by the end of 2018, and the entire 20-mile system is slated to be completed by 2020.

First 110 spans completed in the East Kapolei section of the project were built using the span-by-span erection method with a deck-mounted crane.

Jose Rodriguez was the on-site aerial structures design manager for FIGG.

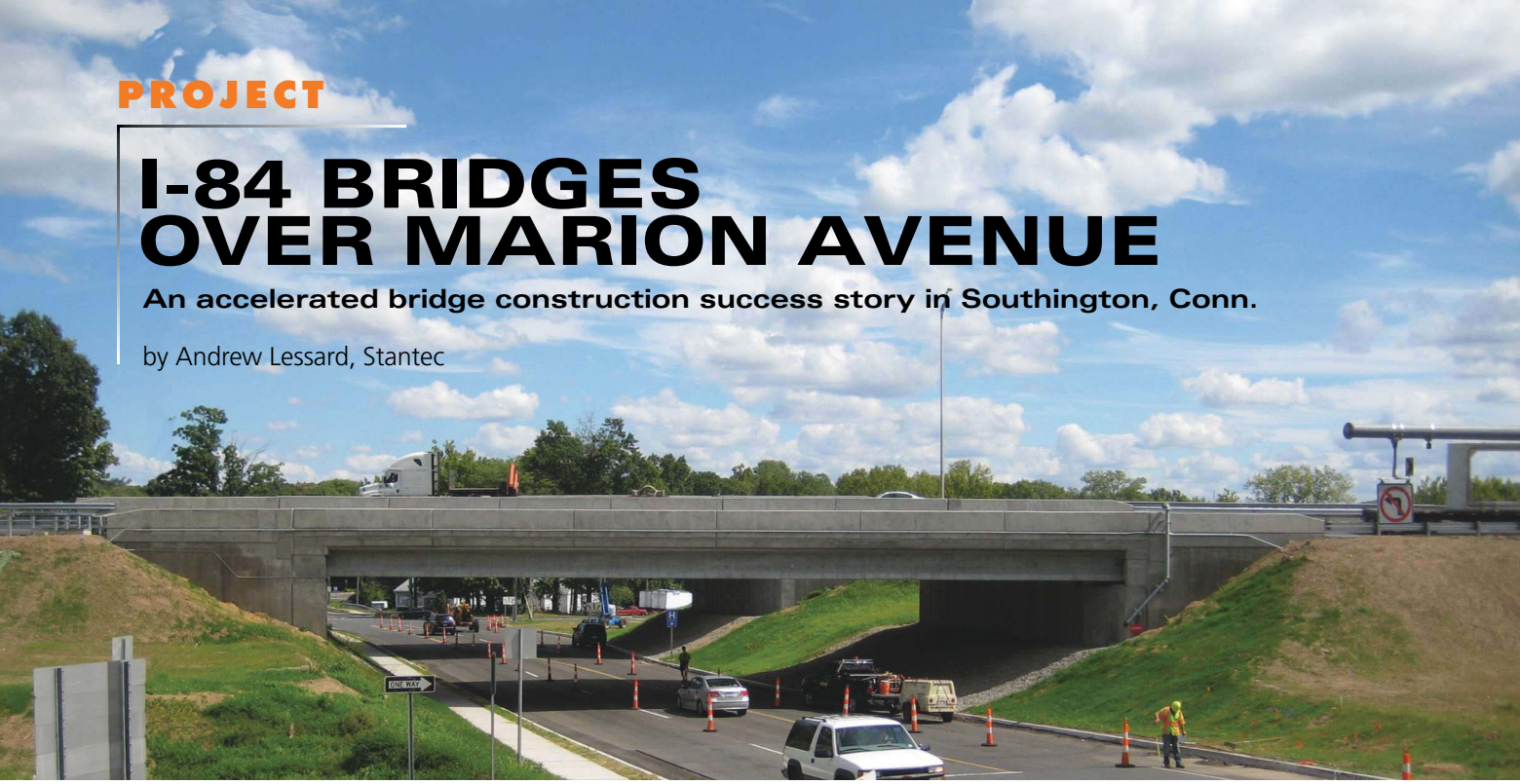
For additional photographs or information on this or other projects, visit www.aspirebridge.org and open Current Issue.

PROJECT

I-84 BRIDGES OVER MARION AVENUE

An accelerated bridge construction success story in Southington, Conn.

by Andrew Lessard, Stantec



Completed project. All photos: Andrew Lessard, Stantec.

For the last few years, state transportation departments across the country have begun using accelerated bridge construction (ABC) to minimize the sometimes-lengthy disruption to the driving public that large construction projects can have. In 2014, the Connecticut Department of Transportation (CTDOT) realized it had an ideal opportunity to employ ABC on a large-scale construction project: the reconstruction of two structurally deficient highway bridges along Interstate 84 (I-84) in Southington, Conn. The resulting effort reconstructed the two heavily traveled bridges in just a single weekend, avoiding what would have been months—or even years—of construction and associated disruption to traffic.

The Marion Avenue Bridges

The two bridges, which run parallel to each other on the eastbound and westbound sides of I-84, were built in 1963 and each carry three lanes of traffic over Marion Avenue. The bridges consisted of a similar cross section but slightly different span lengths. In 2008, the state identified the bridges as structurally deficient due to a deteriorating superstructure and moved them into a priority position for reconstruction.

After determining that the substructure was still in good condition and suitable for reuse, the preferred scope of rehabilitation was to reuse the existing concrete abutments and bridge seats. A design plan was developed based on New England bulb-tree (NEBT) 1200 prestressed concrete beams supporting a cast-in-place concrete deck.

As the team got further into design, however, CTDOT began exploring the idea of using an ABC approach to reconstruct the bridges and minimize the significant impact that this type of project would have on traffic. The I-84 Bridges over Marion Avenue project stood out as a good candidate for ABC thanks to two main factors: schedule and site.

Schedule

Because all three lanes of traffic in each direction of I-84 would need to stay open during construction, traffic congestion became a major concern. With conventional construction, accommodating that requirement would have entailed an overbuild of the bridges and widening of the existing abutments through four stages of construction. Each stage would have been months long, with significant adverse effects to I-84. With ABC, that duration would be condensed into the period of a few days.

Site

The bridges comprise the overpass component associated with a diamond interchange, meaning it would be relatively simple to route traffic around construction using the interchange off-

profile

I-84 BRIDGES OVER MARION AVENUE / SOUTHINGTON, CONNECTICUT

OWNER'S LIAISON ENGINEER: Close, Jensen and Miller P.C., Wethersfield, Conn.

BRIDGE DESIGN ENGINEER: Stantec Consulting Services Inc., New Haven, Conn.

PRIME CONTRACTOR: Northern Construction Service LLC, Weymouth and Palmer, Mass.

HEAVY LIFT CONTRACTOR: Marino Crane/Barnhart Northeast Inc., Middletown, Conn.

PRECAST BEAM DESIGN: Eriksson Technologies, Tampa, Fla.

CONTRACTOR ENGINEERING: Steere Engineering Inc., Warwick, R.I.

and on-ramps. This also meant that there was a large work area for the contractor to construct the new bridges offline with no impacts to mainline traffic.

Before construction began at the site of the I-84 Bridges over Marion Avenue.

Moving to ABC

The switch to an ABC approach, however, meant the project team had to revise the design—which was now at more than 60% complete and tailored to conventional construction techniques—to ensure it was feasible, while maintaining the proposed project schedule. The original design included NEBT 1200 concrete beams supporting an 8.5-in.-thick, cast-in-place concrete deck with epoxy-coated reinforcement and a 3-in.-thick bituminous overlay. The deck concrete had a design compressive strength of 4 ksi, while the beam concrete was 8 ksi. The existing curb-to-curb width of each bridge was to be maintained at 52 ft 10 in. and the span lengths for the new structure were also the same as those for the existing bridge.

The NEBT 1200 beams were chosen as they allowed the existing vertical clearance over Marion Avenue to be retained without requiring adjustments to the crest vertical curve on I-84 where the bridges were located. The beams also allowed for reuse of the existing bridge pedestals. Because the abutments were to be reused, the existing curved horizontal alignment was also to be maintained, and was accommodated by varying the fascia overhangs.

Under the new plan, the contractor would remove both of the existing bridges and replace them with new, prebuilt structures using two sets of self-propelled modular transporters (SPMTs). The design team was able to maintain the original framing concept while adjusting the specifications and construction details to accommodate the necessary phasing. Some of the specific adjustments included the following:

- Allowing more leeway in construction details and dimensions due to potential variations that could result from lifting the bridges using the SPMTs. For example, the interface between the bridge superstructure and abutment backwalls was revised to allow clearance for the lateral movement of the existing and proposed bridges.
- Incorporating the reconstruction of the in-line wingwalls, which required reconstruction to the current CTDOT standard shape, into the initial stages of the project. Concrete parapet closure placements using high-early strength concrete were also detailed to tie the bridge and wingwall parapets together after the new bridges were installed.
- Detailing precast concrete approach slabs with high-early-strength concrete closure placements to minimize the time needed for their installation.
- Ensuring that the site was suitable for the heavy movement patterns of the SPMTs supporting the bridges, and that equipment was available to perform the moves on the site.

Precast concrete approach slabs, cast-in-place concrete deck, and parapet closure placements being prepared for final paving for the I-84 Bridges over Marion Avenue.

CONNECTICUT DEPARTMENT OF TRANSPORTATION, OWNER

PRECASTER: Northeast Precast Products LLC, Cressona, Pa.—a PCI-certified producer

BRIDGE DESCRIPTION: Twin 56.67-ft-wide, three-lane bridge structures composed of prestressed concrete beams supporting a cast-in-place concrete deck with an eastbound span of 102 ft and a westbound span of 103 ft.

STRUCTURAL COMPONENTS: Each bridge is comprised of 10 PCEF 47 prestressed concrete beams spaced at 5.67 ft on center, with a composite 8.5-in.-thick, cast-in-place concrete deck and 16.5-ft-long, precast concrete approach slabs

BRIDGE CONSTRUCTION COST: $7 million (for both bridges)

- Ensuring each prefabricated bridge, which weighed over 2 million pounds, could withstand the structural loads resulting from being lifted via the SPMT approach. Additional reinforcement was added to the deck at the SMPT pick points to accommodate negative moment. The contractor also established a number of monitoring points and strain gauges in the prestressed beams to check for twisting, rotating, or other movement.
- Providing enough space to accommodate the large SPMT equipment. The revised specifications, therefore, detailed the gore areas between the highway and off ramps to ensure the SPMTs can get into and out of the project site and move around as needed.
- Developing regional and local detour routes that directed traffic away from the project site during the weekend closure.

Moving the new eastbound bridge into place.

Extensive work was done by the contractor developing and improving upon the ABC concept.

Once the project was awarded, extensive work was done by the contractor developing and improving upon the ABC concept. Due to timing and availability constraints, the contractor elected to substitute the nearly equal Prestressed Concrete Committee for Economic Fabrication (PCEF) 47 beams in lieu of the NEBT 1200 beams. The contractor also developed a comprehensive work plan with working drawings including the following:

- SPMT design and configuration—two sets of SPMTs were used. One set was used on the eastbound side to remove the existing and install the new bridges, while the other set was used on the westbound side for new bridge installation only. The existing westbound bridge was

demolished in place onto Marion Avenue. The design of the SMPT system was challenging due to the weight of the load, as well as the height at which the load was to be carried (30 ft above grade).

- Demolition and erection procedures.
- Temporary abutments to support the new bridges that were being constructed offline and for support of the existing eastbound bridge after its removal using SPMTs.
- Travel path and bridge staging area site improvements beyond what was envisioned on the design plans to ensure the ground would support the 3-million-pound weight of each SPMT loaded with the a new bridge.
- Three-dimensional, finite-element analysis of the bridges subjected to transport loadings.
- A comprehensive contingency plan, including provisions for maintaining backup equipment and staff on-site during the weekend closure.

Innovation Success

With a plan in place, the contractor began construction of the new bridge superstructures in early 2014. All six lanes of traffic were maintained while the new superstructures were constructed off-line and the site was prepared for the move. By June 2014, the team was ready to make the move, settling into the weekend of June 28 and 29, 2014. In just under 46 hours—12 hours ahead of schedule—the two bridges were lifted into place and reopened to traffic with minor disturbances to the local flow of traffic.

As they were moved into place, traffic was managed through a combination of detour routes and bypass routes around the site. I-84 through traffic was rerouted well in advance of the project site to the east along preferred detour route of I-691 and 1-91. I-84 traffic that did not use the detour was channeled to one lane on the existing Marion Avenue off-ramps and immediately back onto I-84 after bypassing the site. Local traffic on Marion Avenue was routed around the site as well. These proposed traffic pattern changes were heavily communicated to the public by the state weeks in advance, through such media as public websites, press releases and media coverage, traffic alerts, webcam videos, Intelligent Transportation System message boards, and detour signage.

The ABC approach ultimately saved months of construction time, condensing major traffic impacts from what could have been years into a period of one weekend.

The ABC approach ultimately saved months of on-site construction time, condensing major traffic impacts from what could have been years into a period of one weekend. What's more, this approach created a safer work zone as traffic was entirely closed to the construction site. The state was thrilled with the results and now considers the I-84 Bridges over Marion Avenue project a model for continued ABC efforts across the state.

Perhaps most importantly, this project has allowed the state to make much needed improvements to critical infrastructure within a reasonable budget and schedule. These kinds of improvements create safer, more-reliable infrastructure to serve residents and the surrounding community without imposing the inconveniences of lengthy construction schedules.

