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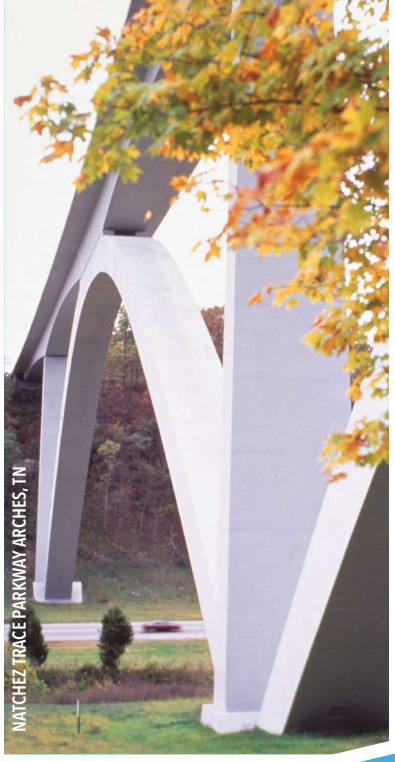
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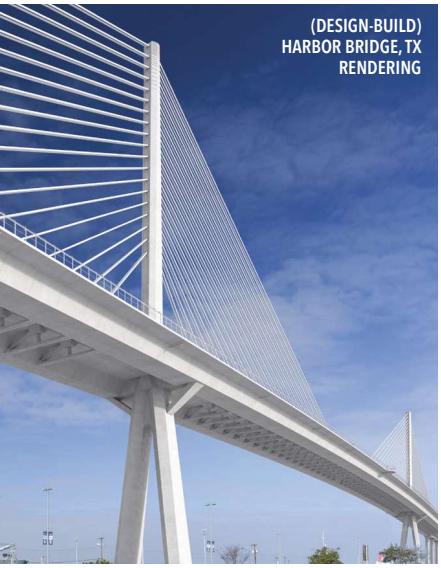
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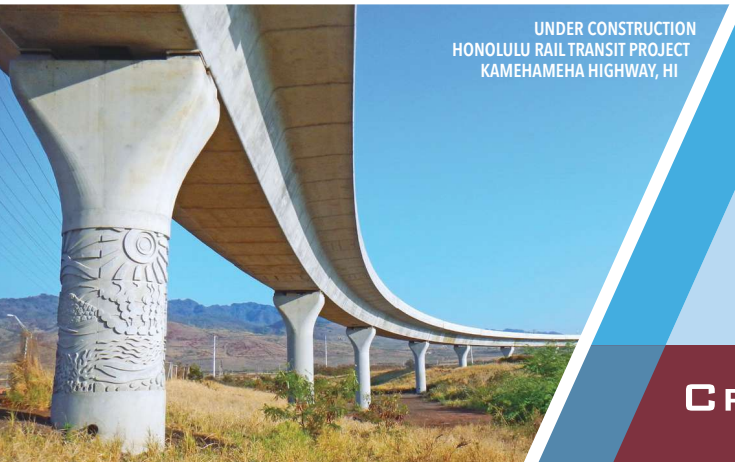
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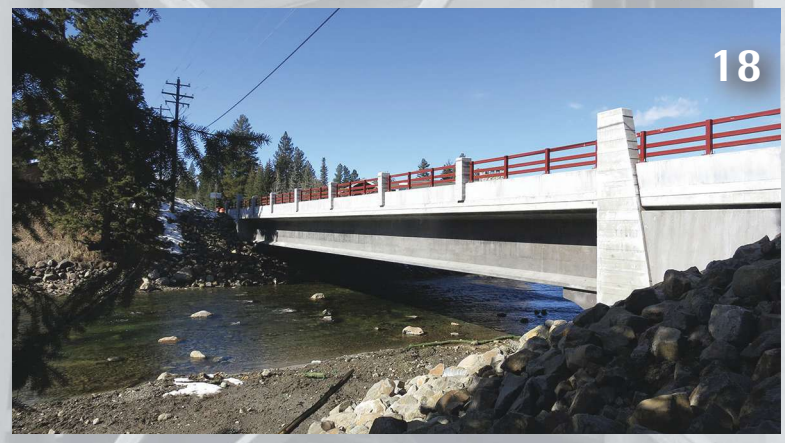

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Features

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Photo: PCI.

Make a New Year's Resolution: "Stay in Touch with Suppliers"

William Nickas, *Editor-in-Chief*

Every New Year brings resolutions and promises of change. Sixteen years ago, software venders were scrambling to be compliant with Y2K. And thank goodness, when the clocks rolled over at midnight, it was without any noticeable interruptions to information work flow. The Y2K-scare had garnered much attention and caused everyone to reengage with their computer venders. But it shouldn't take a scare to keep us in touch with our suppliers.

Recently, I was in Texas visiting five precast, prestressed concrete plants and two suppliers to the concrete industry. These plants hosted tours to 150 professionals who were looking to share information about the latest products and improvement opportunities in the industry. Key to those product and improvement opportunities were the relationships between the plants and their suppliers.

Company culture at Commercial Metal Corporation (CMC) in Seguin, Tex., embraces sustainability, from recycling scrap from cars and appliances, to sending only plastic and upholstery to the landfill. Our young guide at CMC was knowledgeable and passionate about his part in the company that cleans our communities and builds America, by supplying steel reinforcement and other building materials. It was obvious all employees were proud of the plant's history. The established work culture is to remain alert for safety first, but also quality. They even personalize the need for remaining safe and watching out for each other with family photos on the lockers, and in the hallways and gathering areas.

At the Capital Aggregates (CA) cement plant in West Lake Hills, Tex., two innovations were revealed. CA has started producing special Type III cements with either fly ash or slag (supplementary cementitious materials or SCMs) blended into the cement. The chief chemist shared that the blending process at the cement mill allows fly ash resellers and depots to open more of the supply due to their ability to carefully monitor the SCMs into the cement. This also saves storage and handling costs at the batch plant. In addition to normal quality control samples, CA is putting up trial batches and creating test pucks for research and development, but also as a client service.

The next innovation at CA was the diversion of carbon dioxide from the cement calcination process to a new startup factory to mineralize these emissions to make baking soda and bleach. The CA plant personnel introduced us to an executive with SKYONICS, who explained that they hope to be a for-profit technology soon. He explained that the cement plant is making about 1 million tons of cement a year and that they are diverting about 15% of the emissions at this point.

By staying in touch with their suppliers, the precast plants were able to expose us to the company culture at CMC and the innovations at the CA cement plant.

This calendar year brings the 10th anniversary of *ASPIRE*™ and our team wishes to thank all our supporters and advertisers but also all the authors of the interesting concrete bridge articles. Stay in touch with suppliers and integrate new products and solutions into your concrete bridge projects and then share with the readers of *ASPIRE* what you did.

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The SR 417 and Boggy Creek Road Interchange is the first standard delivery project in the country to incorporate curved precast concrete U-girders as the primary design. Photo: Parsons.

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During the December 2015 National Accelerated Bridge Construction Conference held in Miami, Fla., ESCSI offered a number of opportunities for attendees to learn about lightweight aggregate and lightweight concrete. A half-day seminar covered issues related to bridge design using lightweight concrete. During the Re-Use of Substructures in ABC Projects workshop, a presentation was given demonstrating how lightweight concrete can be and has been used for this application. Three papers were presented during the Advanced Materials for ABC session: recent changes in AASHTO LRFD Specifications regarding lightweight concrete, lightweight aggregate as geotechnical fill for ABC projects, and use of lightweight concrete for bridges moved into place. ESCSI has posted the workshop and technical presentations mentioned above on the home page of www.escsi.org.

The ESCSI YouTube channel contains a number of practical and informative videos on internal curing. Most recently, a two-part video series was posted on how to easily produce internally cured concrete. ESCSI videos can be accessed from the YouTube button on the ESCSI home page.

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Dr. Oguzhan Bayrak is a professor at the University of Texas at Austin. Bayrak received the University of Texas System Board of Regents' outstanding teaching award in 2012 and was inducted into the university's Academy of Distinguished Teachers in 2014.

Craig Finley Jr. is managing principal at Finley Engineering Group Inc. He has over 34 years as a consulting engineer, involved in the design, management, construction engineering, and inspection of a wide range of complex bridge types with spans from 40 to over 5000 ft.

Frederick Gottemoeller is an engineer and architect, who specializes in the aesthetic aspects of bridges and highways. He is the author of *Bridgescape*, a reference book on aesthetics and was deputy administrator of the Maryland State Highway Administration.

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Dr. Dennis R. Mertz is professor of civil engineering at the University of Delaware. Formerly with Modjeski and Masters Inc. when the *LRFD Specifications* were first written, he has continued to be actively involved in their development.

CONCRETE CALENDAR 2016–2017

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

February 2-5, 2016
World of Concrete 2016
Las Vegas Convention Center
Las Vegas, Nev.

February 8, 2016
2017 PCI Convention and National Bridge Conference
Call for entries deadline

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 2016 Concrete Convention and Exposition
Hyatt Regency & Wisconsin Center
Milwaukee, Wis.

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

April 24-26, 2016
2016 PTI Convention
Renaissance Long Beach Hotel
Long Beach, Calif.

May 16, 2016
54th Annual PCI Design Awards
Call for entries deadline

May 24-25, 2016
ASBI Construction Practices Seminar
Denver, Colo.

June 6-10, 2016
International Bridge Conference
Gaylord National Resort & Convention Center
National Harbor, Md.

June 26-30, 2016
AASHTO Subcommittee on Bridges and Structures Annual Meeting
Minneapolis Marriott City Center
Minneapolis, Minn.

June 30, 2016
PCA 2016 Concrete Bridge Award Competition
Call for entries deadline

July 25-29, 2016
PCA Professors' Workshop
PCA Campus, Skokie, Ill.

July 31-August 4, 2016
AASHTO Subcommittee on Materials Annual Meeting
Hyatt Regency Greenville
Greenville, S.C.

August 28-31, 2016
AREMA 2016 Annual Conference & Exposition
Hilton Orlando
Orlando, Fla.

October 6-9, 2016
PCI Committee Days and Membership Conference
Loews Chicago O'Hare
Rosemont, Ill.

October 23-27, 2016
ACI Fall 2016 Concrete Convention and Exposition
Marriott Philadelphia
Philadelphia, Pa.

November 7-8, 2016
ASBI 28th Annual Convention
Long Beach Convention and Entertainment Center
Westin Long Beach Hotel
Long Beach, Calif.

January 8-12, 2017
Transportation Research Board 96th Annual Meeting
Walter E. Washington Convention Center
Washington, D.C.

January 17-20, 2017
World of Concrete 2017
Las Vegas Convention Center
Las Vegas, Nev.

CONCRETE INNOVATION IN HAWAII

KSF's focus on creating innovative designs has led to several breakthroughs in concrete materials and construction that have been incorporated into many of the firm's projects

by Craig A. Shutt

The Kahoma Uka Bridge uses an innovative combination of precast, post-tensioning, and cast-in-place concrete elements to achieve the 360-ft span without intermediate supports. All Photos: KSF Inc.

KSF Inc. faces a variety of challenges designing infrastructure projects in Hawaii. Not only must it deal with the full range of seismic zones (and volcanoes) along the chain of islands, but many materials are either limited in supply or available only via overseas shipments. To offset these obstacles, KSF has devised innovative concrete mixture proportions that are becoming standard issue for its projects.

"Most of the projects we create are concrete structures," says Eric Matsumoto, vice president with the Honolulu-based firm. "Hawaii DOT [department of transportation] prefers concrete due to maintenance concerns in the corrosive environment. As a result, we have spent considerable time developing new concrete mixtures and techniques to help us handle the challenges."

Key insights come from Kirk Hashimoto, construction engineer, who previously worked in the concrete industry in engineering, marketing, and concrete mixture development. "He has helped to ensure we are creating the best concrete mixture for a project's needs," Matsumoto explains. "He is involved with the material usage from selection through placement and final finish."

KSF's success has derived in part from developing in-house expertise, Hashimoto notes. "The company's goal has been to provide core competencies in-house so we can take care of every aspect of a project. We are always looking to improve the materials we are using, and since most of our projects are concrete, that has been a point of focus."

The company also has an employee with past experience as a contractor to ensure every design maximizes constructability and efficiency. "He can give us insights into how the project will be constructed to help us create the most efficient and economical product," explains Matsumoto.

In part, that in-house expertise was dictated by the island's isolation, Hashimoto says. "Labor costs are high on the island, and specific skills are not always accessible. We have to be aware of the limitations and work around them. We developed a lot of in-house expertise to avoid having to fly in people to consult on projects."

Materials also are in limited supply. Hawaii basically has one cement supplier, who ships from Asia, and there are no fly ash or other supplementary cementitious materials locally available that meet ASTM requirements consistently. "Those materials are more expensive than cement," Hashimoto says. "Costs generally are higher here."

Innovative Materials

Those obstacles are in part the genesis for some of the company's innovation in concrete. Two innovations that were recently developed in a research program, which was performed in tandem with the Hawaii Department of Transportation (HDOT), were put to use on one recent project and now are used more often.

The first innovation developed structural concrete for use in drilled shafts with special characteristics for workability, nonsegregation,

bleeding, in-place concrete density, and mass concrete. "Concrete in drilled shafts in Hawaii historically has demonstrated undesirable behavior," explains Hashimoto. The bleed water migrates to the top of the shaft during placement, resulting in lower-density concrete at the top of the shaft. That often interferes with the tie-in to the structure, creating a weak point and adding stress. The combination of materials available in Hawaii also increases the heat of hydration, especially with larger shafts. The increased heat can reduce long-term durability.

KSF's engineers experimented with options and devised a concrete mixture that would account for the typical characteristics of local concrete to reduce heat of hydration and bleed water. Lower heat minimized internal cracking, and the reduction of bleed water resulted in more uniformity of the concrete. The result was a drilled shaft superior to previous designs.

'Our goal was to create a tougher mixture that exhibited less shrinkage.'

The second innovation focused on meeting challenges presented by decks with greater water-cementitious materials ratios, which could lead to shrinkage and cracking. "Our goal was to create a tougher mixture that exhibited less shrinkage," says Hashimoto. "We looked at mixture options and optimized them to reduce the factors causing problems. Then we looked at blends of fibers that could increase toughness."

The result was a high-performance concrete that contains entrained air and a combination of macro and micro fibers that minimizes shrinkage and creep characteristics typically encountered with local deck concrete mixtures. The materials and proportions minimized bleeding and improved toughness of the concrete.

Both innovations were applied to the North-South Road bridges on the Island of Oahu, which consist of twin, 165-ft-long, single-span, integral-abutment bridges. They feature 5.5-ft-deep, post-tensioned spliced girders and precast concrete deck panels with a high-performance concrete deck topping. Five-ft-diameter drilled shafts at each abutment support the structures.

The North-South Road project featured several innovative concepts, including a high-performance concrete deck and a new composition for drilled-shaft concrete. The concrete diaphragm is located at the quarter-point splice between girder segments.

The North-South Road on Oahu features twin 165-ft-long, single-span, integral-abutment bridges with post-tensioned, spliced girders and precast concrete deck panels.

"This project provided a good opportunity to try both ideas," says Hashimoto. "It was the first one we'd worked on for some time that was not a repair or replacement project, which gave us an opportunity to design strictly for the site. It was a chance to try these fresh ideas, and that gave the research a major push."

KSF used a different shrinkage-reducing admixture for its work on the Kii Bridge, an 80-ft-long, single-span, integral-abutment bridge. The superstructure consists of 2-ft-deep, precast, prestressed concrete planks with a 5-in.-thick topping and is supported by 3-ft-diameter concrete drilled shafts. The admixture's impact was examined with laboratory tests, field measurements, strain-gage readings, and finite-element analysis. Staged construction, post-tensioning, and soil-structure interaction were modeled with a finite-element program. An additional research project studying soil-structure interaction also was performed.

Contractor Input Critical

Creative results are often achieved in conjunction with contractors, a process enhanced by design-build method and value engineering. "We always work closely with the contractor from conceptual phase to project completion. Good relationships and collaboration result in better designs and construction

The Kii Bridge, an 80-ft-long, single-span, integral-abutment bridge on Oahu, has a superstructure featuring 2-ft-deep, precast, prestressed concrete planks.

in addition to quick resolutions when issues arise," says Matsumoto.

One recent design-build project that benefited from close cooperation was the Kahoma Uka Bridge that traverses the Kahoma Stream in West Maui. As the project was being designed, it was discovered that the path included 30 acres of historical agricultural terraces. To respect this culturally significant site, native Hawaiian groups and lineal descendants were consulted.

With their input, the bridge was designed to realign the roadway away from the terraces. That required the previously straight bridge to have a horizontal curve with a 1200-ft radius and superelevation. The design features precast, post-tensioned, and cast-in-place concrete elements that allowed production of a 360-ft-long curved bridge with no intermediate supports.

The superstructure consists of precast concrete U-girders, stay-in-place precast concrete deck panels, and a cast-in-place concrete topping. Six lines of U-girders placed longitudinally between abutments were framed with eight chords to produce the curved, horizontal alignment.

The bridge can expand, contract, and rotate without imposing significant horizontal loads and moments on the structure and foundation with the aid of friction-pendulum bearings placed at the top of 9-ft-high pedestals at each footing. Each bearing has an 88-in. effective radius of curvature that results in a dynamic period of 3 seconds.

The design allowed the bridge to be built without placing any piers in the stream. "That saved considerable time, as disrupting the stream would have required permitting that would have slowed the schedule," Matsumoto says. "It was an efficient design for many reasons."

Environmental Concerns Common

Environmental concerns and permitting that can slow construction are becoming more commonplace, Matsumoto notes. "We are encountering obstacles of different types more regularly that have to be worked out ahead of time," he says. "Our goal is to resolve any challenges that arise in advance. In Hawaii, the terrain always comes into play in our designs."

According to Matsumoto, some of those issues arise due to the seismic activity on the chain of islands, which includes active volcanoes. "The Hawaiian island chain was created by hot-spot volcanism. The oldest island is located in the northwest portion of the chain while the youngest is in the southeast. The island of Hawaii has an active volcano and is subjected to frequent seismic activity. Due to this hot-spot volcanism, the State of Hawaii has seismic zones ranging from 1 to 4. Thus, we have varying seismic issues to deal with in each project."

Many locations are also in mountainous areas along winding roads. That can restrict the length of precast concrete girders that can be shipped, requiring splicing the segments along with creative techniques for placing them. "Access to sites can be an issue," says Matsumoto.

An example is the Kealakaha Stream Bridge on the Hawaii Belt Road on Hawaii, which traverses a 165-ft-deep and 610-ft-wide ravine with a main span of 360 ft. The bridge is close to an active volcano, which subjects the area to high seismic activity.

The original design was value-engineered by KSF when the contractor realized that building a curved segmental box structure with a travel way superelevation of 6.2%, shoulder slope of 2%, and no topping would be very difficult to construct. KSF redesigned the structure to incorporate Washington state's new precast concrete "super girders" to span 100 and 205 ft along with 150-ft-long cast-in-place box girders above the piers and a cast-in-place concrete deck.
The W95PTG precast concrete girders

The Kahoma Uka Bridge was redesigned after 30 acres of historical agricultural terraces were discovered in the bridge's path.

The 360-ft-long main span of the Kealakaha Stream Bridge crosses a 165-ft-deep ravine. The long span is achieved using a combination of cast-in-place concrete box girders over the piers and 205-ft-long spliced precast concrete girders that close the gap between them.

The Kealakaha Stream Bridge features 205-ft-long W95PTG "super girders" that were spliced and post-tensioned using 50-ft-long segments. In this photo, the completed spliced girder has been moved across the ravine using a launching truss and is being set in place.

were shipped from Spokane, Wash., to the site, but due to the winding roads, the maximum length for any segment was 50 ft. The segments were spliced and post-tensioned at the site.

The 50-ft-long girder segments were placed on a custom rail system of wide-flange members and rollers where they were spliced and post-tensioned. A steel stiffening truss was attached to the middle 90 ft of the 205-ft spliced girder and two four-strand tendons were placed in the girder top flange and stressed to improve lateral stability during installation. The assembled girder was then pulled across the ravine on a launching truss. Once in position, the girder was attached to strand jacks that lowered it into place. When all of the girders were in place, they were spliced to the box girders and post-tensioned.

"It provided a creative way to erect girders over a deep ravine in a restrictive location," says Matsumoto. "We worked closely with our in-house contractor to produce the plan and to ensure it was safe and efficient to work from the top down."

KSF's value-engineering redesign also included friction pendulum seismic isolation bearings on each abutment and pier, their first use in the state. The bearings provide displacement capacities of 12 in. at abutments and 10 in. at piers. The seismic isolation bearings lengthen the natural period of the isolated structure which reduces the lateral loads transmitted to the structure. This approach allowed the team to raise the elevation of the footing, decrease the size of drilled shafts, and eliminate costly soil nail walls around the footings. "In retrospect, without the redesign, the contractor would have lost a considerable amount of time and money, because installing the drilled shafts was more complex and costly than anticipated."

Reinforced Soil

Another implementation of new technology arose with the Kauaula Stream Bridge, which was constructed in conjunction with a University of Hawaii research project on geosynthetic reinforced soil (GRS) integrated bridge systems (IBS). The first bridge in the state to use this process, its abutments feature biaxial, woven polypropylene geotextile, compacted granular fill material, and concrete masonry units for the facing. "This project provided a good opportunity to use the GRS-IBS system and monitor its behavior," says Matsumoto.

The 112-ft-long, 47-ft-wide superstructure comprises precast concrete, post-tensioned U-girders, along with stay-in-place concrete deck panels, and a cast-in-place concrete deck. The installation required neither heavy equipment nor specialized labor, Matsumoto notes. KSF's in-house contractor devised a scheme to place the U-girders that only required steel beams, rollers and hydraulics jacks.

This approach has proven to be effective and economical, he says. "There is a time and place for all of these techniques that we use, and we want to have many options to deal with any variables. Placement of the Kauaula girders proved to be an effective method of erection. We'll continue to propose it when it will help the contractor."