Andrew Lessard is a senior associate at Stantec in New Haven, Conn.

For additional photographs or information on this or other projects, visit www.aspirebridge.org and open Current Issue.

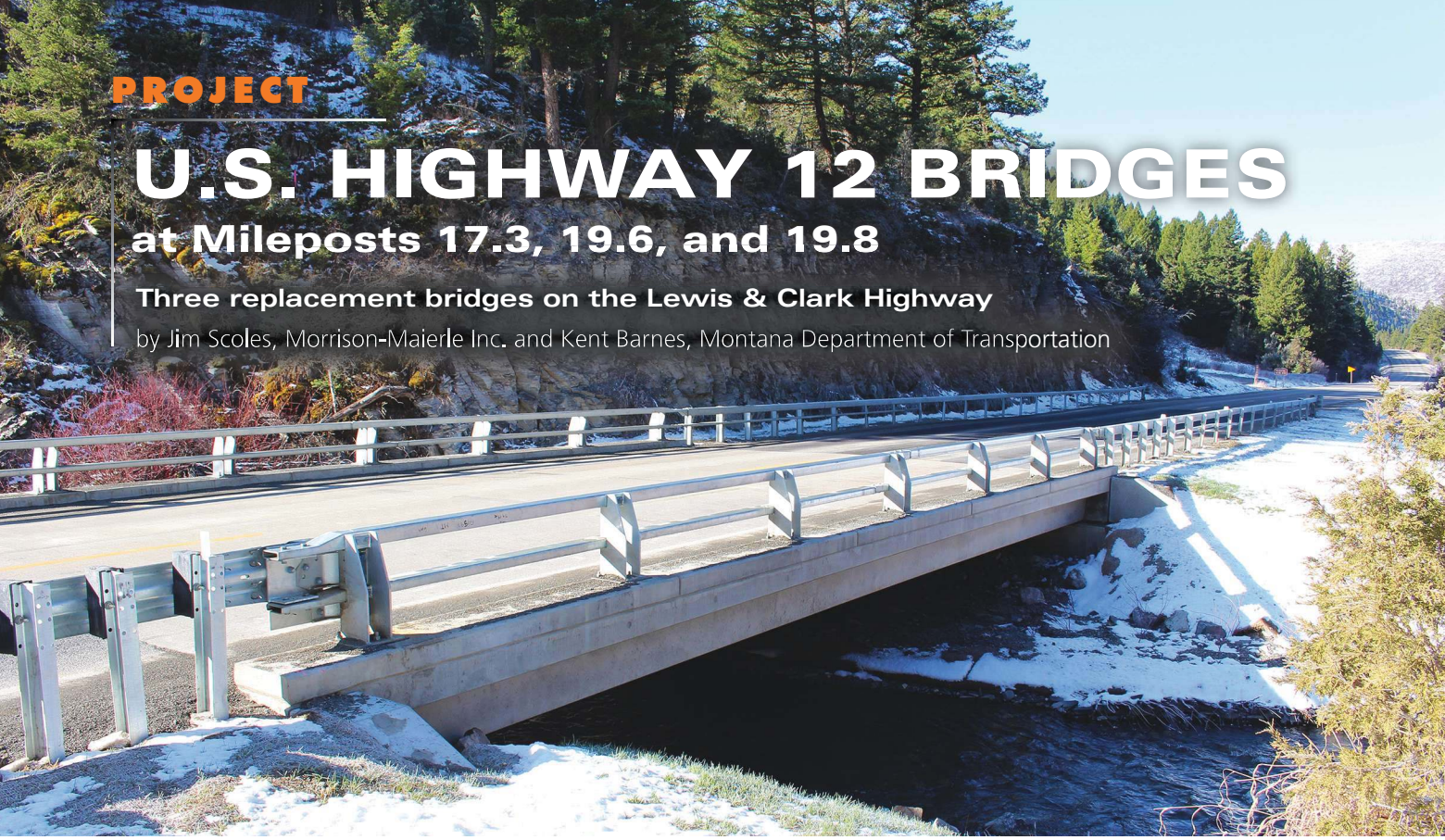
PROJECT

U.S. HIGHWAY 12 BRIDGES

at Mileposts 17.3, 19.6, and 19.8

Three replacement bridges on the Lewis & Clark Highway

by Jim Scoles, Morrison-Maierle Inc. and Kent Barnes, Montana Department of Transportation



U.S. Highway 12 runs along the narrow floor of Deep Creek Canyon in Montana. Three bridges were replaced due to significant scour and severe erosion of the highway embankment. Photo: Sheila Habeck.

In July 1805, the Lewis and Clark expedition passed by the confluence of Deep Creek and the Missouri River in what is now Broadwater County, Mont. Less than two river miles upstream, William Clark mapped the area as "Yorks 8 Islands," and Meriwether Lewis noted the beavers' role in creating the islands. The mouth of the steep, rocky canyon through the Big Belt Mountains lies 12 miles due east as the raven flies.

The Lewis & Clark Highway—also known as U.S. Highway 12—runs along the narrow floor of Deep Creek Canyon. The 24-ft-wide, two-lane highway between Townsend and White Sulphur Springs was constructed in the 1930s with timber bridges at eight locations where the creek and the highway intersect. High flows during spring runoff in 2011 resulted in significant scour of the bridges and severe erosion of the highway embankment. The flooding required emergency response to prevent structural loss and ultimately triggered a project to replace the bridges in this rural area of southwestern Montana.

In the first phase of the project, the Montana Department of Transportation (MDT) replaced three timber bridges with new 54-ft-long, single-span, precast concrete structures using accelerated bridge construction (ABC) techniques. Initially, even with ABC, complete road closures were not considered an option because of the required two-hour, 120-mile detour around the Big Belt and Bridger mountain ranges.

Limited space through the Deep Creek Canyon required staging materials and equipment on the roadway during the weekend closures. Photo: Andy Cullison.

profile

U.S. HIGHWAY 12 BRIDGES AT MILEPOSTS 17.3, 19.6, AND 19.8 / BROADWATER COUNTY, MONTANA

BRIDGE DESIGN ENGINEER: Morrison-Maierle Inc., Helena, Mont.

PRIME CONTRACTOR: Dick Anderson Construction, Helena, Mont.

PRECASTER: Oldcastle Precast, Spokane, Wash.—a PCI-certified producer

GEOTECHNICAL CONSULTANT: SK Geotechnical, Billings, Mont.

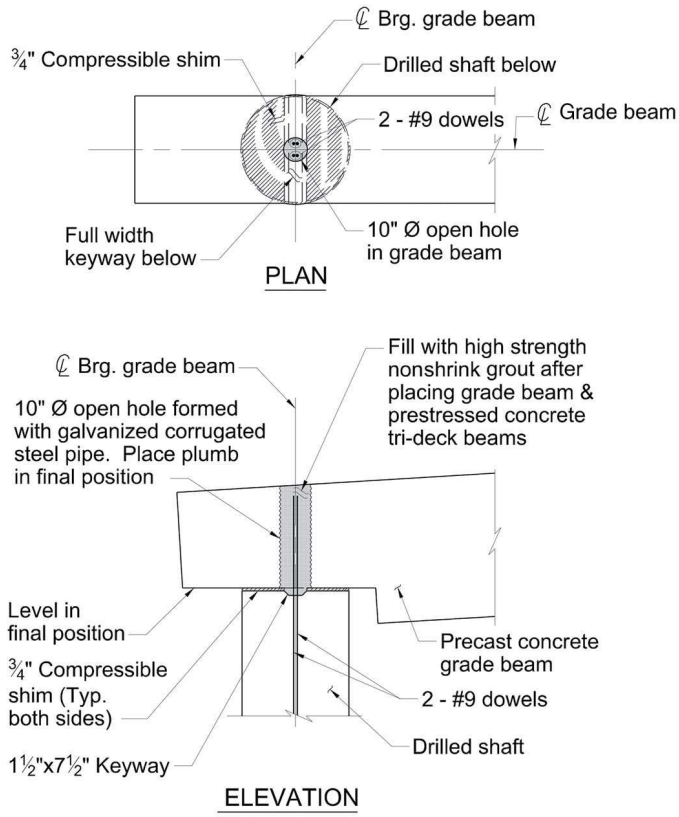
Following preliminary layout of temporary detours, and further consideration of all project elements, ABC construction began to emerge as the favorite for the design team. Several reasons were identified for this shift:

- Poor sight distance at two of the sites raised safety concerns for construction workers and the traveling public.
- One site had only enough room for a single-lane detour.
- Geometric constraints in the canyon provided little room to stage equipment.
- The construction of detour bridges increased the project duration 4 to 5 weeks per bridge.
- Riparian habitat would have to be cleared and even filled in some places to provide the temporary detours.
- Conventional construction with detour was estimated to cost $610,000 per bridge, nearly twice as much as the ABC at $370,000 per bridge, without consideration of user costs.

Minimizing construction time required each bridge to be designed as a complete, modular concrete system. Precast concrete bridge superstructure elements consisting of three stem units with integral deck (known locally as tri-deck) were chosen to achieve project objectives. The five tri-deck prestressed beam lines were longitudinally jointed to create the superstructure and riding surface. The new bridges are simple spans of 54 ft and match the roadway approaches with two 12-ft-wide lanes and 2-ft-wide shoulders.

Two of the bridges are located on tangent roadway alignments. One of these bridges has a normal crown cross section, while the other maintains a 2% transverse slope because of its proximity to a superelevation transition. The third bridge is located on a 1280-ft-radius horizontal curve with a constant 6%

Construction workers set the first precast concrete tri-deck. A vertical rock face on the left and the pristine habitat on the right would have been the only available area for a detour bridge if accelerated bridge construction methods were not used. Photo: Andy Cullison.



This pin connection allows live load rotation about the longitudinal axis of the structure and provides resistance to rotation about the transverse axis. Figure: Morrison-Maierle.

MONTANA DEPARTMENT OF TRANSPORTATION, OWNER

BRIDGE DESCRIPTION: Three single-span, 54-ft-long, 31-ft 8-in.-wide, modular bridges with prestressed concrete superstructure

STRUCTURAL COMPONENTS: Five prestressed concrete tri-deck beams per bridge; precast concrete grade beams, wingwalls, and backwalls; and 36-in.-diameter drilled shafts

BRIDGE CONSTRUCTION COST: $203/ft^2

AWARDS: 2015 Project of the Year by the Rocky Mountain Chapter of the American Public Works Association, and 2015 Engineering Excellence Award, Structural Systems Category, by the American Council of Engineering Companies of Montana

Due to critical road closure time limitations, all bridge components were assembled at the precasting plant to check for fit prior to shipment. Photo: Andy Cullison.

Crews work through the night preparing for bridge end backfill placement and compaction. Photo: Andy Cullison.

transverse slope. An extra 4 in. was added to the width to provide the minimum shoulder width along the full length of the bridge while maintaining a tangent bridge alignment to facilitate ABC construction. The extra width was also added to the other structures to simplify design and fabrication.

Drilled shafts were selected for the foundation over piles or spread footings because of the sloping bedrock at each bridge site. The 3-ft-diameter drilled shafts were socketed into the bedrock on each side of the road just beyond the existing shoulder. Placing the drilled shafts just off the shoulders allowed the construction to occur under single-lane traffic during the day several weeks in advance. When the road was closed, the contractor quickly excavated to the buried drilled shafts, cleaned the surfaces, connected the steel reinforcing using embedded mechanical couplers, and placed shims before setting the transverse spanning, precast concrete drilled shaft cap beams, referred to as grade beams.

A design challenge was to create a drilled shaft-to-grade beam connection that added no construction time. The timeframe for bridge construction was not long enough to accommodate separate grout cure times for the grade beam connection and the superstructure keyway connection. Additionally, the connection had to handle the deflection and rotation of the grade beam cap under dead and live loads as the span between the drilled shafts was long.

A typical fixed-condition moment connection was considered with grouted bars extending into the cap. However, the dead load and live load from the beams on the transverse cap created a large moment that could not be taken through the drilled shafts without significantly increasing their size over what was needed for axial loads. Moreover, void sizes and time required for grouting the connection was determined to make a full moment connection impractical. Other options considered were pot or disc-type bearings, which were not preferred because the connection was buried.

The challenges of this connection were addressed by using a pin connection with a keyway at the top of the drilled shaft. This design allowed the tri-deck prestressed concrete beams to be set immediately after the grade beams were placed on the drilled shafts, and allows grouting after the system is fully assembled. The keyway allows live load rotation about the longitudinal axis of the structure and provides resistance to rotation about the transverse axis.

The precast, conventionally reinforced 3-ft-wide concrete grade beams step from a 4 ft depth between the drilled shafts to a 2 ft 8 in. depth above the drilled shafts. The step in the grade beam allowed the connection to be above the anticipated ground water level, eliminating the need to dewater for grout placement.

The grade beams contain a 10-in.-diameter through void at each of the drilled shaft locations. Four No. 9 steel reinforcing bars extend from the top of each of the drilled shafts into the void. The connection was designed to be grouted after the tri-deck beams were placed. This approach saved time by allowing the superstructure longitudinal keyway and grade beam connection grouting to be completed concurrently.

Once the tri-deck beams were in position, the connections between the concrete grade beams and drilled shafts were grouted with quick-setting, low exothermic epoxy grout as the tri-deck

beams were leveled, welded, and painted to protect the steel ties at 5 ft on center and the continuous keyways between the beam flanges were grouted.

The top flange of the tri-deck beams was designed to be the wearing surface of the bridge deck. To avoid the need for asphalt overlay, the top flange thickness varied from 8.25 in. at midspan to 10 in. at the abutments to account for camber in the beams.