Always Innovating

The firm is analyzing other innovative techniques, including some focusing on ultra-high-performance concrete, modified concrete mixtures, lightweight concrete, and corrosion inhibitors. "We are working on some new ideas, but nothing is at a stage we can talk about," says Hashimoto. "Overall, we are always looking at developing and refining materials to provide more durability with lower impact, especially as service lives are extending from 50 or 75 years to 100 years."

Matsumoto agrees. "Our research and investigation of new techniques will continue to drive our company. We are always looking at new methods and materials. Our goal is to provide clients with the most innovative, economical, and elegant designs possible."

22 Years of Service

KSF Inc. was established in January 1994 by Yuji Kasamoto, Myles Shimokawa, and David Fujiwara, who is now president. The firm originally designed both buildings and bridges but has evolved to focus on infrastructure and especially concrete bridges.

The firm has 18 staff members, including seven licensed and six unlicensed engineers. KSF Inc. also has more than 70 years of collective construction experience. Among its projects are bridges, concrete pavements, underground utility boxes and culverts, and retaining structures.

EDITOR'S NOTE

A project article on the Kahoma Uka Bridge appeared in the Summer 2014 issue of ASPIRE,™ *and the Kealakaha Stream Bridge was featured in the Summer 2010 issue. A list of references related to the projects and research mentioned in this article can be found on the* ASPIRE *website.*

Kahoma Uka Bridge

Kahoma Uka Bridge

Kealakaha Stream Bridge

Lahainaluna Grade Separation

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A SILVER BULLET FOR U.S. PRECASTERS

The time is ripe to make the investment in reusable segmental forms

by Craig Finley Jr., Finley Engineering Group Inc.

A combination of ingenuity, advanced planning, and open-minded teamwork among the owner, general contractor, and the engineer resulted in substantial construction cost savings and a much quicker schedule on the first (and three additional) bridge project as part of Israel's major highway development projects. It also gave Danya Cebus, which was the general contractor and precaster, a significant competitive advantage to win additional work.

The key component for this country's developing infrastructure systems was designing numerous segmental bridges to the same cross section and erection method that best met the contractor's expertise and available equipment. While not new to Europe and other parts of the world, the use of standard precast machines for multiple projects was a novel, and some considered risky, approach in Israel. Although designing to standard precast box cross sections had been proven to have limited interest in the United States in the past, the time may be ripe to reconsider.

The Initial Challenge

The precasting machines were originally designed for the $273 million Ein Ha'Kore Interchange on Highway 431, part of the Cross-Israel Highway, but now more than 1100 segments on three additional bridge projects have been built out of the same two precasting machines. The general contractor knew that efficient construction methods were a key element for success, and they were looking to apply innovative techniques to reach their goal.

Finley Engineering Group brought their international experience to the project to provide a simple solution to a complex problem. All the involved entities—including governmental officials, the owner's engineering representatives (who had completed the preliminary design), and the contractor's ownership group—considered the alternatives and were open to the innovative idea. They agreed that precast concrete segmental bridges, using external tendons with diabolos, were a safe, cost-effective, and reliable long-term solution for this project.

The use of external tendons provided simplified precasting details, rapid erection procedures, and improved long-term durability. They also added technical advantages in the bridge design, such as increased ductility for flexural moment resistance and a significant reduction in principle tensile stresses in the box-girder webs. These benefits allowed for longer, constant-depth span lengths for the bridges, while still meeting the interchange design requirements. External tendons also provided savings on long-term maintenance costs allowing for simpler tendon replacement and periodic inspections.

Casting yard. Photo: Finley Engineering Group Inc.

As a result, the general contractor, who was an experienced precaster but had no segmental bridge experience, was able to win the design-build-operate-transfer project based on its low bid, and finish ahead of the ambitious schedule. This was critical to

Precasting machine. Photo: Finley Engineering Group Inc.

Table 1. Project Details				
Project	Span Length	Width	Number of Segments	Number of Bridges
Highway 431	120 to 200 ft	41 ft	329	5
Section 18	70 to 200 ft	42 ft	352	4
Benyamina Bridges	130 to 220 ft	38 ft	262	2
Highway 722	110 to 200 ft	66 to 100 ft	248	2
		Total	**1191**	**13**

Table 2. Comparison of Quantities for External Versus Internal Continuity Post-tensioning		
Quantity	External	Internal
Web thickness	15 ¾ in.	19 ½ in.
Cross-sectional area	69.97 ft^2	77.38 ft^2
Cantilever post-tensioning	2.67 lb/ft^2	4.23 lb/ft^2
Continuity post-tensioning	3.32 lb/ft^2	2.14 lb/ft^2
Total post-tensioning	5.99 lb/ft^2	6.37 lb/ft^2
Vertical web post-tensioning bars	–	Segment 1
Web principal tensile stress	216 psi	285 psi

the group's public-private partnership concession agreement and project financing goals.

The general contractor invested in the more robust and flexible European style forms with the intention of searching out projects where these forms could be used again, providing the contractor with a competitive advantage.

Key to the general contractor's success was establishing the casting yard in a central location, which reduced transportation of precast concrete segments to no more than 75 miles. They used lessons learned from the first project and expanded their knowledge base for use on each successive project. The general contractor's precast segmental experience became an institutional capability and its first move into this market helped to make them the number one precaster in Israel today. Tables 1 and 2 show the flexibility and variability that standard concrete forms can offer.

Future Successes and Lessons Learned

By taking a long-term view—employing expertise where needed to maximize its strengths and having a willingness to employ innovative risk—the general contractor was able to aggressively target, bid at the lowest costs, and win three additional Cross-Israel Highway projects. All three of these additional projects used the standard concrete forms and construction methods that were employed on the Highway 431 project. They were the Highway 722 Bridge, the Section 18 Interchange (four bridges), and the Benyamina Bridges (two bridges). In all, the two precasting machines were used to build 1100 segments on 13 bridges totaling $471 million in construction costs.

The general contractor credits the following to its long-term success:

- Big picture, long-term approach to its projects, processes, and business development, rather than the norm of a project-specific focus
- Willingness to take on innovation risk to create a market leader position
- Implementation of casting and erection standards with current and future projects in mind
- Commitment to continuous quality improvement with each project, applying lessons learned incrementally to all future projects
- Substantial savings realized through the reuse of casting machines and equipment

The Question—Why Not in the United States?

For the United States to remain competitive in the global marketplace, it has to continue to find, develop, and employ innovative methods and ideas.

Segment erection. Photo: Danya Cebus.

Segmental bridges are prominent in the United States and are a construction methodology with design implications. Standard segment details are defined in the ASBI *Construction Practices Handbook for Concrete Segmental and Cable-Supported Bridges*. For instance, the key features include balanced cantilever details, 45 ft segment widths, 7 to 10 ft segment depths, and spans up to 200 ft.

The United States-style forms for a typical precast concrete segmental bridge can cost upwards of $400,000 each, with two forms usually required on a project, and designing and manufacturing forms are a time consuming portion of a project. In the United States, forms are built by the contractor for a specific project, used one time, and discarded, adding approximately $1 million dollars to the contractor's bid.

Precasters are in a unique position to expand their product line, and they can benefit greatly by offering contractors and owners a lower cost alternative while maintaining or improving quality and maintenance standards of our country's bridges. Reuse of forms also offers a more environmentally friendly option by using fewer resources and reducing the overall waste stream attributed to the project.

Precast concrete segmental bridges are widely accepted in the United States. The time has come for the United States precasters to make the investment in European-style forms to extend their product line and pursue these projects.

SR 417 and Boggy Creek Road Interchange

Florida's first curved precast concrete U-girder bridge

by Thomas W. Stelmack, Thomas E. Davidson, and Kristian Forars, Parsons

Orlando International Airport (OIA) has become the fourth busiest domestic destination in the United States due to Orlando, Florida's, many tourist attractions, job opportunities, and climate. With the steady increase in travelers, the interchange for the south entrance to the airport has become more congested. For many years, planners with the Central Florida Expressway Authority (CFX) have known the need for improving the movements from State Route (SR) 417, part of CFX's tollway around Orlando, to Boggy Creek Road and the south access road to OIA. The third and final phase of the interchange includes two direct flyover movements:

- Ramp H carries drivers from northbound SR 417 over Boggy Creek Road and SR 417 to the south access road of OIA
- Ramp I provides the opposite movement, carrying drivers over Boggy Creek Road and SR 417 from the south access road of OIA

The horizontally curved superstructures for Ramps H and I were originally designed with both steel trapezoidal box girders and precast concrete curved U-girder alternates. The design for the U-girder alternate used PCI Zone 6 concept plans adopted by the Florida Department of Transportation (FDOT) and was completed in less than 12 months. Bids for the precast concrete curved U-girders were ultimately accepted in October 2013, with the low bid of $71 million ($33.78 million for the two ramp structures) estimated to be a savings of $7 to $9 million over a steel design.

Aerial view of both Ramps H and I. Photo: SEMA Construction.

Design Details

Geometry

The project has two flyover ramps that were designed using curved precast concrete U-girders. Ramp H is approximately 55 ft above ground surface at its highest point and consists of three continuous units for a total length of 2708 ft. Each unit is comprised of several spans ranging in length from 141 to 216 ft. Ramp H has a minimum horizontal radius of 1273 ft and the width of the bridge deck is 45 ft 3½ in. with two travel lanes. Ramp I crosses over Ramp H, rising 90 ft above ground surface at its highest point and has two continuous units with a total length of 1411 ft. It has spans ranging in length from 177 to 220 ft, with a minimum horizontal radius of 955 ft, a bridge deck width of 36 ft 3½ in., and one travel lane.

profile

SR 417 AND BOGGY CREEK ROAD INTERCHANGE / ORLANDO, FLORIDA

BRIDGE DESIGN ENGINEER: Parsons, Denver, Colo. and Orlando, Fla.

PRIME CONSULTANT: Dewberry, Orlando, Fla.

INDEPENDENT CHECK ENGINEER: URS Inc., Tampa, Fla.

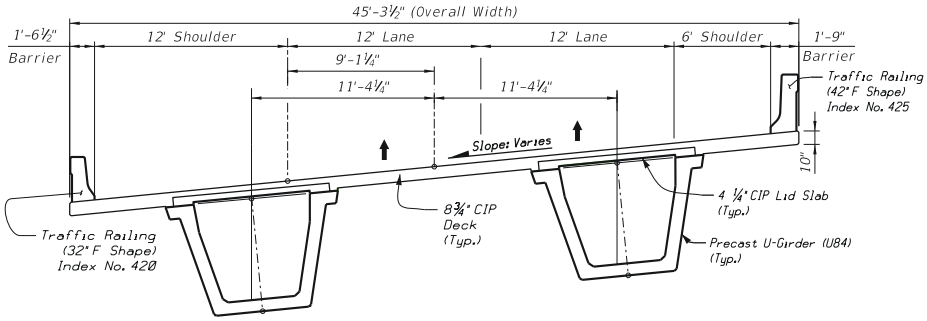
CONSTRUCTION ENGINEER: Summit Engineering Group, Littleton, Colo.

CONSTRUCTION INSPECTION: A2 Group Inc., Orlando, Fla.

POST-TENSIONING INSPECTION: FIGG, Tallahassee, Fla.

PRIME CONTRACTOR: SEMA Construction, Orlando, Fla.

View of curved forms with reinforcing steel and core form in place. Photo: Parsons.