In order to help ensure the contractor's success, the project contract required assembly of all precast concrete components for each bridge at the precasting plant, after which they were checked for fit-up prior to shipment. During the dry-fit, the precast concrete grade beams were placed on temporary concrete footings to match the grades and cross slopes of their final positions. The exercise eliminated fit-up issues and greatly reduced the contractor's risk of exceeding the contract time requirements.

The contract schedule for the weekend closures of the Lewis & Clark Highway began at 6 p.m. on Friday for each bridge, and the road reopened at 7 a.m. the following Monday. A $38,000/day incentive/disincentive clause was included in the contract based on each hour. The ABC methods used required a detailed, step-by-step schedule for every hour of the allowable 61-hour period.

This carefully choreographed sequence of events was perfected as the project progressed. Each of the three bridges opened ahead of schedule, and actual closure times shortened as the contractor became more familiar with the process. The first structure was completed and the road opened to traffic at 3 a.m. on Monday, the second at 3 p.m. on Sunday; and the third at 1 p.m. on Sunday.

Jim Scoles is chief bridge engineer for Morrison-Maierle Inc. in Helena, Mont. Kent Barnes is Bridge Bureau chief for the Montana Department of Transportation in Helena.

For additional photographs or information on this or other projects, visit www.aspirebridge.org and open Current Issue.

Sustainability

Project activities are anticipated to enhance habitat characteristics, channel morphology, and general stream conditions through the installation of longer single-span bridges. Design elements were included to re-establish stream channel function through the crossings by providing access to overbank floodplain within the crossing and incorporating stream bank stabilization and re-vegetation.

Eliminating the need for temporary detours and the associated temporary fills resulted in significantly reduced impacts to the riparian environment and fisheries in the vicinity of each bridge. The impacted areas could have been rehabilitated, but it would have been years before pre-project conditions were again realized.

Project construction took place in the summer during low-flow conditions and a short window of time, that is, three consecutive 61-hour work periods. The contractor chose appropriate methods to isolate the areas where in-stream construction was required. These methods ensured that flowing water did not contact construction areas, and increased levels of sedimentation and turbidity were kept to a minimum.

Design elements were included to re-establish stream channel function through the crossings by providing access to overbank floodplain within the crossing and incorporating stream bank stabilization and re-vegetation. After a long Montana winter, these design decisions are already helping re-establish the native plants along the stream banks. Photo: Sheila Habeck.

EDITOR'S NOTE

This article was drawn from a presentation at the 2014 Accelerated Bridge Construction Conference. The Accelerated Bridge Construction-University Transportation Center has given permission to publish it in this periodical.

WORKING TOGETHER TO CREATE THE FUTURE

Sharing new technology through PCI Bridge TechnoQuests

by William Nickas, Precast/Prestressed Concrete Institute, and John Dick, J. Dick Precast Concrete Consultant

The cement and concrete industries have made remarkable advancements during the past 20 years or so. The precast concrete industry has greatly benefitted from these improved technologies, and has also contributed to advancing the construction industry.

A trade association or technical institute is often the entity called on to manage the change for an industry. It is relied upon to galvanize the industry's varied goals and objectives, since an industry serves many markets and clients. Through the work of the association, a focused message can be refined, presentations created, questions anticipated and answered, experienced professionals mobilized, and the technology broadcast to the end user.

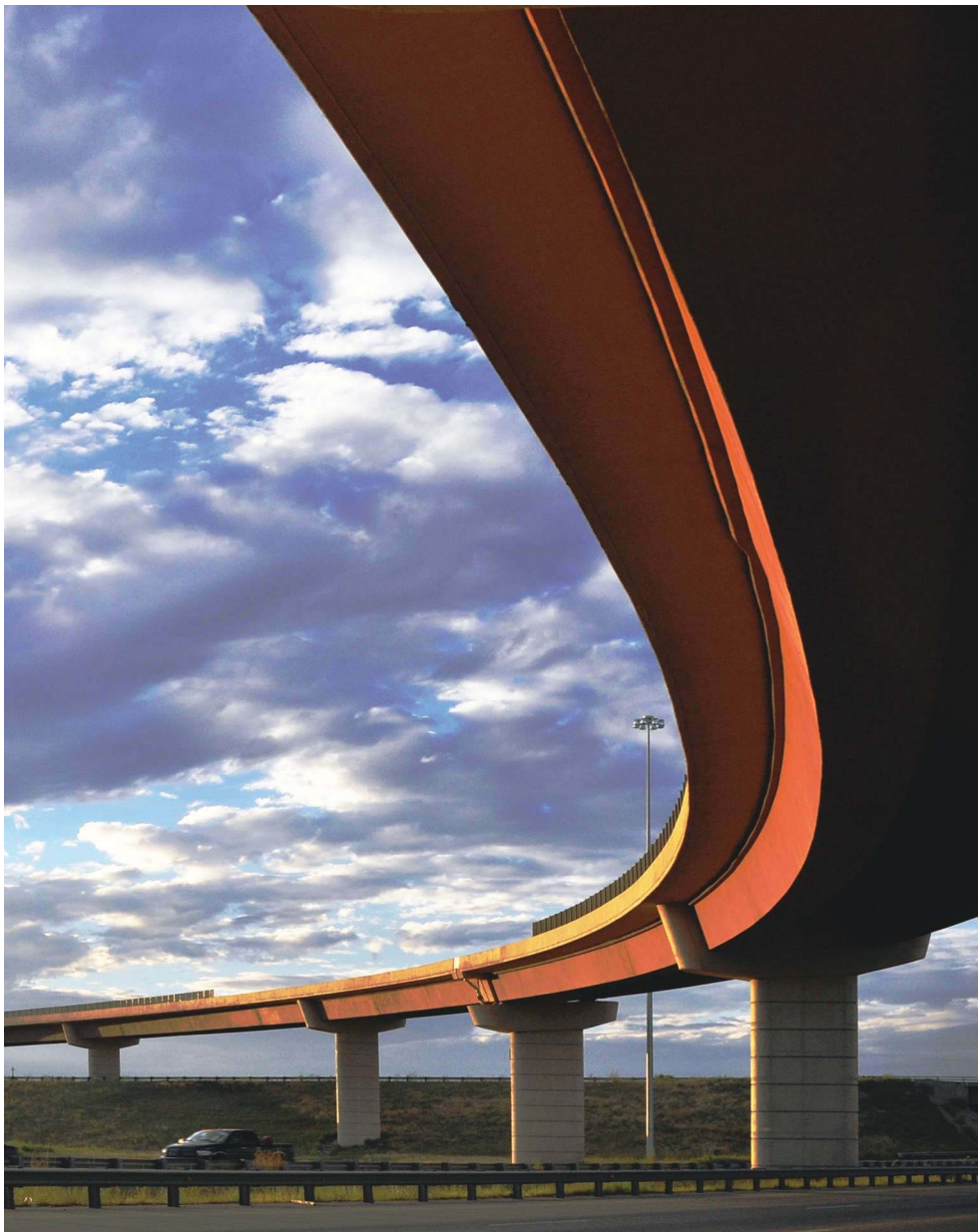
A new bridge type is being offered to the design community. In a few areas, horizontally curved, precast concrete, trapezoidal box-girder bridges are being built economically with spans approaching 300 ft. They are competing very effectively on a first-cost basis against other alternative systems. Furthermore, these U-girders, as they are also known, feature outstanding aesthetic appeal and will result in exceptional long-term, low-cost performance.

This article focuses on the way the precast concrete industry, with the aid of the Precast/Prestressed Concrete Institute (PCI), has contributed to this advancement. PCI is educating its members concerning the new opportunity. It is an entirely new market for the precast, prestressed concrete industry. PCI has refined the technology, packaged it in drawings and descriptions, presented it to groups of end users, and is conducting on-site learning experiences called TechnoQuests.

Exploring the Beginnings

The current work began in 1993 when PCI took note of a potential new market for the precast concrete industry. The idea resulted from a tour of a project underway in Denver, Colo. A horizontally curved girder was being precast on site because it was too large and heavy to cast in a plant and then transport. After casting, the girder was lifted onto the piers. After that, the Colorado Department of Transportation (DOT) continued development of the concept.

They refined it to encourage production in established precast concrete manufacturing plants, a step considered important to compress schedules, reduce cost, and improve quality. It would take a decade; a Colorado prestressed concrete industry with vision; collaboration between the owner,



Flyover Ramp A at I-70 and SH58 was built in 2008, the sixth plant-produced curved U-girder bridge built in Colorado up to that time. Photo: Summit Engineering Group.

and contractors, specialty designers, as well as the precasters to refine the solution and make it cost-effective and constructable. PCI continued to stay abreast of developments, waiting for just the right timing to promote this system to a wider audience.

Putting Wheels in Motion

In 2004, at a PCI Bridge Producers Committee meeting, a Denver precaster told the audience about the new project to fabricate horizontally curved precast concrete U-girders.

Following that meeting, articles were written and published, speakers invited to conferences, and committee work commenced. In 2005, through its Committee on Bridges, PCI established the Subcommittee on Curved Concrete Bridges. Their work, a state-of-the-art report, was published by PCI in 2012.

In 2010, several PCI-certified producer-member companies in Florida (specifically Coreslab Structures, Dura-Stress, Gate Precast, and Standard Concrete Products) pooled resources and teamed with the Summit Engineering Group and PCI to develop a set of 20 drawings for a family of "standard" U-girders. The package of standard plans included framing plans, erection details and sequences, splicing details, and typical reinforcement. The producers who spearheaded this effort are in PCI's "Zone 6" geographical region in the southeast United States. The plans are now referred to as the PCI "Zone 6 Concept Plans" and are available as PDF or CAD files. With the plans, a designer can estimate reinforcement and post-tensioning quantities, hauling weights, and erection sequences.

Acceptance in Florida

As a result of PCI's work, the Florida DOT posted a notice on its Transportation Innovation website titled "Curved Precast Spliced U-Girder Bridges" in 2012. The site makes available for download a new set of the Zone 6 plans that were revised to reflect the specific Florida state policies and practices. The Florida DOT concluded that the girders offer lower fabrication times, faster construction, longer spans, and exceptional aesthetic appeal compared to conventional construction. The address of this website is http://www.dot.state.fl.us/structures/innovation/UBEAM.shtm.

The I-25 Trinidad (Colorado) Viaduct features haunched girders that provide 265-ft clear span over a rail yard. Photo: PCI.

Precast concrete lid slabs with deck overhang cantilevers are used with precast stay-in-place deck panels between girders on the I-25 Trinidad Viaduct. Girder lid slabs can become a composite part of the deck slab. Photo: PCI.

Girder lid slabs are critical to the performance of girders, increasing the torsional stiffness 50 to 100 times. They can become a composite part of the deck slab. Here, the CFX project girders at the abutments are formed and ready for the cast-in-place concrete lid slabs. Photo: J. Dick Precast Concrete Consultant.

An innovative solution to provide space for post-tensioning is the girder "tongue"—an extension of the bottom flange that supports one end of the girder. Girder tongues support the girders on abutment of Ramp H, SR 416–Boggy Creek Interchange seen during TechnoQuest 2 Tour. Photo: J. Dick Precast Concrete Consultant.

Specialized casting forms capable of being configured to cast curved sections with variable radii. A steel casting form viewed during TechnoQuest 1. Precast lid slabs are visible in the background. Photo: PCI.

That was followed by Florida DOT Structures Design Bulletin 13-07 dated June 6, 2013, with the subject, "Spliced Pretensioned/Post-tensioned U-Girders." This document contains specific requirements for the design and construction of these girders. The bulletin was reviewed and approved by FHWA. It can be viewed at: http://www.dot.state.fl.us/structures/Bulletins/2013/StructuresDesignBulletin13-07.pdf.

PCI Bridge TechnoQuest 1

A TechnoQuest is an opportunity to learn from those with experience in a cutting-edge technology. It is conducted where attendees can view projects completed or under construction. It includes presentations on those things that went well and others that could have been done differently and perhaps better.

In September 2012, PCI conducted the first bridge TechnoQuest in Denver, Colo. There were 18 attendees from the Texas DOT, the Texas precast concrete industry, what was then the Orlando Orange County (Florida) Expressway Authority—later to become the Central Florida Expressway Authority (CFX)—and six members of that agency's consulting engineering firms. The presenters included an engineer from the Colorado DOT, three consulting engineers with experience in the region, two precast concrete producers who made curved U-girders in Colorado, and PCI. There were eight classroom sessions to teach the technology and the group toured two precast plants and six bridges.

Just before the TechnoQuest, the CFX had accepted bids on an interchange near the Orlando International Airport—the SR 417/Boggy Road Interchange. The project plans specified curved steel girders. Three of six contractors bid alternates with precast concrete curved U-girders. The contractors had indeed taken notice of presentations of the PCI Zone 6 plans and the Colorado bridges. All three concrete alternate bids were below the lowest bid on the contract documents. But it turned out all bids were rejected because the project needed to be revised and rebid for other reasons. However, it was obvious the curved concrete girders needed to be taken seriously.

The realignment of the highways and the construction of the structures were split into separate contracts. As a result of the previous bidding and confidence gained from the Denver TechnoQuest, CFX redesigned three of the flyover ramps in precast concrete based on the PCI Zone 6 plans. The project was rebid in October 2013. This is the first time that a curved long-span structure in the United States had been designed and specified in precast concrete. All of the Colorado bridges had been design-build projects or contractor alternates.

Production Technology Transfer

In November 2013, PCI conducted one of its regular series of production workshops. These are designed to provide exposure to regional production practices for precast concrete producers from all over the country. Because it was held in Denver and due to interest in Colorado's curved U-girders, the group of some 150 plant personnel visited two U-girder production plants, some participants visited several completed bridges, and explored U-girder bridges then under construction in Trinidad, Colo.

Work in progress to preassemble a reinforcement cage with post-tensioning ducts during TechnoQuest 2 Tour of Dura-Stress Inc. plant in Lessburg, Fla. Inset: No. 9 bars headed at top and bottom to develop in shorter length. Photos: J. Dick Precast Concrete Consultant.

PCI Bridge TechnoQuest 2

With the Florida SR 417 project well underway, PCI held its second bridge TechnoQuest in September 2014 in Orlando. This time over 80 engineers attended. Throughout two days, there were 14 classroom sessions featuring those who had experience with the CFX project and others with experience in previous work and there were several tours. The first tour was of the precast concrete manufacturing plant where nearly all of the plant processes were able to be seen. The group next toured the steel fabricator's plant that manufactured the unique steel forming system. The company explained how the form could be adjusted to cast girders with varying radii as small as 500 ft.

From there, everyone visited the jobsite and was able to get up onto and even into the U-girders that had been erected just a few days before. The owners, contractor, and designers were there to explain the construction and answer questions.

Conclusion

The innovative Colorado DOT created a bold alternative to what was then a traditional method of construction. The industry's technical institute, PCI, envisioned a remarkable new opportunity and organized an effort to nationalize it for the benefit of owner agencies and the industry. Through the development of plans, promotion in numerous venues, and technology transfer opportunities called TechnQuests, curved, precast concrete U-girder bridges are being proven as attractive, cost-effective bridges in several new locations.

William Nickas is managing director of transportation systems for the Precast/Prestressed Concrete Institute in Chicago, Ill. John Dick is a precast concrete consultant and former editor-in-chief of ASPIRE.

For additional photographs or information on this or other projects, visit www.aspirebridge.org and open Current Issue.

EDITOR'S NOTE

This article is a condensation of a much longer and more-detailed paper written by the same authors. The paper includes more information about how U-girder bridges are designed and constructed. Many of the questions raised by designers at the TechnoQuests and elsewhere about this unique structural system are answered in the full-length paper. There are more photographs and references to more information. The full-length paper has been posted on the ASPIRE™ *website and can be viewed and downloaded by selecting the Resources button.*

Extradosed Prestressed Concrete Bridges

Proportioning guidelines and a design example

by Dr. Steven L. Stroh, AECOM

The Odawara Port Bridge, Japan, is the world's first extradosed prestressed concrete bridge, completed in 1994. Photo: Steven Stroh.

Extradosed prestressed concrete bridges are a relatively new development in bridge engineering, with only about 60 of these completed worldwide. The extradosed prestressed concrete bridge has the appearance of a cable-stayed bridge with "short" towers, but behaves structurally closer to a prestressed concrete girder bridge with external prestressing.

Based on work by the author in developing the design for the first extradosed prestressed concrete bridge in the United States, the Pearl Harbor Memorial Bridge, and from reviewing existing extradosed prestressed concrete bridge designs worldwide,[1] this article discusses proportioning guidelines for this bridge type. These guidelines are then applied to the design for the Pearl Harbor Memorial Bridge.

Span Length Range

Extradosed prestressed bridges can be considered in the transition region of span lengths between traditional girder bridges and the longer-span bridge types such as truss, arch, and cable-stayed. Sources in Japan, where most of the extradosed bridges have been constructed, have set the applicable span range for extradosed prestressed bridges to be generally between 100 and 200 m (328 and 656 ft). A review of extradosed prestressed bridges worldwide shows span lengths ranging from 172 to 902 ft. Based on these data, a range from 300 to 600 ft is shown to be a common span range for typical bridges of this type, with span range applicability of 200 to 900 ft.

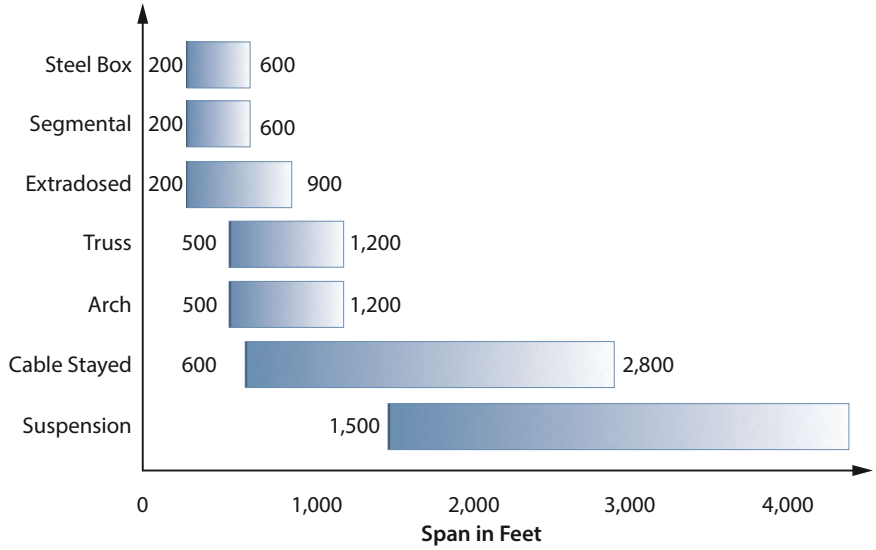
Side Span Ratios

The ratio between the main span length, *L*, and the side span length, *L1*, has an influence on the vertical reactions or anchoring forces at the anchor pier, the moment demands on the girder, and stress changes in the stay cables. For concrete cable-stayed bridges, an economical side-to-main span ratio (*L1/L*) is about 0.42. For concrete girder bridges, *L1/L* should range from about 0.8 for conventional cast-in-place-on-falsework construction to about 0.65 for balanced-cantilever construction.

A review of worldwide data on extradosed bridges shows that *L1/L* varied from 0.33 to 0.83 with a mean of 0.57.[1] The standard deviation is 0.12, so one standard deviation each side of the mean gives a range for *L1/L* of 0.45 to 0.69. This places extradosed bridges essentially between the envelopes of concrete cable-stayed bridges and balanced-cantilever-constructed concrete-girder bridges.

Multi-span Bridge Application

Cable-stayed bridges are typically either two-span or three-span arrangements. These span arrangements are ideal for cable-stayed bridges because back stay cables can be provided from the anchor piers to the top of the towers to provide stiffening of the towers. A recent review of more than 1200 examples of cable-stayed bridges worldwide revealed that only seven cable-stayed bridges are multi-span bridges (meaning more than three spans).[1] Design of a multi-span cable-stayed bridge presents a special challenge, in that there is no opportunity



Span ranges for common bridge types. Drawing: Federal Highway Administration.

Ibi River Bridge, Japan, is a multi-span, extradosed prestressed concrete bridge. Photo: Steven Stroh.

for backstay cables in the central spans, and special design considerations must be made to address the resulting flexibility of the structural system. Solutions for multi-span cable-stayed bridges include the provision of very stiff towers or providing crossing backstay cables that are anchored from the central tower to the base of adjacent towers.

Extradosed prestressed bridges do not rely on back stay cables. So, unlike cable-stayed bridges, multi-span extradosed bridge arrangements do not require special measures. A recent review of more than 60 extradosed bridges built around the world to date showed 19 of these bridges had four or more spans (representing nearly 30% of the bridges built).[1] The extradosed bridge type is well suited to long multi-span bridge arrangements.

Tower Height

An important parameter for extradosed bridges, and one that differentiates them from cable-stayed bridges, is the tower height. The tower height directly influences the stay stress variation under live load (the fatigue stress range) and the proportion of loads shared between the girder and the cables.

For cable-stayed bridges, the optimal ratio of the tower height, *H*, to main span, *L*, is typically between ¼ and ⅕.

For extradosed bridges, the role of the cables is to act as external post-tensioning tendons and provide prestress to the girder. For an extradosed bridge, the post-tensioning is elevated above the cross section of the girder using a short tower, providing a much larger eccentricity, and therefore a more efficient use of the prestressing steel. However, if we continue raising the tower, at some point the vertical component of the cable reaches a force level that starts to carry a significant portion of the vertical live load of the structure.

A review of extradosed bridges constructed worldwide shows a typical range of the *H*/*L* to be between ⅐ and 1/13, with an average value of 1/10.[1]

Girder Depth

Extradosed prestressed bridges are typically constructed in balanced cantilever and their behavior is similar to that of a girder bridge constructed in balanced cantilever, but with more efficient external prestressing. Therefore, a reduction in the structural depth at the support is expected when compared to a girder bridge. For extradosed bridges in the 300- to 600-ft span range, the span-to-depth ratio at the tower varies from 25 to 35 based on a review of existing bridges.[1] Within this range, a typical span-to-depth ratio is about 30.

The girder depth at midspan for an extradosed bridge should be similar to a girder bridge constructed in balanced cantilever. Therefore, a recommended midspan depth-to-span ratio for variable-depth extradosed bridges is 50.

Case Study—The Pearl Harbor Memorial Bridge

The Pearl Harbor Memorial Bridge in New Haven, Conn., is the first extradosed prestressed concrete bridge built in the United States. The Pearl Harbor Memorial Bridge is a three-span, continuous, cast-in-place, segmental concrete box-girder structure with a 515-ft-long main span and 249-ft-long side spans.

The northbound and southbound roadways are carried on separate parallel structures, each having five lanes, a tapering auxiliary lane, and 10-ft-wide shoulders, on a deck that varies in width from 95.4 to 107.6 ft. The superstructure consist of a five-cell concrete box-girder section. The girder depth varies through a parabolic haunch from 11.5 ft at midspan to 16.4 ft at the towers. The cast-in-place segmental concrete box-girder superstructure is post-tensioned longitudinally and transversely.

At the two interior supports, the twin decks are supported by three towers that extend above the deck and an additional intermediate column below each deck. The hollow towers have a constant cross section and are elliptical in shape. Foundations are 8-ft-diameter drilled shafts founded on rock.

Pearl Harbor Memorial Bridge is the first extradosed prestressed concrete bridge designed in the United States. Photo: AECOM.

Table 1—Recommended Proportioning Guidelines for Extradosed Prestressed Concrete Bridges

Parameter	Guideline
Common span range	300 to 600 ft
Side span-to-main span ratio	0.6
Girder depth at tower*	25 to 35
Girder depth at midspan†	50
Tower height above deck‡	1/10

* Guideline on ratio of main span length-to-girder depth at tower.
† Guideline on ratio of main span length-to-girder depth at midspan.
‡ Guideline on ratio of tower height above deck-to-main span length.

Application of Recommendations

The side span-to-main span length ratio for the Pearl Harbor Memorial Bridge is 0.48, which is at the low end of the recommended range. The span length was selected for the Pearl Harbor Memorial Bridge based on geometric constraints. In final design, the result of this relatively short side span was an uplift condition at the anchor piers under certain live-load conditions.

A concrete counterweight was cast inside the box girder at each anchor pier to balance this uplift condition and result in a net positive reaction under all load conditions. It is noted that if the side spans were shortened even more, this uplift condition would become a significant design issue. Therefore the lower range limit to the side span ratio is an important design parameter.

The tower height selected for the Pearl Harbor Memorial Bridge (measured from deck level to the uppermost cable) is 60 ft. This gives an *H*/*L* of 1/8.6. The tower height for the Pearl Harbor Memorial Bridge was selected with a slightly taller tower height than the suggested value, but well within the suggested range. This decision was made based on the wide deck of the bridge, and the desire to reduce demand on the girder system.

A girder depth of 16.4 ft at the towers was selected in order to provide adequate negative moment capacity for the cantilever construction of the girder prior to installation of the first stay. This gives a span-to-depth ratio of 31.4, in the middle of the recommended range. At midspan, a depth of 11.5 ft was selected. This depth was chosen in part to provide a 6.5 ft internal clear height within the box girder for inspection access purposes. This depth gives a span-to-depth ratio of 45, close to the recommended range. For more information about this bridge, see *ASPIRE*™ Fall 2012.

Closing Remarks

Extradosed prestressed concrete bridges are an emerging bridge technology, applicable to bridges in the 300- to 600-ft span range. Basic proportioning parameters for this bridge type are well understood and are summarized in Table 1.

Reference

1. Stroh, Steven L., 2012. *On the Development of the Extradosed Bridge Concept*, a dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy, Department of Civil and Environmental Engineering, College of Engineering, University of South Florida, Tampa, February 8, 2012.

Steven L. Stroh is the Complex Bridges Practice leader for AECOM in Tampa, Fla.

For additional photographs or information on this or other projects, visit www.aspirebridge.org and open Current Issue.

AESTHETICS COMMENTARY

by Frederick Gottemoeller

In the Fall 2012 issue of *ASPIRE*, I commented on the aesthetics of the Pearl Harbor Memorial Bridge in New Haven, Conn., identifying three aesthetic challenges for extradosed bridges:

- the appropriate shape for the towers
- the appropriate size and shape of the girder
- striking a good visual balance between the girder and the towers

This article provides specific proportioning guidelines for tower heights and girder depths based on structural considerations. Aesthetic considerations suggest that girder depths should be near the shallow end of the depth range to have the best appearance. However, aesthetic considerations don't provide such clear guidance with regard to tower height. Considering the tower only, one might think that taller would be better. But the eye judges the combination of tower, cable array, and girder together, not the three elements separately. The eye most strongly reacts to the ratio of the tower height and the girder depth (at the tower) to the span length. The higher that ratio (thus the shorter the tower) the more graceful will the bridge appear. Inevitably the short tower will appear stubby. So, what is the solution for that?