Typical superstructure section for Ramp H. Ramp I is similar, but is 36 ft 3 ½ in. wide. Figure: Parsons.

View of typical girder end details, including thickened webs, top strut, shear keys and reinforcing steel to be tied with closure placement reinforcement. Photo: Parsons.

First Ramp H girder being lifted into place, supported by abutment and shoring tower. Photo: A2 Group.

Substructure

As the ramp structures were designed initially as steel box girders, the CFX decided not to change pier locations or foundation sizes, so the precast concrete U-girder alternative utilized the previously completed steel alternative substructure design. The prescribed span lengths were not a problem for the concrete design and the foundation designs also worked well with the concrete alternative. The foundations are driven precast, prestressed concrete piles with cast-in-place concrete footings, supporting cast-in-place, reinforced concrete piers and caps. Most piers are radial, with a few being skewed to avoid roadways or Boggy Creek.

Superstructure

The cross sections of both flyover ramps consist of two PCI 84-in.-deep U-girders with an 8¾-in.-thick concrete deck and 10-in.-thick overhangs. One of the challenges of a precast concrete U-girder structure is the heavy self-weight of the segments. To keep the segments to a manageable length and weight for lifting and transport to the site, the spliced and post-tensioned method of construction was used. The bridge was broken up into segments, with one segment over each pier and a segment in between. The length of the girder segments was limited to a maximum of 110 ft in order to enable each segment to be transported, although the maximum length was later increased to 115 ft 4 in. by the contractor. The basic sequence includes precasting the curved U-shaped segments, supporting each segment on temporary shoring towers, and splicing the sections together using post-tensioning. Gaps for the closure placements between each of the curved segment sections, which consist of cast-in-place concrete, are typically 2 ft wide. Post-tensioning tendons run through internal ducts from the beginning to the end of each unit, connecting all of the U-girder segments in the unit when stressed.

Due to the complex geometry, numerous phases of construction, and changes in support/loading conditions of the segments, a three-dimensional computer model was developed that included a detailed, time-dependent, staged construction analysis with both girder lines and deck slab. This analysis ensures that the segments meet all design requirements during each phase of construction and also allows for accurate calculation of camber.

The U-girder section has a web thickness of 10 in. to accommodate the 4-in.-diameter ducts for the 15-strand tendons. A variable thickness bottom flange is used throughout the girder line and varies from a typical value of 9 in. to 1 ft 9 in. over the piers to provide additional capacity to meet allowable stresses over the pier. The open U-girder section ultimately requires a cast-in-place lid slab to increase the torsional resistance of the

CENTRAL FLORIDA EXPRESSWAY AUTHORITY, OWNER

PRECASTER: Dura-Stress Inc., Leesburg, Fla.—a PCI-certified producer

POST-TENSIONING CONTRACTOR: Freyssinet Inc., Sterling, Va.

U-GIRDER FORM SUPPLIER: Tucker's Machine & Steel Service Inc., Leesburg, Fla.

BRIDGE DESCRIPTION: 2708-ft-long and 1411-ft-long curved, spliced, and post-tensioned precast concrete U-girder flyover ramp structures

STRUCTURAL COMPONENTS: Ninety-two 84-in.-deep U-girders with a 8¾-in. cast-in-place concrete deck; cast-in-place pier caps, columns, and footings; and 24-in.-square driven precast, prestressed concrete piles

BRIDGE CONSTRUCTION COST: $33,775,000 ($194 per ft^2)

section prior to deck placement and is constructed after erection to limit the transport weight.

The post-tensioning is made up of three types of tendons. Each segment has either four-strand flat ducts or 12-strand round polyethylene ducts in the bottom flange depending on girder segment length/location and are used to enable the section to carry its self-weight. The U-girder section also has four draped continuity tendons (each consisting of 15 strands with 19-strand anchors) running through each girder web from beginning to end of the unit. The continuity tendons stitch all the girder segments together and the profiles along the girder lines are designed to keep the stresses (Service III tension and Service I compression) within allowable limits during all stages of construction and during service. The precast concrete U-girder top flange provides room for top post-tensioning tendons if needed for tensile stresses over the piers after erected.

View of erected Ramp I girders spanning above Ramp H. Photo: Parsons.

Construction Details

Being the first precast concrete curved U-girder bridge constructed in Florida, new forms were purchased by the precaster for the project. The curvature is obtained by using short straight sections with small angle breaks between the sections, for both external and internal forms. The forms are designed to handle different radii and adjustment is relatively simple. While the design plans used the same radius for each adjacent girder within spans, the contractor modified the radius of several segments slightly to further minimize the number of form radius changes.

Reinforcing bar cages, utilizing welded wire mesh, were built outside the forms and the completed assemblies lifted

First Curved Precast U-girder Bridge for Design-Bid-Build Project

The SR 417 and Boggy Creek Road interchange is a milestone project in that it is the first standard delivery project in the country to incorporate curved precast concrete U-girders as the primary design. Previous projects constructed in Colorado have all been a result of value engineering redesigns or an alternate design allowed by the contract documents and completed by the contractor. As such, the SR 417 and Boggy Creek Road interchange is the first project bid with full and complete construction documents and specifications as part of the bid package.

Spliced and post-tensioned precast concrete curved U-girders can be compared to the development of precast concrete segmental box girders in the United States. Both are phased construction methods utilizing multiple segments to make up a span requiring time-dependent phased analyses. For precast concrete segmental construction, as owners became more comfortable with the method, segmental designs became more common in standard delivery bid packages when it made sense, due in part to the Federal Highway Administration mandate for two complete alternative designs for projects costing more than a certain value. The primary difference with curved U-girders is that some states have selected standard sections to be used, which will allow precasters to purchase the forms with the assurance that they can be used for multiple projects, similar to standard precast concrete bulb-tee girders.

However, it is imperative that, similar to segmental designs, the contract documents need to provide complete details to construct the bridge and the assumptions made by the designer, while allowing the contractor the freedom to make modifications based on fabrication and construction preferences. The items that can be modified and the responsibilities of the contractor when making modifications need to be clearly stated in the contract documents. Listed below are items that should be included in contract documents and items that should be the contractor's responsibility.:

Designer's responsibility:

- Girder splice locations
- Post-tensioning layout and anchor details
- Diaphragm details
- Girder geometry, including camber and elevations
- Erection sequence

Contractor's responsibility:

- Temporary supports
- Lifting details
- Girder stability and temporary bracing

Girder at site ready to be lifted into place. Photo: Dura-Stress Inc.

The items shown in the plans that are allowed to be modified—and if the changes are made, what design and submittal requirements are subsequently required of the contractor for the changes—should be discussed with the owner and decisions made based on their preference. This will help to allow the contractor to submit its best bid and also minimize any confusion about responsibilities and requirements during construction. For this owner, producing standard project delivery packages with precast concrete curved U-girders was the best way to introduce the product to their region and to get projects built using them.

in one piece and placed in the forms. Additional longitudinal reinforcing bars were used in the webs and bottom slabs of the girders to address lifting and handling operations. To allow quicker girder removal from the forms, stresses were checked considering only the reinforcing steel, allowing the bottom flange tendons to be stressed in the yard later once the required concrete strength was achieved.

Shipping of the girders (maximum weight of 340 kips) was by multi-axle trucks to the construction site about 40 miles from the fabrication yard. Girders were shipped at night to arrive early in the morning so that placement could occur during lane closure windows.

Girder segments were supported at each end by steel shoring towers and, at the ends of the units, by an abutment or pier. Segments initially spanned over permanent piers and were adjusted to the necessary elevations. Bearings were then grouted in at the permanent piers and 2-ft, cast-in-place closure joints that accommodated construction tolerances were made between segments. Post-tensioned diaphragms were cast at the piers to tie the two-girder system together and transmit loads to the bearings.

One of the continuous post-tensioning tendons was stressed in each web prior to casting a lid slab, which closed the top of the boxes for torsional rigidity. This initial post-tensioning prevented cracking in the closure placements when the 4¾-in. lid slab was subsequently cast on permanent metal deck forms. Thereafter, the remaining three post-tensioning tendons were stressed. After post-tensioning and grouting was completed, shoring was removed and the girders were ready for placing the deck slab as with any other composite continuous girder system.

Summary

The design and construction of the SR 417 and Boggy Creek Road interchange has introduced a new girder type to the state of Florida. Where curved structures are required and aesthetics are important, the PCI U-girder provides an alternative to the steel trapezoidal box girder, increasing competition and allowing for more competitive pricing. This new system brings the advantages of standard precast concrete construction, including durability, quality and speed of construction, and minimal disruption to the area below the bridge, to curved concrete U-girders.

Mr. Donald W. Budnovich, resident engineer for CFX comments, "The Central Florida Expressway Authority is excited to provide our customers and the Central Florida Region a more efficient means to enter the Orlando International Airport through the construction of this interchange. The innovative design employing the post-tensioned curved concrete U-girders, provides a durable and cost-effective project that we expect will serve Central Florida for decades to come."

Thomas W. Stelmack is west sector technical director and Kristian Forars is a senior project engineer with Parsons in Denver, Colo. Thomas E. Davidson is area manager with Parsons in Orlando, Fla.

EDITOR'S NOTE

For more information on the design consideration used for this project, see the article on p. 30.

AESTHETICS COMMENTARY

by Frederick Gottemoeller

Multi-level, high-speed interchanges between limited access roadways can be visually confusing places. Cars are moving fast along continuously curved paths, and paths merge and split as they go. The piers and abutments of the various bridges limit drivers' views of the paths ahead. Even if their views meet the minimum sight distances set by safety criteria, drivers are still left with a sense of unease about what lies ahead. If some of the intervening piers and abutments can be eliminated, drivers can better see through the interchange, alleviating their unease.

So, the first benefit of the precast concrete curved U-girders at Boggy Creek is that they allow longer spans and thus fewer piers. The piers are set well back from the edges of the under-passing roadways. The piers themselves, for the most part, use single stems that have minimal effect on the view. Drivers can see a long way ahead, with no interference from intervening piers. As an additional benefit they can better enjoy the passing landscape.

The second benefit of the precast concrete curved U-girders is that they are seen as one long, continuous curved girder. In a high speed highway environment, visually simpler is almost always visually better. The pier details enhance this result. The girders sit up above the pier caps on low concrete blocks. From many angles you can actually see sky between the bottom of the girder and the top of the pier caps (See cover photo). The bearings are so small compared to the length of the pier caps that the girders seem to be supported on pinheads. The full sweep of the girders is visible. They almost look like they are floating on air.

As a further enhancement, coating the girders with a color that contrasts with the piers and deck emphasizes the curvature and continuity of the girders. To top it all, the color chosen is an excellent complement to the lush Florida vegetation.