The two Japanese bridges deal with a short, stubby tower by tapering it vertically. The Odawara Port bridge minimizes the apparent thickness of the tower at the top by accommodating the cable anchorages in a separate element that protrudes from the body of the tower. The contrast in thickness between the base and the top makes the top seem smaller than it really is.

The Ibi River bridge takes a different approach. It uses a compound taper to minimize the width of the tower as soon as it emerges from the deck. Having a single row of towers and planes of stays along the median simplifies the appearance of this bridge and makes the structural achievement seem even more dramatic.

In tower design, as in any aesthetic endeavor, it is wise to remember that there are many options available.

Do We Teach or Do They Learn?

The devil is in the details

by Dr. Oguzhan Bayrak, University of Texas at Austin

This year marks the fifteenth year of my teaching career at the University of Texas. Prior to joining the University of Texas, I taught a few structural concrete design classes to civil engineering students at the University of Toronto and to students at the school of architecture at Ryerson University. Whereas the topics of my classes were reasonably similar, I was able to observe different learning patterns exhibited by engineers and architects at an early stage in my teaching career.

I had to use different explanations, perspectives, and teaching techniques when teaching architects than those I had previously used for engineers. Architects and engineers possess different sets of talents and backgrounds. During my formative years as an educator, I witnessed first-hand that structural engineering education was a function of the audience and that it was a complex task.

This early teaching experience proved to me that an effective teacher is one who thoroughly knows her/his subject matter and takes the time to prepare lectures and classroom presentations that are well suited for the audience. With that preamble, I will focus on teaching bridge engineering at the University of Texas.

I had not had the opportunity to teach a bridge engineering class until Spring 2015 semester, despite the fact that I have spent a great majority of my career at the University of Texas conducting research on a variety of topics that relate to concrete bridges and bridge engineering. As I write this article, we are nearing the end of the semester and it is an opportune time for me to take a step back and reflect on the semester from a teaching and learning point of view. More specifically, I will focus on designing reinforced concrete bridge substructures using the strut-and-tie method (STM).

2015 Concrete Bridge Class, University of Texas at Austin: From left to right, Chris Williams, Albert Aquvannoa Limantono, Gabriel Polo, Paul Biju-Duval, Chase Slavin, Dylan Chuey, Katelyn Beiter, Wiam Al Aawar, Amir Reza Ghiami Azad, Randale Shinn, Melvin Goh, Caroline Weston. All Photos: Katelyn Beiter.

2015 Concrete Bridge Class Project at the University of Texas at Austin.

In designing reinforced concrete foundations and bridge bents, one must separate disturbed or discontinuity regions (D-regions) from Bernoulli or beam regions (B-regions). While using legacy methods (for example, sectional design methods for shear, flexure, and the like) in designing B-regions is appropriate, their use in designing regions where plane sections do not remain plane (that is, D-regions) is not advisable. Naturally, the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications* acknowledge this fact and include detailed provisions to be used for the application of this method in bridge design.

However, to the best of my knowledge, most structural designers have been reluctant to use STM in designing bridge substructures. This reluctance can be understood to a great extent by considering the relatively recent introduction of this technique in our

design specifications. This reluctance can be further explained by considering the tremendous flexibility offered by STM. In other words, this design technique is considerably less regimented than our legacy design techniques and therefore it leaves a substantial amount of room for creativity and a perceived element of risk coupled with that freedom.

Considering all of the challenges listed previously, I decided to devote a great majority of my bridge design class to discussing this technique and its application to a variety of concrete bridge components. I did not stop there, I decided to go further. I assigned a project in which student teams designed a substructure element by using STM, and presented their designs in class in the form of an interim submission.

Next, the design teams were asked to build the elements they had previously designed at the Ferguson Structural Engineering Laboratory and to conduct tests on the laboratory specimens stemming from their designs. The ultimate goal was to observe the behavior of the test specimens they had designed.

As I write this article, students taking my class are almost ready to test their specimens. With that said, even before the structural testing, they have already developed substantial understanding in regards to the practicality of their designs. They had to live with the constructability issues that stemmed from some of their design decisions.

For example, the team that employed bent bars (truss bars) in their design had to spend additional time in assembling their reinforcing bar cage in relation to the team that utilized an orthogonal grid in their design, to support the same design loads. Conversely, the design involving bent bars utilized a lesser quantity of reinforcement in relation to the design in which the orthogonal reinforcement was used.

What is the moral of this story? Well, it does not appear that we can get something for nothing in structural engineering.

Next, the student teams will load their test specimens to failure to observe their structural behavior. The cracking patterns observed at service load levels and those observed at ultimate will serve the purpose of explaining the load transfer mechanisms.

Student comments I have received to date indicate that our students greatly value the emphasis I placed on STM. Students in my class are able to identify the load paths that are so essential in substructure design such as compression fields, tension fields, reinforcing bar anchorage, stress concentrations under bearing pads, role of crack control reinforcement, and the list goes on. Ultimately, the most important aspect of STM, or substructure design for that matter, is the need to clearly visualize the load paths created by various design loads and design the substructure elements accordingly.

Equally important are the reinforcing bar details such as column-to-cap connections, column-to-foundation connections, detailing of the ledges of the inverted-tee caps, and hanger reinforcement details, just to name a few. Whereas the concrete element has no way of knowing the methods employed in its design, its structural behavior will most certainly be influenced by its reinforcement detailing. Students taking my bridge engineering class will soon observe the influence of their reinforcing bar detailing decisions on structural behavior. Undoubtedly, the students will learn the fact that the devil is the details, and quite frankly, no design method forces a structural engineer to think through all of the primary details better than the STM in the context of reinforced concrete design.

From the Mouths of Students

At the end of the semester, I asked students in my first bridge engineering class to reflect back on what they had learned. When asked, "Can you please summarize your experience with the class and/or the project?," I received the following responses:

"This class has given me a unique learning experience where theoretical concepts are put to the test (literally!) and hands-on experience with lab testing procedures has enhanced my engineering education in a way that traditional classroom-style teaching is not able to achieve."

"The Strut-and-Tie Method has presented itself as a tool all bridge engineers should adopt to gain a full understanding of the forces transferring through deep beams."

"During this class, it has been exciting to learn about soon to be implemented techniques while, at the same time, having it firmly fixed in reality, with an emphasis on construction considerations and the unique opportunity to work through a class problem from design to construction. It's one thing, in a class, to listen to a lecture on constructing a beam, but another to get to experience it for yourself."

"Dr. Bayrak's class, based not just on structural theories, but engaging us, students, in hands-on lab experience, is a fun and complementary way to understand the real challenges in fabrication, understanding the behavior of the members we design."

"Bridge class, with all its components (including but not limited to the project), has been an enlightening and rather comprehensive entry into the world of strut-and-tie modeling of concrete bridges for me as it involved both learning and applying strut-and-tie modeling not only on the levels of theory and design, but also on the levels of construction and (soon) testing."

"It has been a pleasure to finally get some hands-on experience in the lab, which up until now has been an opportunity that I have regretfully missed out on. In addition, it has been interesting to learn something new and fresh (STM) that I and many others have little to no experience with."

"It was a great design-build team effort to optimize deep beam reinforcement using strut-and-tie theory, and even extending those concepts for curved reinforcement in a reverse curvature bending configuration. So it is definitely an interesting project."

Precast Replaces Historic Bridge in Glacier National Park

by Linda Josephson, Oldcastle Precast Inc.

New bridge being installed. Photo: Sam Barchus, Northbank Civil & Marine.

At the outlet of Swiftcurrent Lake in Montana's Glacier National Park, the Swiftcurrent Bridge provides the only major access to the Many Glacier Hotel, a National Historic Landmark visited by thousands of park visitors every year. The five-span historic bridge and its piers, completed in 1930, were losing their structural integrity and were threatened by high water and spring ice break ups on the lake. The new clear span bridge, installed in November 2014, was designed to be consistent with the historic character of the area and preserve the visual elements of the existing bridge, yet expand hydraulic capacity and comply with modern codes.

The precast concrete deck consists of six voided slabs that are 85 ft long and 30 in. deep with a 12-in.-deep overhanging sidewalk on one side and a 12-in.-deep overhanging bridle path on the other. These exterior slabs are colored black, so with the overhanging portions it gives the appearance of a slimmer slab than it is, mimicking the 12 in. depth of the old bridge. Five of the voids have 8-in.-diameter pipe within them for utility runs.

The precast concrete substructure consists of two 30-ft 8-in.-long pile caps with curved wing walls. The pile caps are installed just outside the old abutment locations resulting in slightly expanded clearance. Interior piers were eliminated. Otherwise, the new bridge matches the width and location of the old.

The bridge replacement occurred between the close of the hotel in September and the onset of impeding winter weather in the park at an elevation of 4800 ft. To minimize field time to erect, (in addition to precasting the overhangs) pile caps and slabs were trial-erected in the precast yard to ensure fit and identify any issues to be mitigated in the plant. Additionally, wing walls were welded to the pile caps before shipment. Even with some delays drilling for the pilings, relatively mild weather allowed the deck delivery to proceed. Driving rain and 55 mph winds at the site made difficult conditions when the trucks arrived. Mercifully, the weather calmed down in a few hours and all pieces were set. The end result is a handsome bridge in a spectacular setting.

Trial fit in precast yard. Photo: Linda Josephson, Oldcastle Precast Inc.

Linda Josephson is a project manager with Oldcastle Precast Inc. in Spokane, Wash.

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FHWA LWC Research Leads to LRFD Specification Changes

by Dr. Ben Graybeal, Federal Highway Administration, and Dr. Gary Greene, Trine University

Lightweight aggregate stockpiles at precast concrete plant. All Photos and Figures: FHWA.

Lightweight concrete (LWC) is a class of concrete materials that has been available for use in structural and nonstructural applications for many decades. Modern use dates back to the middle of the twentieth century when the production of expanded shales, clays, and slates began and thus the necessary coarse and fine aggregates became available from suppliers spread across the United States. Bridge design specifications in the United States have included provisions related to LWC for much of this timeframe, up to today with the American Association of State Highway and Transportation Officials' (AASHTO's) *AASHTO LRFD Bridge Design Specifications*.

The AASHTO Subcommittee on Bridges and Structures (SCOBS) recently considered and adopted a series of recommended changes to the AASHTO LRFD specifications that increase the consistency, uniformity, and usability of the provisions as they relate to LWC. Previously, the LWC provisions were constituent-based and they only addressed concrete unit weights below 120 lb/ft^3. The AASHTO LRFD specifications define normal-weight concrete as concrete having a unit weight between 135 and 155 lb/ft^3.

This created challenges for designers who were unlikely to know the type of fine or coarse aggregate that would be used in the concrete and who might want to use a concrete whose unit weight was between 120 and 135 lb/ft^3. The existing provisions also required the use of a lower shear resistance factor due to the perception that there was limited data available on the performance of LWC in shear.

The bridge design and construction community had recognized these challenges to the use of LWC by the early 2000s, leading to the growing interest in executing research efforts that would systematically address the needs. The largest efforts were completed through:

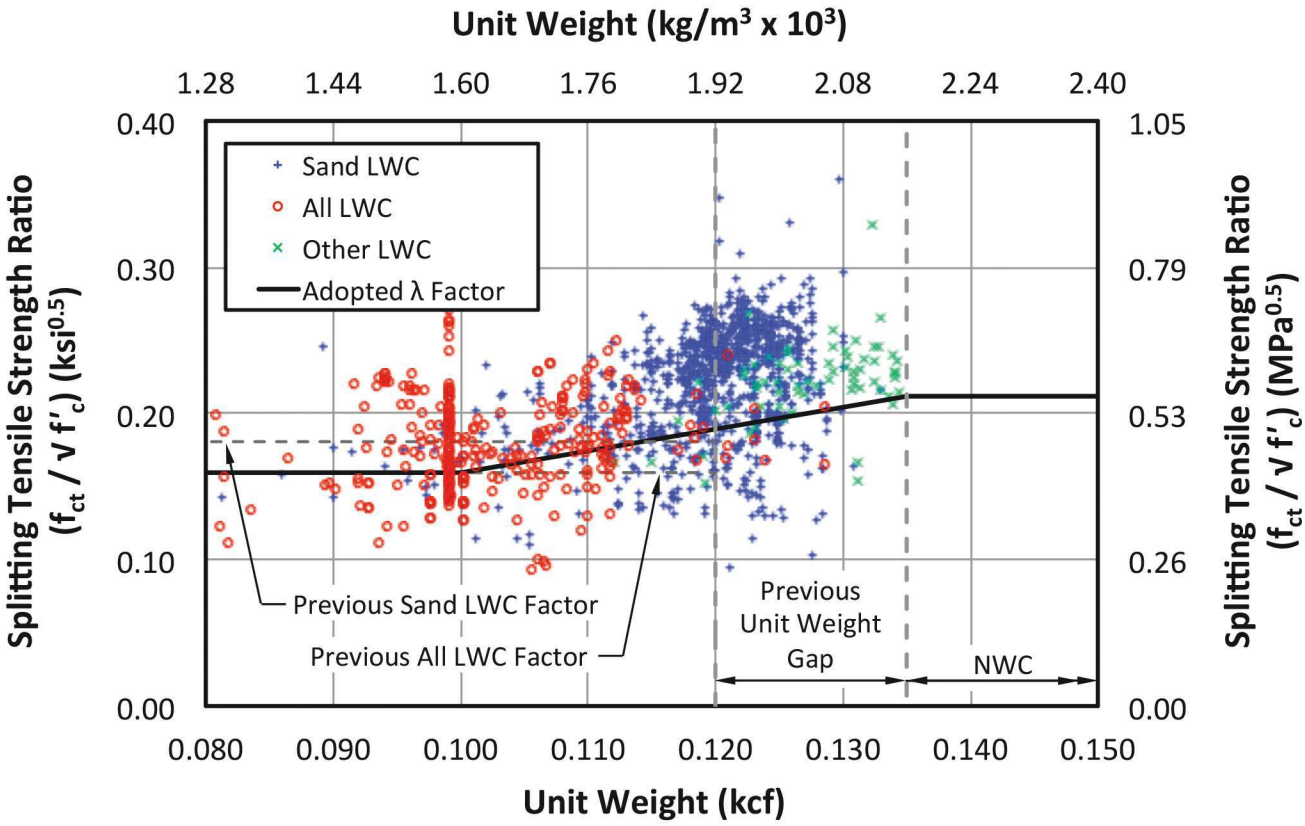
- the National Cooperative Highway Research Program's Project 18-15, which produced Report 733, and
- the Federal Highway Administration's (FHWA's) Turner-Fairbank Highway Research Center, which executed an ambitious experimental and analytical study that has culminated in working with AASHTO SCOBS Technical Committee for Concrete Design (T-10) to revise the LRFD specifications.