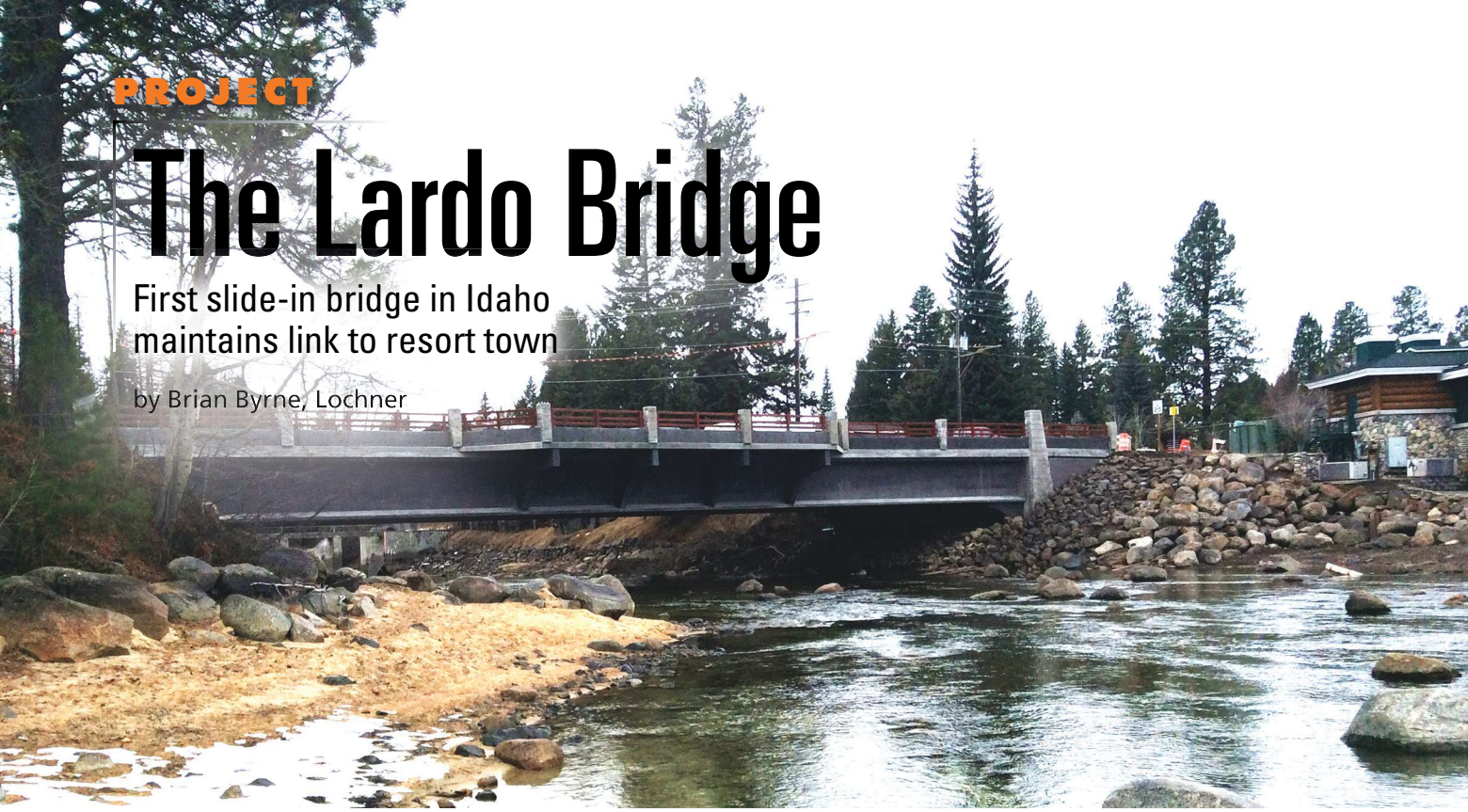
High-level flyover ramps are always the most prominent feature of these interchanges. It is always worthwhile to give some thought and, yes, even spend some money, to improve their appearance. The appearance of the whole interchange will benefit. And where, as here, the interchange is a gateway to a whole region, improving the appearance of the interchange will enhance visitors' impression of the entire region.

PROJECT

The Lardo Bridge

First slide-in bridge in Idaho maintains link to resort town

by Brian Byrne, Lochner



The completed north elevation of the Lardo Bridge. Photo: Ralph L. Wadsworth Construction.

The Lardo Bridge replacement, in the resort town of McCall, Idaho, is Idaho Transportation Department's (ITD's) first implementation of slide-in-bridge-construction (SIBC) technology for a permanent superstructure. It also was administered as their first federal aid design-build contract. The new single-span, 155-ft-long precast concrete girder bridge replaces an existing 80-year-old, five-span bridge carrying State Highway 55 (SH 55) over the North Fork of the Payette River at the outlet of Payette Lake.

McCall is a beautiful resort town of about 3000 permanent residents surrounded by the Payette National Forest and is reachable from Boise in just over 2 hours. SH 55 runs north into the busy downtown area of McCall and then cuts westerly along Payette Lake. Much of the lodging, restaurants, tourist attractions, and scenery are located along SH 55 and the Lardo Bridge is the connection between these points of interest. Therefore, detouring through-traffic during the peak tourist season was not acceptable for the replacement of the bridge.

McCall's tourist season extends from Memorial Day through Labor Day and picks up again from Thanksgiving through New Years for snowmobilers and skiers. With SH 55 as the main route along the lake, ITD wanted to minimize delays on SH 55. Therefore, ITD let this project as a design-build with A + B bidding to encourage the design-build teams to minimize construction durations and minimize impacts to the peak tourist season. To maintain connectivity along SH 55 between lodging, restaurants, and recreation areas, both lanes on the existing bridge remained open during the summer while the new structure was built to the side. The superstructure and stub abutments were slid in the fall before tourists returned for winter outdoor activities.

Superstructure and Belvedere

The structure's out-to-out width is generally 54 ft, which is 16 ft wider than the existing bridge. The clear opening between abutments met the 152-ft requirement for hydraulics, and resulted in a single-span length between centerline of bearings of 155 ft with no skew. The superstructure consists of six, 90-in.-deep, Utah bulb-tee (UBT-90), precast, prestressed concrete girders. The bridge has an asymmetric cross section with a wide cantilevered mid-span viewing platform, which was termed a belvedere, allowing

The concrete girders also provide an aesthetic that compliments, rather than distracts, from the natural scenic setting.

profile

THE LARDO BRIDGE / MCCALL, IDAHO

BRIDGE DESIGN ENGINEER: Lochner, Meridian, Idaho and East Hartford, Conn.

PRIME CONTRACTOR: Ralph L. Wadsworth Construction, Draper, Utah

PRECASTER: Forterra Structural Precast (formerly Hanson Structural Precast), Salt Lake City, Utah—a PCI-certified producer

pedestrians to stop and enjoy the vista of the lake and surrounding mountains.

The request for proposal (RFP) required the belvedere to be a minimum of 50 ft long and cantilever 8 ft out beyond the 10-ft sidewalk on the north side. To support this design load, corbels spaced at 15 ft 4 in. centers were cast against the exterior face of the bulb-tee girder. In line with the corbels, 12-in.-thick intermediate concrete diaphragms were provided in the first bay backed up by 6-in.-thick intermediate concrete diaphragms in the adjacent four bays.

To accommodate the eccentric loading of the belvedere slab overlook, an asymmetrical girder arrangement was provided. The typical section layout was analyzed with a grid model to determine the distribution of loads from the overlook. The girders in the two northern bays are spaced at 7 ft 6 in. on center and the next three bays are 10 ft 4 in. The normal deck overhang is 3 ft on the north side and 5 ft on the south side.

The viewing platform is created by cantilevering from the edge girder at mid span. Photo: Ralph L. Wadsworth Construction.

During the proposal phase of the project, we looked carefully at a structural steel option for the superstructure; however, the precast concrete UBT-90s were significantly less expensive. The concrete girders also provide an aesthetic that compliments, rather than distracts, from the natural scenic setting.

Belvedere overlook with concrete patterns in sidewalk. Photo: Lochner.

Girder Transportation and Erection

Transportation of girders was a significant concern with their 156 ft 6 in. length, their weight at 168,000 lb each, and their depth of 7 ft 6 in. The precaster is located near Salt Lake City, Utah, and the girders were to be transported more than 500 miles to the site, up through the mountains. To accomplish this, a somewhat less direct route was taken to the project site to minimize superelevations, tight curves, and grades. For several of the tightest curves, oncoming traffic had to be stopped in advance to allow the truck to navigate the girder through the turn. For the delivery of the last girder, a self-propelled truck with a low-clearance cab supported the back end of the girder and helped to negotiate the curves and turns.

To set the girders, two cranes were used with one placed behind the east abutment and the other placed just off the northwest corner of the bridge, wedged up near the Shore Lodge resort. Each girder was delivered to the north side of the existing bridge, with one-way alternating traffic prior to the girder pick.

Substructure

Based on the minimum hydraulic opening, the new abutments were nearly in line with pier 1 and 4 of the existing bridge. In order to build the new abutments below the existing structure, we needed to increase the span length, which was unpalatable due to the increased costs for our design-build team. We therefore decided to construct the nearly 6-ft-tall, cast-in-place concrete abutment stems on temporary steel falsework to the north of the existing bridge. The top of the temporary falsework was at the high water level, and so a temporary cofferdam consisting of super sacks (gravel-filled bags) and pumps kept the construction zone dry when the lake levels were high during the summer.

The cast-in-place pile caps are located immediately outside of the existing bridge footprint so that the piles could be driven and the concrete for the pile cap was placed while traffic was

Cranes picking the first 156-ft 6-in.-long precast concrete beam. Photo: Ralph L. Wadsworth Construction.

IDAHO TRANSPORTATION DEPARTMENT, OWNER

BRIDGE DESCRIPTION: A single-span, 155-ft-long, precast, prestressed concrete beam bridge

STRUCTURAL COMPONENTS: Six UBT-90, bulb-tee girders, composite with an 8-in.-thick concrete deck; fully integral with stub abutments with post-tensioning to span between cast-in-place pile caps placed in the four quadrants

BRIDGE CONSTRUCTION COST: $3.2 million

AWARDS: ACI Intermountain Chapter "2015 Excellence in Concrete Award" and ACEC of Idaho "2014 Engineering Excellence Award"

Construction of west abutment stem wall on temporary falsework, adjacent to the Shore Lodge. Photo: Ralph L. Wadsworth Construction.

maintained on the existing alignment. The abutments do not have a flexural connection to the pile caps. This decision was made to allow the slide-in of the bridge to occur more smoothly and to simplify the connection.

As the stem wall is relatively short, flexural demand is easily accommodated by the 3 ft width of reinforced concrete. The bridge is restrained from nonthermal movement through the use of external shear keys, passive soil pressure, and friction between the pile caps and the abutment shoes. There are no elastomeric bearings below the abutments. In this manner, the bridge acts much like a three-sided culvert or stiff leg structure, albeit with a much longer span.

Each abutment stem wall is designed to span the 47 ft 6 in. distance between the pile-cap centers. Straight post-tensioning bars were used to supplement the nonprestressed reinforcement in the 5 ft 10 in. deep stem. The jacking force of the permanent post-tensioning was calibrated so that it supported 90% of the dead load of the structure, ensuring that the abutments wouldn't have an upwards camber that could have hindered the bridge slide. Four 1³/₈-in.-diameter, Grade 150, post-tensioning bars are placed in the abutment stem and provide 540 kips of force after losses.

On the bottom of the abutment stem, four 12-in.-high blockouts were set to allow vertical jacks to be placed. The blockouts also create five concrete "shoes" that were the primary points of contact for the bridge during the slide operation. The leading and trailing edges of the shoes are beveled to help ride up and onto the elastomeric slide pads. Stainless steel plates are cast into the bottom of the concrete shoes.

Once the existing bridge was demolished, a 2-ft-thick concrete "slide slab" was constructed to span between the pile caps and provide a surface upon which the bridge was to be slid. Concrete ledges were designed and detailed to extend from the pile cap and support the ends of the slide slab. However, the slab is primarily supported by soil.

A substantial concern during design was that as the structure was moved onto the slide slab, the supporting soils would settle and slow down the slide process. In order to mitigate this potential, the option of driving some piles to support the slide slab was discussed. However, this option was thought to be too costly and would require an additional mobilization of the pile-driving equipment. The existing wood-pile-supported pier cap from the old bridge was left in place.

However, the slide slab was not centered on the pile cap and so there was concern that the back side would differentially settle, twisting the slide slab and creating an uneven surface for the bridge slide. Instead, we found that the most effective solution was to over-excavate behind the existing pier caps and to place flowable fill, which improved the soil and load distribution. The slide slabs were then built on top of the flowable fill. Ultimately, there was no observed settlement and this method was successful.

Once the bridge was slid into its permanent location, wingwalls were cast in place. Dowels extended from the abutment stem and diaphragm to tie into the wingwalls. Two layers of tar paper were set below the wingwalls before casting to create a slip plane and accommodate thermal movement.

Project Aesthetics

Due to the project setting in a resort community, an aesthetics plan was required to be developed with the city of McCall. Given the tight time schedule for this design-build project, the progression of the design had to accommodate changes to some of the bridge elements and their geometry.

Last girder in place on abutment stem wall. Photo: Ralph L. Wadsworth Construction.

Fortunately, the elements that were most heavily influenced by aesthetics decisions were also those that were among the last to be constructed.

The parapet rail was mounted on a 2-ft-tall by 10-in.-wide concrete rail. These parapets meet TL-2 crash testing requirements and have been borrowed from a standard Virginia Department of Transportation detail. The parapets are broken up by 16- by 16-in. concrete pedestals that are spaced at roughly 22 ft centers along the edge of the deck. Down-facing lighting is provided in the pedestals for both safety and accent. Additionally, pole-mounted light fixtures will be selected and installed by the city on eight of the pedestals.

Tapered concrete monuments are provided at each abutment to visually anchor the bridge, with the top extending 6 ft 6 in. above the sidewalk. The city of McCall chose a board form finish for the abutment monuments and pedestals with a relief depth of 1 in. and a different finish for the parapet faces.

Brian Byrne is a senior structural engineer with Lochner in East Hartford, Conn.

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

EDITOR'S NOTE

This article has been taken in part from a paper written for the 2015 International Bridge Conference, IBC-15-82, which discusses the bridge slide aspects in more detail.

U.S. 90 Bridge over Biloxi Bay, Biloxi and Ocean Springs, MS
Engineer: Parsons Corporation
Photo: Courtesy of Parsons

PCI Precast/Prestressed Concrete Institute

Precast Provides the Resiliency Needed to Withstand a Cat 5 Hurricane

After Hurricane Katrina destroyed the U.S. 90 Bridge over Biloxi Bay, the Mississippi Department of Transportation knew the new 1.6 mile bridge would have to be able to withstand a Category 5 hurricane. It also had to include a 250-foot navigation span, require minimal maintenance and be constructed quickly. Precast concrete's inherent strength, durability, and speed made it the obvious choice.

Discover **High Performance** Precast

Versatile | Efficient | Resilient

For more information and free continuing education, visit

www.pci.org/hpprecast

I-10/SR 303L System Traffic Interchange

Arizona uses efficient concrete bridge geometry to ensure cost effectiveness for its largest system traffic interchange

by Andrew Baird, AZTEC Engineering

Anchoring the south end of Maricopa County, Arizona's, newest freeway is a multi-directional system traffic interchange linking State Route 303L (SR 303L) with Interstate 10 (I-10). The I-10/SR 303L System Traffic Interchange (System TI) is a five-level interchange with an embedded frontage road system. The five levels consist of SR 303L one level below grade, at-grade embedded frontage roads and local cross roads, I-10 mainline one level above, and two additional levels of system ramps resulting in structures as high as 90 ft above ground. The embedded frontage road system, a unique design in Arizona, not only enhances the operations of the traffic interchange, but increases the overall footprint to yield the largest System TI in Arizona.

Summary of Phase I Structures

At the project onset, the design team sought to use cast-in-place concrete box-girder bridges where feasible. Twelve of the 14 structures are cast-in-place, post-tensioned box-girder bridges of various widths and lengths as shown in Table 1. Four of the structures are freeway-to-freeway flyover directional ramps: east to-north (EN), south-to-east (SE), west-to-south (WS), and north-to-west (NW).

At more than 17 spans and 3412 ft, the longest structure is the Ramp EN bridge. It includes six columns that are 90 ft tall and have integral connections to the superstructure. The two bridges at I-10 over Sarival Avenue at the eastern end of the project are American Association of State Highway and Transportation Officials (AASHTO) Type V precast concrete girder bridges.

Collectively, all the major structures include more than 16,700 linear ft of drilled shafts ranging from 48 to 120 in. in diameter, more than 13.8 million lb of reinforcing steel and 71,200 yd^3 of structural concrete.
The post-tensioned structures used

Completed system traffic interchange. Note the frames for Phase II structures already completed over I-10. Photo: Visions in Photography.

profile

I-10/SR 303L SYSTEM TRAFFIC INTERCHANGE / GOODYEAR, ARIZONA

BRIDGE DESIGN ENGINEERS: Arizona Department of Transportation Bridge Group, Phoenix, Ariz.; AZTEC Engineering Group Inc., Phoenix, Ariz.; HDR Engineering Inc., Phoenix, Ariz.; and T.Y. Lin International, Tempe, Ariz.

PRIME CONTRACTOR: Pulice Construction Inc., Phoenix, Ariz.

PRECASTER: Royden Precast LLC, Phoenix, Ariz.—a PCI-certified producer.

POST-TENSIONING SUPPLIER: Dywidag Systems International USA, Long Beach, Calif.

Table 1. Summary of Phase I Structures–All bridges are post-tensioned, cast-in-place box girders except for Sarival Avenue Bridges, which are AASHTO girders with a composite deck.

Bridge	Spans	Length	Width	Depth
Sarival Ave (Two bridges)	3	245 ft 0 in.	253 ft 1 in.	6 ft 1 in.
Ramp EN	17	3412 ft 0 in.	43 ft 2 in.	9 ft 0 in.
Ramp SE	14	2527 ft 6 in.	43 ft 2 in.	8 ft 6 in.
I-10 EB OP	4	465 ft 6 in.	91 ft 2 in.	5 ft 6 in.
I-10 WB OP	4	465 ft 6 in.	91 ft 2 in.	5 ft 6 in.
I-10 WB Frontage Rd over Ramp WN	1	170 ft 0 in.	37 ft 2 in.	7 ft 9 in.
McDowell Rd. Over Ramp WN	1	105 ft 8 in.	144 ft 4 in.	5 ft 0 in.
I-10 WB Frontage Rd over Ramp SW	3	356 ft 4 in.	37 ft 2 in.	6 ft 8 in.
SR303L SB Frontage Rd Over Ramp SW	3	204 ft 11 in.	37 ft 2 in.	3 ft 9 in.
McDowell Rd TI UP	3	356 ft 8 in.	126 ft 4 in.	5 ft 6 in.
Thomas Rd TI UP	2	311 ft 0 in.	35 ft 0 in.	6 ft 9 in.
Ramp NW (Frames 3 and 4)*	4	873 ft 0 in.	43 ft 2 in.	9 ft 6 in. (max)
Ramp WS (Frames 3 and 4)*	5	971 ft 0 in.	43 ft 2 in.	9 ft 6 in. (max)

*Bridges added during Phase I Construction
Note: EN = east to-north, SE = south-to-east, WS = west-to-south, and NW = north-to-west.

Post-tensioning for Ramp SE. Photo: AZTEC Engineering.

Ramp SE and Ramp EN where the new I-10 was shifted to the south to allow for construction of ramps from the north. Note the soffit fill usage of the lower profile Ramp SE. Photo: Visions in Photography.

North half of flyovers completed. I-10 detoured to completed westbound roadway with the southern section of bridges approaching I-10. Photo: Visions in Photography.

0.6-in.-diameter, low-relaxation strands with tendons ranging from 20 to 25 strands. Jacking force per bridge ranged from 14,950 to 20,610 kips. All of the cast-in-place, post-tensioned, box-girder bridges used 4.5 ksi Class S concrete.

The cost per square foot for the AASHTO Type V bridge was $65. The total area of the cast-in-place, post-tensioned, box-girder bridges was 509,357 ft^2. The cost per square foot ranged from $72.67 to $131.06 with the average being $82.87.

With structures accounting for 30% of the overall construction cost and 72% of the overall construction schedule of the entire project, it was imperative for the structures to be designed as efficiently as possible. This included, but was not limited to, minimizing the height of the structures, reducing the number of spans, and utilizing on-site material for construction. Efficient bridge design is not just isolated to a structures team but rather the entire project team. This article summarizes how the design team worked together to create not only the largest System TI in Arizona, but the most cost-effective and efficient bridge geometry.

Optimize Geometry to Reduce Bridge Spans

As with all System TI, the first step in the design is geometry optimization to reduce bridge heights and lengths. This exercise was critical to ensuring the project was within the program budget as the embedded frontage road system meant that the traffic interchange was more expensive than originally planned. The result was an hourglass appearance where the northbound and southbound frontage roads were pulled closer to the SR 303L mainline, allowing the number of bridge spans to be reduced. Specifically the I-10 bridges over SR 303L—which at design start were each six-span, cast-in-place, post-tensioned, concrete box-girder bridges at a structure depth of 6 ft—were each reduced to four spans and a structure depth of 5 ft 6 in.

The horizontal and vertical alignments of the freeway-to-freeway ramps were also refined. The largest and tallest bridge on the project (Ramp

ARIZONA DEPARTMENT OF TRANSPORTATION, OWNER

BRIDGE DESCRIPTION: Twelve, post-tensioned, cast-in-place, box-girder bridges ranging in length from 105 to 3412 ft; two precast, prestressed concrete girder bridges with composite decks

STRUCTURAL COMPONENTS: 48- to 120-in.-diameter drilled shafts; 12 cast-in-place, post-tensioned box girder bridges; fifty-two 55-ft-long AASHTO Type V girders; forty 129-ft-long AASHTO Type V girders; and cast-in-place concrete abutments, pier caps, blade-shaped columns, and decks

BRIDGE CONSTRUCTION COST: $52 million

AWARDS: ACEC Arizona— 2015 Engineering Excellence Grand Award

EN) was reduced by over 400 ft from a 3866-ft-long, 20-span bridge to a 3412-ft-long, 17-span bridge.

Sequence Construction to Reduce Falsework

A mandate of this project was to maintain three lanes of traffic in each direction of I-10. As part of the geometry refinement, the design team shifted the I-10 alignment north approximately 200 ft. This enabled the construction of the entire westbound I-10 to be built without impacting existing I-10 traffic.