FHWA's research included the full-scale testing of nearly 100 beams and girders to assess behaviors including shear resistance, reinforcement detailing, and strand bond, as well as the compilation of databases of material scale and structural scale test results that allowed for the development and assessment of potential design provisions.

The framework for the specification revisions comprises four interrelated topic areas: LWC definition, material properties, resistance related to tensile strength, and resistance factors.

The revised definition speaks to concrete containing aggregate conforming to AASHTO M 195, *Standard Specification for Lightweight Aggregates for Structural Concrete*, and having an equilibrium density not exceeding 135 lb/ft^3 as defined by ASTM C567, *Standard Test Method for Determining Density of Structural Lightweight Concrete*. This definition will allow a designer to engage LWC on terms that are known at the design stage, leaving it to the contractor to develop the concrete mixture proportions with the appropriate set of properties.

LWC can be produced at compressive strength levels up to 10 ksi; however, it is recognized to commonly exhibit slightly less tensile strength and a slightly lower modulus of elasticity as compared to a similar compressive strength normal-weight concrete. Both high-strength LWC and concrete whose unit weight was between 120 and 135 lb/ft^3 have seen increased usage. The material properties of LWC have been refined within the material properties section of Section 5 of the AASHTO LRFD specifications through the revised modulus of elasticity equation that was approved in 2014 and the tensile response revisions that were just recently approved in 2015 and will be included in the 2016 Interim Revisions to the AASHTO LRFD specifications.



Splitting tensile strength of lightweight concrete as a function of unit weight and type. Note: f_{ct} = concrete splitting tensile strength; f'_c = concrete compressive strength.

Shear testing of lightweight concrete prestressed girder.

Many service and strength limit state provisions throughout Section 5 rely on the tensile strength of the concrete as a key parameter in the predictive expressions. Most commonly, these expressions inferred the tensile resistance of the concrete through the use of a $\sqrt{f_c'}$ term. The approved specification revisions implement a framework that allows the reduced tensile strength of LWC to be addressed by defining a modification factor that can be referenced from any relevant Section 5 provision. In short, this factor, λ, can be determined either from ASTM C496 splitting tensile strength test results on the particular mixture proportions or from a calculation based on the unit weight of the concrete. The value of λ varies linearly from 0.75 for a 100 lb/ft³ concrete to 1.0 for a 135 lb/ft³ concrete. For concretes heavier than 135 lb/ft³, λ =1.0.

The resistance factor for LWC in shear had previously been set to a lower value than that used for normal-weight concrete. These resistance factor values were based on a smaller number of full-scale shear tests than normal-weight concrete and the fact that LWC can have a lower shear resistance than normal-weight concrete. This led to uncertainty in the level of conservatism provided by the shear design expressions for LWC and thus a lower resistance factor. A set of recent studies[1] on LWC shear performance has expanded the database of results and demonstrated that the use of the same shear resistance factor for LWC and normal weight concrete is appropriate. The phi-factor for both is now set at 0.90.

The two LWC-related ballot items that passed AASHTO SCOBS in April 2015 comprised a total of 39 sub-items spread across the AASHTO LRFD specifications and one item in the *AASHTO LRFD Bridge Construction Specifications*. In total, the revised specifications allow for a more consistent, uniform treatment of LWC and thus will facilitate broader use of this important construction material.

Reference

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Control of Flexural Cracking

by Dr. Henry G. Russell, Henry G. Russell Inc.; Dr. Danielle D. Kleinhans, Concrete Reinforcing Steel Institute; and Dr. Dennis R. Mertz, University of Delaware

Control of flexural cracking by distribution of reinforcement is addressed in Article 5.7.3.4 of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*.[1] This article requires that the spacing, s, of nonprestressed steel reinforcement in the layer closest to the tension face shall satisfy the following equation:

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \qquad \text{(AASHTO Eq. 5.7.3.4-1)}$$

in which:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

where:

γ_e = exposure factor
= 1.00 for Class 1 exposure condition
= 0.75 for Class 2 exposure condition

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.)

f_{ss} = calculated tensile stress in nonprestressed steel reinforcement at the service limit state not to exceed 0.60 f_y (ksi)

h = overall thickness or depth of the component (in.)

In most situations, Eq. 5.7.3.4-1 results in reasonable bar spacings. However, the following two situations exist where the equation results in small bar spacings and even negative values for s:

1. The use of Grade 100 reinforcement in bridge decks, which was permitted in the 2013 Interim Revisions.[2] This affects both ASTM A1035 and the new ASTM A615 (AASHTO M31) Grade 100 bars.
2. The clear cover is relatively large compared to the section depth. An extreme example is where piles extend into pile caps and the bottom layer of pile cap reinforcement is placed above the top of the piles. This results in a value of d_c at least equal to the embedment depth of the piles into the pile cap.

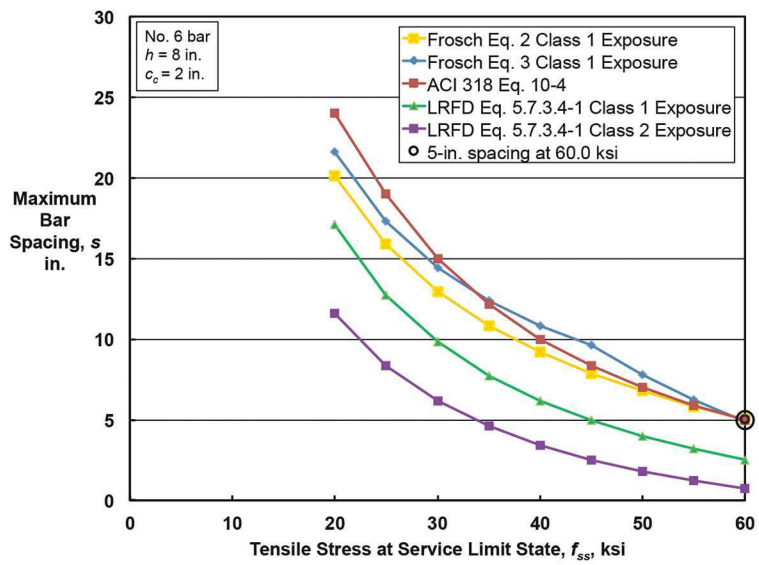

Figure 1. Comparison of crack control equations from several sources. All Figures: Henry Russell.

Equation 5.7.3.4-1 was developed by DeStefano et al.[3] based on a review of past research, parametric studies, and various crack width predictive methods. The equation is based on an assumed crack width of 0.017 in. for Class 1 exposure. It replaced a previous equation developed by Gergely and Lutz.[4] In the previous equation, the value of clear cover, c_c, used to calculate d_c was limited to a maximum value of 2 in. In the current equation, d_c is to be calculated using the actual concrete cover.

To ascertain the validity of the small values of s, values calculated using Eq. 5.7.3.4-1 were compared with values calculated using equations developed by Frosch[5] and ACI Committee 318[6] as described in the next sections.

Frosch Equations

Frosch[5] developed the following equation to predict crack widths with uncoated reinforcement:

$$w_{cu} = 2\frac{f_s}{E_s}\beta_s\sqrt{(d_c)^2 + \left(\frac{s}{2}\right)^2} \qquad \text{(Eq. 1)}$$

in which:

β_s = $1.0 + 0.08d_c$

where:

w_{cu} = maximum crack width for uncoated reinforcement (in.)

f_s = stress in steel reinforcement (ksi)

E_s = modulus of elasticity of reinforcing bars (ksi)

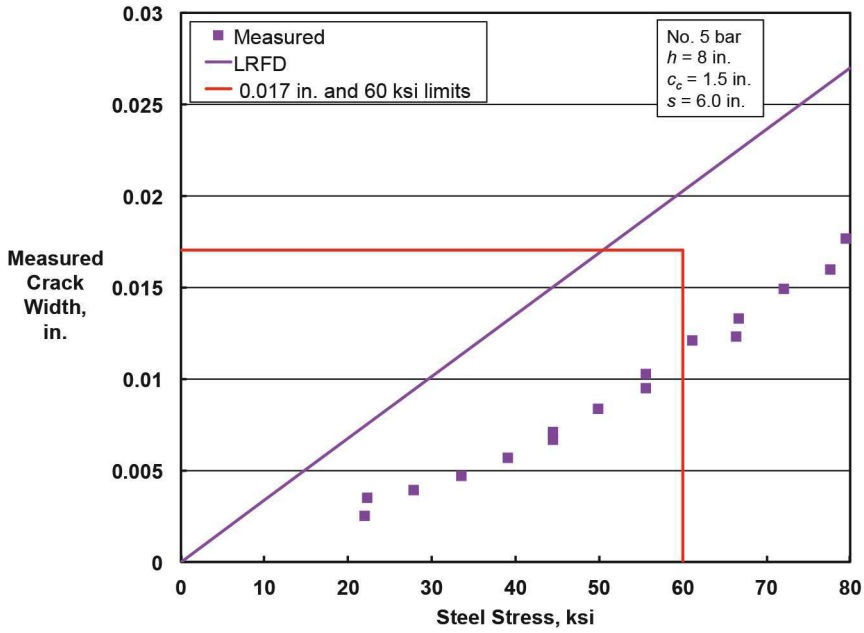

Figure 2. Comparison of crack widths versus steel stress for data from Sim[7] and relationship assumed for AASHTO LRFD Eq. 5.7.3.4-1.

The equation can be rewritten to solve for maximum permitted reinforcement bar spacing as follows:

$$s = 2\sqrt{\left(\frac{w_{cu}E_s}{2f_s\beta_s}\right)^2 - (d_c)^2} \qquad \text{(Eq.2)}$$

Frosch compared crack widths calculated by his equation with existing test data for reinforcement stress levels, f_s, ranging from 20 to 50 ksi, but not including any at 60 ksi. Based on maximum crack widths between 0.016 and 0.021 in., Frosch then proposed the following simplified design equation:

$$s = 12\alpha_s\left[2 - \frac{d_c}{3\alpha_s}\right] \le 12\alpha_s \qquad \text{(Eq.3)}$$

where:

α_s = reinforcement factor = $\frac{36}{f_{ss}}\gamma_c$

γ_c = reinforcement coating factor: 1.0 for uncoated reinforcement; 0.5 for epoxy-coated reinforcement, unless test data can justify a higher value.

ACI 318 Equation

In 2005, ACI Committee 318[6] adopted the following simplified version of Frosch's equation:

$$s = \frac{600}{f_{ss}} - 2.5c_c \le \frac{480}{f_{ss}} \qquad \text{(ACI Eq. 10-4)}$$

and f_{ss} may be taken as 2/3 f_y.

Comparison of Calculated Bar Spacings

The maximum bar spacing calculated from the Frosch equations, ACI 318 equation, and the current AASHTO LRFD equation for Class 1 or Class 2 exposure are shown in Figure 1 for an 8-in.-thick slab containing No. 6 bars with a clear cover, c_c, of 2 in. As expected, the maximum spacing per the Frosch equations and ACI 318 equation agree reasonably well and are all greater than the maximum spacing calculated using the AASHTO LRFD equation for both Class 1 and Class 2 exposure conditions. Figure 1 indicates that the AASHTO LRFD equation is more conservative than the Frosch or ACI 318 equations, which is generally the case for bridge decks.

Clearly, one variable that influences the calculated maximum bar spacing using the AASHTO LRFD equation is the exposure factor—either 1.0 or 0.75. The exposure factor is applied to the first term of Eq. 5.7.3.4-1. Mathematically, this results in bar spacings being calculated as the difference between two numbers that are close together and, as mentioned previously, can result in negative values.

The exposure factor for Class 1 exposure condition of 1.00 is based on an assumed crack width of 0.017 in. When this crack width is substituted into Frosch's Eq. 2 and 3 for a No. 6 bar with h = 8 in., c_c = 2 in., and f_{ss} = 60 ksi for Grade 100 reinforcement, the following bar spacings are obtained:

Frosch's equation: 5.01 in.
Frosch's design equation: 4.90 in.

This is illustrated in Figure 1 with convergence of the two lines for Frosch's equations and the ACI equation at a spacing of about 5 in. and tensile stress of 60 ksi.