Once I-10 westbound was complete, six lanes of traffic (three in each direction) were shifted to the completed freeway section allowing the construction of eastbound I-10. Overhead flyovers (Ramp SE and Ramp EN) were sequenced to follow the shift of I-10, eliminating the need for falsework above live traffic. Both Ramp SE and Ramp EN were constructed from north to south, with the hinge ending just north of existing I-10. Once the concrete for the decks was placed and falsework removed, the traffic shift allowed the remainder of the two main flyovers to be constructed from south to north, meeting at the hinge point constructed in the previous stage.

To utilize the established construction-sequencing plan, the project added elements of a future phase to the Phase I construction. Frames of west-to-south (WS) and north-to-west (NW) were to be constructed while I-10 was detoured, ensuring there would be no more impacts to I-10 during future phases of the System TI. The project team collaborated with the Arizona Department of Transportation (ADOT) to choose the best approach to facilitate this addition. Ultimately, the project team implemented a design-build approach of planning the phasing and design of these bridge ramps in an expedited manner. Both the engineer and contractor worked side by side to meet the expedited schedule. The design and construction of these added features, which included high falsework bridge construction, was accomplished with a minimal 5-month impact to the critical path of the complex project.

Simplification of construction staging was another goal at the onset of the project. In order to accomplish this task and reduce the construction stages to four, both ADOT and the design team worked with the city of Goodyear to develop a crossroad closure plan and detour route to minimize the amount of travel through a work zone. Half-diamond traffic interchanges at Citrus Road and Sarival Avenue were constructed in the early stages of the project, creating a detour route for both freeway and local traffic, allowing for the full closure of Cotton Lane. This cleared an entire work zone for the ADOT contractor. Sarival Avenue is the only bridge on the project to use AASHTO Type V precast concrete girders to eliminate the need for falsework and expedite the bridge construction to meet the required schedule.

Utilize Material for Soffit Fill

The combination of basin, roadway, and structural excavation and the depressed SR 303L construction yielded a massive earthwork project with over 3.4 million yd^3 of excavation. Structural excavation for Ramp SE and Ramp WN resulted in over 1 million yd^3 of excavation alone. One of the primary uses of this material was as soffit fill for the structures up to 35 ft. Construction sequencing, detours and closures allowed the contractor to maximize the use of soffit fill, reducing bridge heights and falsework.

Aesthetics

As the majority of the construction was structures, the overall aesthetics of the system relied heavily on the structural elements. The three predominant features that contribute to the aesthetic appeal of a bridge are the superstructure type, span configuration, and architectural treatment.

Post-tensioned, box-girder bridges offer uniform simple lines, a relatively shallow structure depth and integral pier caps, which make for an aesthetically pleasing structure.

Type

Post-tensioned, box-girder bridges offer uniform simple lines, a relatively shallow structure depth and integral pier caps, which make for an aesthetically pleasing structure.

Use of excess material as soffit fill for one of the lower profile bridges. Photo: AZTEC Engineering.

Excavation for foundations of Ramp SE and Ramp EN adjacent to I-10. Photo: Visions in Photography.

Span Configuration

Maximizing the span length creates an appearance of openness and freedom for the traveling public.

Architectural Treatments

Bladed columns that are not highly visible, or with heights less than 50 ft, have a vertical mountain rustication treatment centered in the wide faces of the column. Bladed columns that are highly visible and have heights above 50 ft have the lizard icon treatment with an accent color. Piers, wingwalls, retaining walls, abutment faces, and outer faces of the barriers were designed to accommodate integral artwork designs and rustication patterns so that aesthetic compatibility can be maintained with the Phase I portion of the I-10/SR 303L System TI. In addition, all exposed bridge surfaces, with the exception of the roadway surface and inside face of barriers, were painted.

A complex project in both size and scope, the completion of this project serves as an example of how to successfully deliver a project that is both cost effective and aesthetically pleasing.

Andrew Baird is a project manager with AZTEC Engineering in Phoenix, Ariz.

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

PEARL HARBOR MEMORIAL BRIDGE

Stay-cable installation for the new bridge

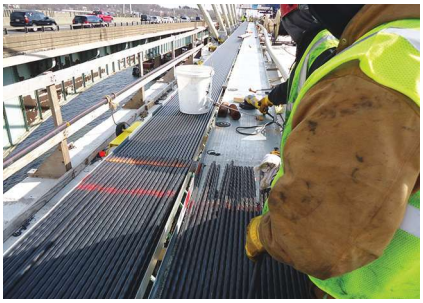
by Wade S. Bonzon, Figg Bridge Engineers Inc.

The new $635 million, 10-lane Pearl Harbor Memorial Bridge is the engineering and aesthetic highlight of the $2.0 billion, Interstate 95 (I-95) New Haven Harbor Crossing (NHHC) Corridor Improvement Program, now nearing completion in New Haven, Conn. The concrete segmental main-span unit features a 515-ft-long span over the Quinnipiac River with symmetrical back spans of 249 ft. The structure consists of twin box-girder superstructures, each supported by two planes of stay cables. Sixty-four stay cables support each girder, with the middle towers sharing cable anchorages for both box girders.

To avoid Federal Aviation Administration flight path restrictions, this concrete segmental extradosed cable-stayed bridge features shorter towers than normally found on a conventional cable-stayed bridge, with correspondingly flatter stay-cable angles. In addition, the extradosed structure type allowed for a shallower section depth than a conventional box-girder superstructure, providing necessary vertical clearances for shipping traffic.

The shallow-angled stay cables apply greater load along the axis of the bridge compared to a conventional cable-stayed bridge. The structure also features a stiffer deck system than most typical cable-stayed bridges, making fatigue stresses due to live-load effects on the cables less of a concern. While the stresses in conventional stay cables are limited to 45% of the strands' minimum ultimate tensile strength (MUTS) for American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications* Service Load Combination I, the contract documents for this project specified that the extradosed cables could be stressed up to 55% of MUTS for the same load combination. Similar increases in stresses are allowed for other AASHTO load combinations.

Stripping of the stay-cable strand ends in the layout tray. Photo: FIGG Engineering Inc.



Aerial photo of the cable-stayed main span. Photo: Walsh-PCL JV II and Shoreline Aerial Photography.

All of the stay cables are comprised of 48 individual 0.6-in.-diameter, 7-wire, low-relaxation strands, which are each greased and encapsulated in a tight fitting HDPE coating. A 9-in.-diameter, high-density polyethylene (HDPE) sheathing pipe provides the outermost layer of protection to the strands.

All stay-cable strands were stressed individually with a monostrand jack. To ensure that all strands in a stay were stressed to the same force, the elongation method was used to control strand-by-strand stressing forces. In theory, if all strands are elongated the same amount they will all carry the same force. In practice, this required careful quality control during strand preparation, installation, and stressing.

All strands needed to have their protective sheathing removed in the anchorage regions to be properly gripped by the three-part anchor wedges. The stripped length was carefully controlled so that exposed strand portions remained fully within the protective "wax box" of the anchorage, which was filled with protective grease after completion of stay stressing.

After HDPE coating removal, each strand was scored with a small v-notch reference mark in the unstressed length near each end. These reference marks were used to match elongations during stressing, so the distance between marks for each strand of the stay cable needed to be consistent to within less than +/-⅛ mm. Depending on the stay location, distances betweenreference marks varied from 114 to 221 ft.

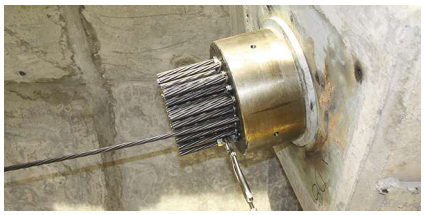
To achieve the required precision and consistency of the reference mark placements, sheet metal layout trays were set up on the bridge deck. Each tray was fabricated so that two rows of 24 strands each could be laid onto the tray snugly together, thus keeping all of the strands straight and parallel with each other in the tray. Reference marks were aligned across each row of strands using a heavy steel angle as a straightedge. Independent checks of the cut strand lengths, sheathing removal lengths, and reference mark locations were performed by the contractor and the inspector.

The sheathed sections of the strands were too wide to slide through the holes in the anchorages, so the stripped strand ends were inserted into each anchorage from the rear. This required installing strands through temporary openings in the stay pipes at deck level and at the tower entrances. Each strand was first inserted through the fixed-end anchorage at deck level and secured with a special clamping tool to keep the anchor wedge of each strand a consistent distance from the reference mark. Strands were then pushed up the length of the stay pipe to the upper access window where the strand was inserted through the matching hole in the tower anchorage and stressed using the monostrand jack. The process was repeated for each subsequent strand in a preplanned order for all 48 strands.

To maintain matching forces in each strand, the first strand was stressed to a prescribed force. When the second and all subsequent strands were stressed, each new strand was tensioned until the reference mark on the current strand physically lined up with the reference mark on the first strand. In this way, all the strand reference marks matched at the fixed-end (using the clamping tool) and at the adjustable end (by elongating until the marks aligned). This ensured that each strand was elongated the same amount.

Strand stressing was typically performed in three steps. First, all 48 strands were installed and stressed individually to a force equivalent to 15% MUTS. This allowed the internal damper assemblies to be slid down the galvanized steel guide pipes and bolted in their final position. The second stage of tensioning was performed to approximately 50% of the final stay-cable force to monitor cantilever deflections and strand elongations for conformance with the theoretical predictions. The final stage of strand tensioning was performed to fine-tune the strand forces to closely match the target stay force value at that stage of cantilever erection. Individual strand lift-offs confirmed that all strand forces matched within a +/-2% tolerance range and that the total stay force matched the predicted force at that construction stage.

Stressing a single strand using a monostrand tensioning jack. Photo: FIGG Engineering Inc.



Fixed-end anchorage with wedge alignment clamp. Photo: FIGG Engineering Inc.

At the completion of the main span, a final round of stay-cable lift-offs were performed to confirm that all stay forces matched the theoretical forces within +/-5%, a tolerance defined in the contract specifications. The stay-stressing procedures and quality-control checks were effective: only 1 of the 128 stay cables on the project needed a force adjustment at the end of construction. This saved time on the schedule as it avoided time-consuming stay force adjustments on the project schedule's critical path.

The stay-cable installation and stressing procedures used for the new Pearl Harbor Bridge replacement project proved to be simple and robust, resulting in stays installed in conformance with the project requirements and with good construction practices. The high quality of the stay installation operation will ensure good long-term performance of a critical part of the bridge's structural system for decades to come.

Wade S. Bonzon is a regional director in the Texas office of FIGG Bridge Engineers Inc. in Dallas.

EDITOR'S NOTE

Current practice for extradosed bridge design now limits the stay stresses at service load conditions to 60% of MUTS.

Accelerated Curing of Precast Concrete Products

by Dr. Henry G. Russell, Henry G. Russell Inc.

Steam curing of precast, prestressed concrete beams. Photo: Henry G. Russell Inc.

Precast concrete products are typically produced on a 24-hour cycle to achieve the economy of production. For precast, prestressed products, this typically requires the use of accelerated curing to achieve the concrete compressive strength required at transfer of the prestressing force. Transfer is generally accomplished no later than 18 hours after the concrete is placed and may be achieved at 12 hours or less. In some situations, such as the use of high-strength concrete, the accelerated curing can be achieved by the internal heat of hydration. In most situations, however, it is necessary to utilize an external source of heat, such as steam or radiant heat, to achieve the necessary concrete strengths.