Proposed Solution

With larger bar spacings, there is concern about larger crack widths. Sim[7] conducted flexural tests using 8.0-in.-deep beams containing No. 5 Grade 60 and 100 uncoated reinforcing bars at a center-to-center spacing of 6.0 in. and with a clear cover of 1.5 in. Figure 2 shows the reported values of average crack widths and corresponding steel stresses. All measured crack widths at steel stress levels below 60 ksi were less than the crack width of 0.017 in. assumed for Class 1 exposure condition as depicted by the red lines in Figure 2. The 60.0 ksi limit is the maximum stress permitted for Grade 100 reinforcement in Eq. 5.7.3.4-1.

Figure 2 also shows the relationship between crack width and steel stress that was assumed by DeStefano et al.[3] in their development of Eq. 5.7.3.4-1. The diagonal line labelled "LRFD" is calculated for the cross section tested by Sim. The measured crack widths are all less than the assumed values for the same stress levels.

To address the two situations described above, the AASHTO Subcommittee on Bridges and Structures has approved the following addition to the Commentary of Article 5.7.3.4:

"In certain situations involving higher-strength reinforcement or large concrete cover, the use of Eq. 5.7.3.4-1 can result in small or negative values for s. Where higher-strength reinforcement is used, an analysis of five crack control equations suggests a bar spacing not less than 5.0 in. for control of flexural cracking. Where large concrete cover is used, past successful practice suggests a value of d_c not greater than 2.0 in. plus the bar radius for calculation purposes. The limit of 2 in. plus bar radius is the same limit that was used in the Specifications prior to 2005 when the current version of Eq. 5.7.3.4-1 was introduced."

This revision will become effective with the 2016 Interim Revisions. Although the revision provides a simple solution, more research concerning Eq. 5.7.3.4-1 is needed as higher-strength reinforcing bars become available.

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Dr. Henry G. Russell is an engineering consultant; Dr. Danielle D. Kleinhans is a structural/transportation engineer with the Concrete Reinforcing Steel Institute; and Dr. Dennis R. Mertz is a professor of civil engineering at the University of Delaware.

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Improving Durability Through Certification of Epoxy-coated Reinforcing Steel

by Dr. David McDonald, Epoxy Interest Group

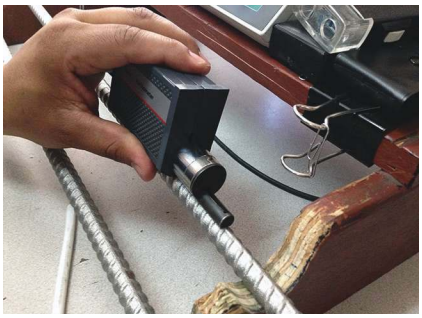
In 2012, the Concrete Reinforcing Steel Institute (CRSI) was approved by the American National Standards Institute (ANSI) as a Standards Development Organization (SDO). The hallmarks of the American National Standards process include

- consensus by representatives from materially affected and interested parties,
- broad-based public review and comment on draft standards,
- consideration of, and response to, comments from voting members as well as public review commenters,
- incorporation of approved changes into a draft standard, and
- right to appeal by any participant that believes that due process principles were not sufficiently respected during the standards development.

History of Epoxy-coated Bars

Epoxy-coated reinforcing steel was introduced in 1973, in response to significant bridge deck corrosion concerns in the 1960s.[1] The increased use of epoxy-coated reinforcing steel was rapid and by 2014, over 82,000 bridge decks and many other structures contained epoxy-coated reinforcing steel.

The early history of epoxy-coated reinforcing steel was marred by a few notable failures, including corrosion of coated reinforcing steel in piers of bridges in the Florida Keys observed in 1986.[2] This corrosion—exacerbated by porous concrete, thin concrete cover, and inadequate manufacture and protection of the coated steel—led many research programs to question the value of the epoxy-coating system.



Measurement of steel surface roughness to ensure adequate surface area for coating bonding. Photo: Western Coating Co.

Placing of epoxy-coated reinforcing steel. Photo: Epoxy Interest Group of CRSI.

One of these research programs involved testing of bent, coated reinforcing steel from seven manufacturers.[3] After approximately two years of testing, five of the systems exhibited poor performance, while two of the systems exhibited excellent performance. It was concluded that the performance was based directly on the application of the coatings and manufacturing procedures.

Initial Certification Programs

In 1991, CRSI launched a voluntary epoxy-coated reinforcing steel certification program. This program required independent review of plant procedures using unannounced inspection. Improvements in epoxy-coated bar consistency between plants was dramatic. For example, prior to the certification program, certain plants were coating products that had 50 to 70% dust contamination, which was termed backside contamination. Currently, this value averages 15%.

The CRSI program has worked synergistically with the ASTM specifications, leading to changes that improved product quality. Such changes to the requirements include:

- Increased coating thickness
- Introduction of required steel roughness prior to coating
- Requirements for clean substrates (backside contamination)
- No cracking allowed during bending
- Improved storage and handling

Cracking of the coating during bending is considered unacceptable now; however, in 1983 certain agencies permitted unrepaired hairline cracking in the coating and did not require repairs, nor did damage repair, if the damaged area was less than 25 mm^2 (0.04 $in.^2$).[4]

Development of National Standards

During 2014 and 2015, CRSI produced three national standards relating to epoxy-coated reinforcing steel:

- CRSI Standard for Epoxy Coating Plant: Straight Bar Lines
- CRSI Standard for Epoxy Coating Facilities: Custom Lines
- CRSI Standard for Epoxy-Coated Steel Reinforcing Bar Fabrication Facilities

To develop these standards, the documents underwent rigorous and open consensus-balloting procedures. Balloting occurred within a task group, the standards committee, and through public comment. These standards are the basis for future plant inspections.

Process Certification

CRSI is currently pursuing accreditation from ANSI under International Organization for Standardization (ISO) 17065, *Conformity Assessment — Requirements for bodies certifying products, processes and services.*[5] This program requires CRSI to maintain an Independent Manufacturing Certification Committee (IMCC) that is responsible for all items related to certification within CRSI. This committee is made up from industry experts, department of transportation officials, university academics, and others that have significant knowledge of ISO processes. The program is independent of the rest of the CRSI operations and follows written rules.

The adoption of ISO 17065 requires all steps in any CRSI certification program to be documented and audited. Thus plants, inspectors, and CRSI are all audited for conformance with their quality control procedures and they must address nonconformances. The program also requires certified plants to address any customer complaints.

Summary and Benefits

The adoption of ISO 17065 along with the development of national standards by CRSI has led to a more transparent certification program for epoxy-coated reinforcing steel. The programs provide a clear method for improving processes.

The benefits of CRSI certification programs to the final customer of epoxy-coated reinforcing steel is clearly found with products that are more robust and durable than the product used in the 1980s. The program provides a clear and open method for customers to provide their input into the programs and all concerns must be addressed.

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Measurement of coating thickness as part of the plant quality-control procedures. Photo: Epoxy Interest Group of CRSI.

Background

The Concrete Reinforcing Steel Institute (CRSI) has been developing programs and processes to seek accreditation by the American National Standards Institute (ANSI) for its epoxy-coating plant and fabrication certification programs. ANSI accreditation signifies that the procedures used by CRSI meet essential requirements for openness, balance, consensus, and due process. The move toward ANSI accreditation strengthens the CRSI certification programs relating to epoxy-coated reinforcing steel.

A typical example of epoxy coating flexibility. This No. 4 bar is bent around a 1.5-in.-diameter pin. No cracking of the coating is observed. Photo: David McDonald.

David McDonald is the managing director of the Epoxy Interest Group of the Concrete Reinforcing Steel Institute in Schaumburg, Ill.

When Should Mass Concrete Requirements Apply?

by John Gajda and Jon Feld, CTLGroup

Insulated columns for a bridge project. Photo: John Gajda.

Mass Concrete Mechanism

Temperature builds within a concrete placement when the rate of heat generation by the hydration of cementitious materials exceeds the rate of heat loss through the surfaces of the placement. Concretes with a high cementitious materials content will generate heat quite quickly; quicker than the heat can escape. Thick placements also trap heat such that the cementitious materials at the center cannot readily dissipate the heat. Saying this differently, if one were to use a concrete mixture that contains, for example, 600 lb/yd^3 of cementitious materials (where 75% is Type I/II portland cement and 25% is class F fly ash) in an 8-in.-thick bridge deck, the rate of heat dissipation would be fast enough that the concrete does not get overly hot and therefore does not behave as mass concrete. However, if the same concrete mixture was used in a 6-ft-thick placement, the story would be different; the interior of the placement would get quite hot since the heat cannot dissipate as quickly as it is generated; the concrete placement would behave as mass concrete.

The question of when a placement should be considered to be mass concrete is often debated. From the American Concrete Institute (ACI) definition, mass concrete is "any volume of concrete in which a combination of dimensions of the member being cast, the boundary conditions, the characteristics of the concrete mixture, and the ambient conditions can lead to undesirable thermal stresses, cracking, deleterious chemical reactions, or reduction in the long-term strength as a result of elevated concrete temperature due to heat from hydration." This is an excellent definition, however, it does not provide the simple and measurable guidance that most engineers and contractors are looking for; it does not provide a quantitative definition, such as a thickness-based definition.

ACI 301, *Specifications for Structural Concrete*,[1] attempts to provide simple and measurable guidance in the "notes to the specifier section" at the back of the document, by stating that placements that are 4-ft thick and greater should be considered mass concrete. In placing the guidance in the notes section, rather than the specification section, the actual thickness that is specified is left to the discretion of the specifier.

In this same section, ACI 301 also states that placements with a minimum cementitious materials content of 660 lb/yd^3 should also be considered mass concrete. This latter guidance, although well meaning, has resulted in treatment as mass concrete for placements which probably should not be considered mass concrete. For example, we have written letters demonstrating that a 1-ft-thick wall constructed with a concrete containing 675 lb/yd^3 of cementitious materials will not behave as mass concrete.

So when should a placement be treated as mass concrete? When the maximum temperature in the placement exceeds the typical industry standard limit of 160°F and/or when the temperature difference in the placement exceeds the typical industry standard limit of 35°F between the interior of the placement and a point that is 2 to 3 in. below/inside the center

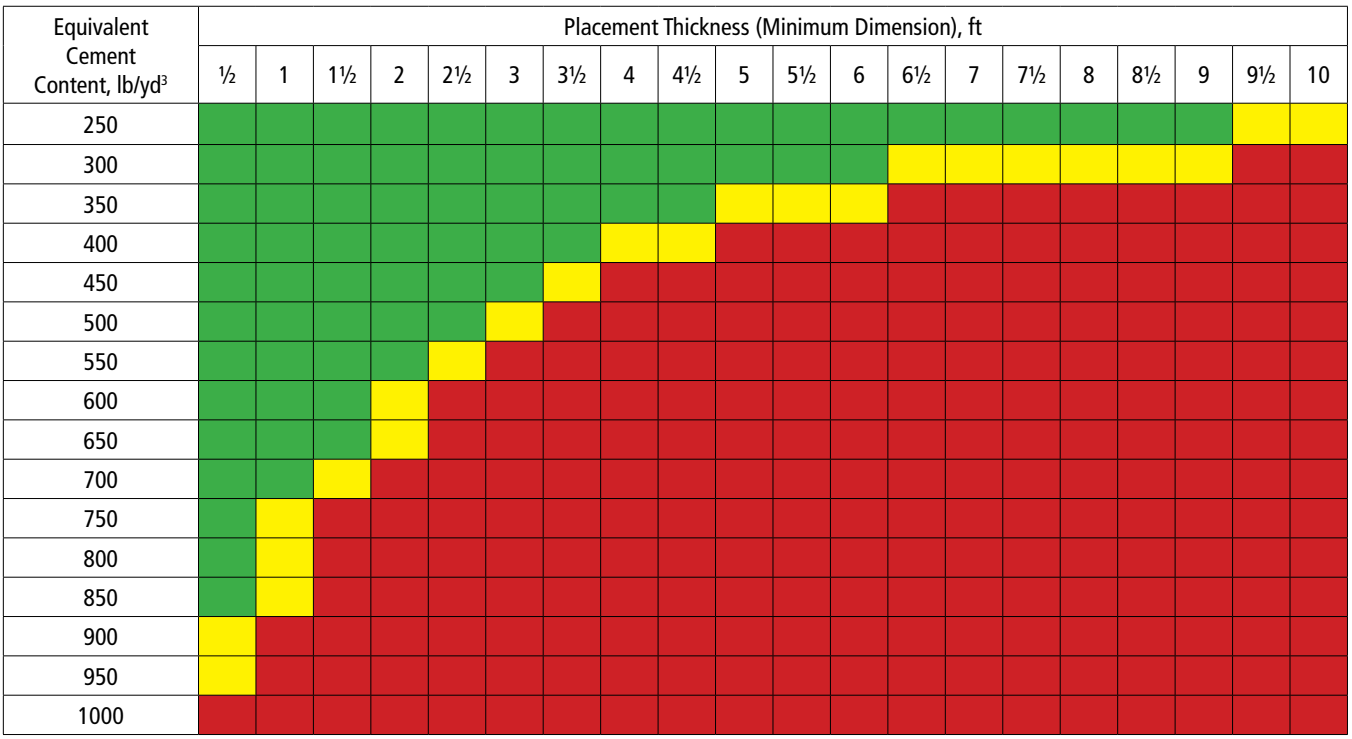

Chart of placement versus equivalent cement content for normal-weight concrete. Red is mass concrete, green is not mass concrete. Yellow is a buffer zone that is left to the discretion of the specifier. Chart: John Gajda.

of a nearby surface. Placements that will not exceed these limits under typical placement conditions do not need to be considered mass concrete. What exceeds these limits is a function of the placement thickness and the concrete mixture proportions, specifically the type and quantities of the cementitious materials that are used.

We have been working to develop a better definition of mass concrete; one that takes into consideration both the placement thickness as well as the concrete mixture design. Thermal modeling was performed using mass concrete software to look at the maximum temperature and temperature difference in a series of different thickness placements with various concrete mixture proportions. In these mixture proportions, the cementitious materials such as portland cement, fly ash, slag cement, silica-fume, and metakaolin were converted to an "equivalent cement" content based on an equation (shown below) from an article in the August 2014 edition of *Concrete International* magazine.[2] Although this equation is not perfect, in that it probably does not adequately cover every combination of cementitious materials, from our experience, it is reasonably accurate as a "ballpark" comparison method for most concretes.

Equivalent Cement Content = (*Cement* + 0.5**FAsh* + 0.8**CAsh* + 1.2**SFMK* + *Factor***Slag*), lb/yd³

where:

Cement is Type I/II portland cement, lb/yd³;

FAsh is Class F fly ash, lb/yd³;

CAsh is Class C fly ash (no distinction is made for the calcium oxide content of the fly ash, which is the main heat generating portion), lb/yd³;

SFMK is silica fume or metakaolin, lb/yd³;

Slag is slag cement (no distinction is made for Grade 100 or Grade 120), lb/yd³; and

Factor is a variable which depends on the total percentage of cement being replaced (1.0 to 1.1 for 0 to 20% cement replacement, 1.0 for 20 to 45% cement replacement, 0.9 for 45 to 65% cement replacement, and 0.8 for 65 to 80% cement replacement).

For the modeling, we assumed that the concrete would not be placed during cold-weather conditions (that is, it would not be insulated), it would not be thermally protected (that is, treated as mass concrete), it would be constructed using steel forms, and it would not be subjected to cold rain or windy conditions. We also assumed that the temperature of the delivered concrete was 10°F above the average air temperature, with the average air temperature being the average of the daily high and the daily low air temperatures on the day of placement, the preceding several days, and the following several days.

We looked at the maximum temperature and maximum temperature difference that were predicted by the thermal modeling. In virtually all cases, the temperature difference was the limiting factor; in other words, without insulation on the surface of the placement, modeling showed that the temperature difference would exceed the industry standard 35°F temperature difference limit before the maximum temperature exceeded the 160°F maximum temperature limit.

From this, a placement thickness-versus-equivalent-cement-content chart was developed and is shown in this article. Again, although not perfect, this chart provides a reasonable definition of when a placement should be considered mass concrete. We believe it is along the lines of what the industry is looking for.

References

1. American Concrete Institute (ACI). 2010. *Specifications for Structural Concrete*. ACI 301-10, Farmington Hills, Mich.: ACI.
2. Gajda, J., M. Weber, and I. Diaz-Loya. 2014. "A Low Temperature Rise Mixture for Mass Concrete." *Concrete International*, ACI, August.

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EDITOR'S NOTE

For more information on mass concrete, see the HPC Bridge Views *article at www.hpcbridgeviews.com/i47/Article1.asp.*

CONCRETE BRIDGE PRESERVATION

CBP

Plymouth Avenue Bridge

by Jason Caravello, Dywidag Systems International USA Inc.

The Plymouth Avenue Bridge is a four-lane, four-span, twin-box, segmental, box-girder bridge carrying Plymouth Avenue over the Mississippi River on the north side of Minneapolis, Minn. The bridge boasts a total length of 943 ft with the longest span reaching 143 ft. Built in 1983, it was the first segmental bridge in Minnesota and it was built using a form traveler for cast-in-place concrete construction.

During a 2010 routine inspection, the bridge was closed after corroded post-tensioning tendons were discovered, among other forms of deterioration. After a careful review and performance of an inspection analysis, the bridge was deemed adequate for pedestrian and bicycle traffic only. In early 2011, the bridge was closed to vehicular traffic until a repair plan could be implemented.

Upon further evaluation and a root-cause analysis, it was determined that the cause of the corroded post-tensioning tendons was the failure of the bridge's drainage system. The system was designed in such a way that the drainage pipes were brought inside the box girders. A failure occurred in the system that led to deicing chemical leakage accumulating and ponding in the interior of the box, thereby corroding the reinforcing bars and post-tensioning tendons in the bottom slab regions.

The Fix

The Minnesota Department of Transportation was in need of a viable strengthening solution that allowed for a long-term, economical approach to return the bridge to carrying vehicle traffic over the Mississippi River. To accomplish this, they engaged a bridge consultant to design the solution. Upon careful review of all the contributing conditions and factors, it was determined that an external, supplemental multi-strand, post-tensioning system would be installed within the interior of the box girders. Additional repairs were also performed such as epoxy injection, concrete repair, and drain modifications.

Looking west from the northeast. Photo: Jason Carvello.

The general contractor chose a post-tensioning partner who could provide the required resources for this technically demanding project. The design required five new tendons with twelve 0.6-in.-diameter strands to be installed inside both the eastbound girder and the westbound girder for a total of 10 new draped tendons. Each tendon was anchored in new concrete blisters created at each end of the bridge and was run through new concrete deviators cast at box floor locations. All exposed ducts were made of high-density polyethylene (HDPE), while the embedded pipes in the deviators were steel. After tensioning, all tendons were grouted. Repairs to the bottom of the box included partial and full depth concrete demolition, replacement in select areas that had sustained damage, and epoxy injection of the cracks. This concrete repair work was performed by another contractor.

Challenges

The design required embedded steel ducts to deviate tendons in two planes. It was difficult to obtain field measurements for the production and installation of these components. In addition, the concrete anchor blisters installed at the upper regions of the girder interior left little space to maneuver a multi-strand jack into position to stress the new tendons. As is very common with repair and strengthening related projects, these types of access issues are common and need to be overcome in the field operations. Therefore, the post-tensioning contractor was tasked with developing a unique stressing protocol, which called for stressing each strand individually with single strand jacks. As a result, a special stressing system was developed to accomplish this, which proved to work very successfully.

In the end, this project was completed safely, on time, and on budget. The bridge once again serves as a vital link across the Mississippi River in Minneapolis.

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Concrete Connections is an annotated list of websites where information is available about concrete bridges. Links and other information are provided at www.aspirebridge.org.

IN THIS ISSUE

http://www.asbi-assoc.org/index.cfm/publications/publications
This is a link to the publications page on the ASBI website from which the "Durability Survey of Segmental Bridges" mentioned in the Perspective article on page 10 can be purchased.

http://www.fhwa.dot.gov/bridge/britab.cfm
This is a link to the "Tables of Frequently Requested NBI Information" on the FHWA website that is used as the basis for the analysis discussed in the Perspective article on page 10. From this webpage, various tables of data from the NBI can be downloaded, including data files from 1992 to the present.

http://scholarcommons.usf.edu/etd/4232/
This is a link to a webpage for Steven Stroh's dissertation for the University of South Florida titled "On the Development of the Extradosed Bridge Concept." The dissertation provides the basis for the Project article on page 28 and can be downloaded from this webpage.

http://www.honolulutransit.org/
This is a link to the webpage for Honolulu Rail Transit, the owner of the bridge described in the Project Profile on page 12. Information on the website includes a map of the entire project and progress reports on construction.

http://www.pci.org/uploadedFiles/Siteroot/Design_Resources/Transportation_Engineering_Resources/PCI%20Zone6%20Curved%20Spliced%20Girders.pdf
This is a link to a PDF version of the "Zone 6 Concept Plans" for curved spliced U-girders mentioned in the Profile on curved girders on page 24.

http://www.dot.state.fl.us/structures/innovation/UBEAM.shtm
This is a link to the Florida DOT webpage "Curved Precast Spliced U-Girder Bridges" that was mentioned in the Profile on curved girders on page 24. A version of the PCI Zone 6 U-girder details that have been revised to reflect specific Florida state policies and practices can be downloaded from this site, along with slide shows, presentations, and other information.

http://www.dot.state.fl.us/structures/Bulletins/2013/StructuresDesignBulletin13-07.pdf
This is a link to the archived Florida DOT Structures Design Bulletin 13-07 dated June 6, 2013, and titled "Spliced Pretensioned/Post-tensioned U-Girders," which is mentioned in the Profile on curved girders on page 24. This document contains specific requirements for the design and construction of U-girders to Florida standards.

http://www.trb.org/Main/Blurbs/168612.aspx
This is a link to the TRB blurb on NCHRP Report 733, *High-Performance/High-Strength Lightweight Concrete for Bridge Girders and Decks,* which is mentioned in the FHWA article on page 36. A link is given to the blurb rather than the report because there are appendices to the report that can also be downloaded.

http://www.fhwa.dot.gov/publications/research/infrastructure/structures/bridge/15021/15021.pdf
This is a link to recently published FHWA Report FHWA-HRT-15-021 titled, "Lightweight Concrete: Shear Performance," which gives justification for some of the changes to the AASHTO LRFD Specifications mentioned in the FHWA article on page 36. The report will eventually be reformatted and posted in HTML as a webpage.

Bridge Technology

NEW http://www.nhi.fhwa.dot.gov/training/course_search.aspx?tab=0&key=130103&sf=0&course_no=130103
This is a link to the course description for the recently released National Highway Institute (NHI) web-based training course FHWA-NHI-130103 titled "Post-Tensioning Tendon Installation and Grouting." This free 6-hour course, for which CEU credit can be received, delivers content on post-tensioning principles, system components, and installation procedures—including testing and quality control procedures—which will assist supervisors, inspectors, and construction inspectors in performing their job. The course is intended to be an online complement to FHWA's "Post-Tensioning Tendon Installation and Grouting Manual."

NEW http://www.wsdot.wa.gov/publications/fulltext/Bridge/ABC/2015_ABC_Workshop.pdf
This link is to the participant's manual for the WSDOT 2015 ABC Workshop. It includes handouts for presentations given at the workshop.

NEW http://www.ltrc.lsu.edu/pdf/2015/FR_528.pdf
This link is to a report developed for the Louisiana Department of Transportation and Development titled "Development of Wave and Surge Atlas for the Design and Protection of Coastal Bridges in South Louisiana." The report discusses the development of a wave and surge atlas for coastal Louisiana and the determination of the vulnerability of select coastal bridges.

www.aspirebridge.org
Previous issues of *ASPIRE*™ are available to search and as pdf files, which may be downloaded as a full issue or individual articles. Information is available about free subscriptions, advertising, and sponsoring organizations.

www.nationalconcretebridge.org
The National Concrete Bridge Council (NCBC) website provides information to promote quality in concrete bridge construction as well as links to websites and publications of its members.

www.concretebridgeviews.com
This website contains 78 issues of *Concrete Bridge Views* (formerly *HPC Bridge Views*), an electronic newsletter published jointly by the FHWA and the NCBC to provide relevant, reliable information on all aspects of concrete in bridges.

Reorganization of Section 5

by Dr. Dennis R. Mertz

Since the adoption of the first edition of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications* in 1994, yearly interim revisions have been written to Section 5, Concrete Structures, by members of the AASHTO technical committee T-10, Concrete Design; researchers; or other friends of the committee. Much care was taken by the original National Cooperative Highway Research Program (NCHRP) Project 12-33 team to maintain organizational, philosophical, and technical consistency throughout the specifications. The yearly interim revisions since 1994, while well meaning, have not always maintained this consistency. Section 5 is also ready to be reorganized after these many years of interim revisions.

Transportation Pool-Funded Study TPF-5(271) began in January 2013 to develop the reorganization. Led by the Kansas Department of Transportation, 16 states and the Federal Highway Administration (FHWA) are funding the study. Modjeski and Masters Inc. is facilitating the reorganization for AASHTO T-10 with technical assistance from the American Segmental Bridge Institute (ASBI) and the Precast/Prestressed Concrete Institute (PCI). As part of this work

- stakeholders were surveyed regarding needs for reorganization or clarification,
- interim changes since 1994 were reviewed for consistency, and
- an annotated outline for the new section was developed prior to beginning the reorganization itself.

The reorganization is centered on distinguishing the design of B- (beam or Bernoulli) Regions or D- (disturbed or discontinuity) Regions. B-Regions are regions of concrete members in which Bernoulli's hypothesis of straight-line strain profiles—linear for bending and uniform for shear—applies. D-Regions are regions of concrete members encompassing abrupt changes in geometry or concentrated forces in which strain profiles more complex than straight lines exist. In addition to the reorganization and rewriting to reestablish consistency, certain design provisions are being enhanced including: the strut-and-tie method (STM), harmonized segmental design and non-segmental design, anchorage to concrete, improved distinction between pretensioned and post-tensioned provisions, and expanded durability provisions.

Article 5.13, Anchors, of the proposed reorganized Section 5 of the AASHTO LRFD Specifications has adopted by reference Chapter 17 of ACI 318-14 as the procedures to design, detail, and install anchors with amendments as appropriate for application to highway bridges. Thus, wholesale reproduction of the voluminous ACI provisions are avoided. Professor Ron Cook of the University of Florida, the primary author of ACI Chapter 17, assisted with the development of proposed LRFD Article 5.13.

The reorganized Section 5 is nearing completion after 15 drafts, as of this writing. AASHTO T-10 will make the final draft available to the Subcommittee on Bridges and Structures (SCOBS) for review in the fall of 2015. This huge proposed agenda item is on schedule for consideration by SCOBS at their summer 2016 meeting in Minneapolis, Minn. If adopted at that time, it would become a part of the next edition of the AASHTO LRFD specifications in 2017. AASHTO T-10 is also preparing an accompanying table cross-referencing the articles in the new reorganized Section 5 with the current section to assist with transitioning to the new format.