Curing Cycle

A curing cycle generally consists of four stages or time periods:

- Stage 1 – Initial set
- Stage 2 – Temperature gain
- Stage 3 – Constant temperature
- Stage 4 – Cool down

Application of heat should not begin until the concrete has achieved its initial set except to maintain a minimum temperature of 50°F inside the enclosure. The application of heat too early can be detrimental to subsequent strength gain. After initial set, the temperature rise must be controlled to prevent damage to the concrete. The American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Construction Specifications* limits the average temperature rise of the concrete to a maximum of 40°F per hour with a maximum concrete temperature of 160°F. A constant temperature, which can be less than 160°F, is then held until the desired concrete compressive strength, as measured on concrete cylinders cured with the beams or matched cured, is achieved. In the cool down period, the *AASHTO LRFD Bridge Construction Specifications* require that the cooling rate of the concrete not exceed 40°F per hour until a temperature of 20°F above the air temperature is reached. For precast, prestressed concrete, transfer of the prestressing force to the concrete should be accomplished immediately after the heat curing is discontinued. Otherwise, vertical cracks in the girders may result from thermal contraction.

For precast, prestressed products, achievement of the required concrete compressive strength for transfer means that the strands can be released and the product can be removed safely from the casting bed to the storage area. For this stage, handling stresses need to be carefully considered as described in the Precast/Prestressed Concrete Institute (PCI) *Bridge Design Manual*.[1]

The AASHTO *Standard Specifications for Highway Bridges* used to require steam- and heat-cured products that would be exposed to salt water to be kept wet for at least 7 days. Research has shown that moist curing of precast concrete products after a proper heat curing period does not improve the durability properties. Studies by Klieger[2] showed that an additional moist curing period following the accelerated curing cycle is not

necessary and may be detrimental to the freezing and thawing durability of the concrete. The AASHTO requirement for 7 days wet curing was removed in 1992.

Benefits of Accelerated Curing

In addition to achieving the required concrete compressive strength at transfer more quickly, accelerated curing reduces the volume of permeable voids,[3] which improves freezing and thawing durability, reduces chloride permeability, and decreases absorption. Accelerated curing also decreases creep and shrinkage and, therefore, prestress losses. To quote a PCI Special Report:[4]

> "Historically, properly heat-cured concretes produced at low water-cement ratios have been found to have strength and frost resistance properties equal to or better than conventionally cured concretes."

Precast, Nonprestressed Products

The required concrete compressive strengths for precast, nonprestressed concrete products are generally lower than those required for precast, prestressed concrete products. Consequently, accelerated curing may or may not be used depending on the required strength and individual plant practices. When accelerated curing is used, it has the advantages mentioned previously and additional curing is not needed. When accelerated curing is not used, the durability properties will not be as good even though the same compressive strength may have been achieved. Consequently, additional wet curing is advantageous if durability is a concern. A longer curing period will produce a more durable concrete although there is a point of diminishing returns. This is generally taken as 5 to 7 days and is not considered as excessive. The use of curing compounds is not as beneficial as wet curing and can affect subsequent coatings and aesthetic appearance.

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Dr. Henry G. Russell is an engineering consultant who has been involved with the applications of concrete in bridges for over 35 years.

Design Considerations for Horizontally Curved Precast Concrete U-Girders

by Thomas W. Stelmack, Parsons

Plant produced horizontally curved, precast concrete U-girders began in Colorado about 10 years ago with the Ramp K flyover ramp linking Interstate 25 (I-25) and State Highway 270 (SH 270) in Denver. This generated interest in the precast industry and led to the development of the PCI Zone 6 concept plans in 2012. A full discussion of the history of the curved U-girder development may be found in the Summer 2015 issue of *ASPIRE*.™ Several more similar structures were constructed in Colorado, all being contractor-developed alternate designs or value-engineering change proposals. The SR 417/Boggy Creek Road interchange in Orlando was the first standard design-bid-build project in the United States to use the curved precast concrete U-girder concept (see article beginning on page 14). This milestone project illustrates that the concept is viable for typical project delivery, in addition to contractor alternate designs and design-build.

Figure 1. Detail for the expansion end of girder with tongue extension for bearing support. All Photos and Figures: Parsons.

Applications

While most curved bridge structures can be designed and constructed with the precast concrete curved U-girders, the most common application has been interchange flyover ramps where aesthetics are important, the curve makes chorded girders impractical, and highways and ramps below make precast concrete construction advantageous. The girders are also well suited to rail structures with a girder line below each track.

Special Design Considerations

One of the most critical and limiting aspects of this girder type is the girder weight. When possible, additional dead loads such as diaphragms, lid slabs, and bottom slab thickening, should be designed such that they can be cast after the girders are erected at the site. This requires analyzing the casting sequence, checking stresses, and providing the necessary details in the plans.

Figure 2. Detail for an interior end of girder adjacent to closure joint.

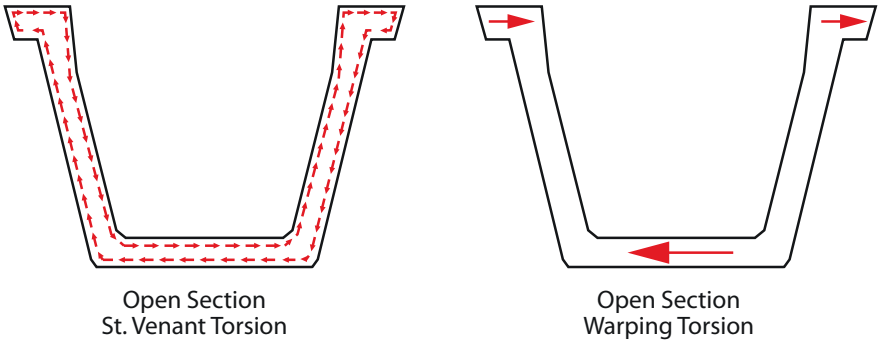

Figure 3. St. Venant and warping torsion shear flow.

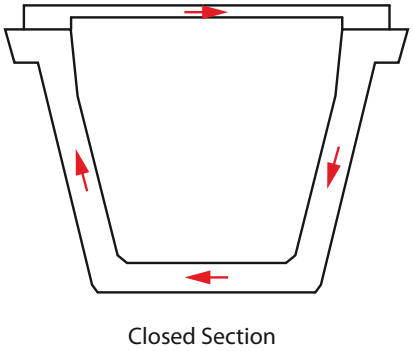

Figure 4. Torsional shear flow with lid slab cast on U-girder to form a closed section.

One detail that has been developed to reduce girder weight is the use of a lower "tongue" at the expansion joint ends of girders (Fig. 1). This detail provides sufficient concrete to support the girder on the permanent bearing, while allowing the contractor to cast in place the remainder of the diaphragm, both within and between the girders. Not only does this save weight, but it also allows the reinforcing steel and post-tensioning hardware for the continuity tendons to be placed efficiently by crews that are typically more experienced with this type of construction detail.

Geometric limitations for U-girders are typically driven by the transportation and lifting restrictions. Assuming that the girders are transported over state-owned highways, the girder weight and dimensions will control the girder length and radius. Girders weighing up to about 300 kips with a length of about 100 ft have been successfully used, but the designer should always verify whether loads like these are acceptable for a project. The minimum radius depends on the girder length and the total allowable width for transportation, but radii as low as 800 ft have not been a problem. By using the spliced and post-tensioned method of construction, span lengths up to about 230 ft are currently obtainable for 84-in.-deep, constant-depth standard sections.

Another important design consideration is the girder to pier cap connection. Options include using bearings, or making the girders integral with the caps. Integral caps can be more economical, as they allow the cap and diaphragm to be one rather than separate and it saves the cost of bearings. It also eliminates the need for bearing inspection and maintenance by the owner. However, the columns need to be of sufficient height and flexibility in order to not penalize the design of the substructure from superstructure longitudinal rotation and translation.

The most unique and probably least-understood aspect of designing a curved precast concrete U-girder structure is the fact that the girders are curved. This requires special care and understanding of the behavior during each phase of the construction process, as the behavior and critical aspects change as the elements and structural system change. For the individual girder segments, generally the contractor is responsible for lifting, support locations, and stability. The designer should ensure that the girder ends are stiffened or braced so that no distortion occurs at the ends. This has been done with both steel bracing and integral concrete stiffening ribs at webs and bottom slab and a strut between the top of webs. The ribs also allow for a thicker closure joint and additional shear key area (Fig. 2).

American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications* Article 4.6.1.2.4c addresses horizontal curvature for closed box and U-girders. It states that curvature may be neglected in the analysis for main axis bending and shear if girders are concentric, bearings are not skewed, the arc span divided by the girder radius is less than 0.3 radians, and the girder depth is less than the box width at mid-depth. The author has found that the difference between the outer and inner girder live load shear can be 45% and flexural moments 25% for an arc span to radius ratio of even 0.17 radians. Therefore, for multiple box-girder sections, it is recommended that curvature be considered when the arc span to radius ratio is greater than about 0.10 radians.

The other design aspect of horizontal curvature to be considered is torsion. A full discussion of torsional analysis and design is beyond the scope of this article. However, it is important to understand that there are two fundamentally different cases to be considered for torsion: for the open U-girder section and for the closed box-girder section once the decks slab has developed sufficient strength.

Open sections, which are flexible and weak in torsion, resist applied torsion by both St. Venant and warping torsion (Fig. 3). Warping torsion resistance becomes insignificant on longer spans and cracking also significantly reduces the torsional stiffness. It has been found that typically the individual girder segments can resist their own weight without additional bracing of the webs along their length, although it should always be checked. However, they are not capable of resisting the torsion due to their self-weight and that of the wet deck slab concrete once they are connected and span from pier to pier. By using a thin lid slab to close the open sections (Fig. 4), the torsional stiffness of the section is increased by about 100 times. Thus, the sequence requires the lid slabs to be cast after the closure placements have been made and the first level of continuity post-tensioning has been stressed. Next, the remaining continuity tendons are stressed, the shoring tower supports removed, followed by the casting of the full deck slab.

Besides the reinforcing steel design for the added web shear due to torsion and live-load distribution between girders, the design for the

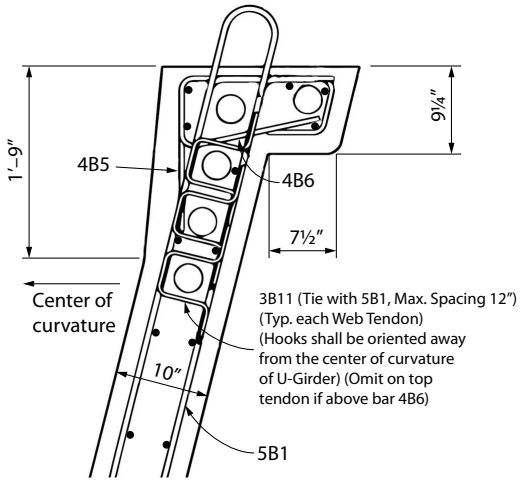

Figure 5. A typical reinforcing detail for restraining curved tendon ducts in the web.

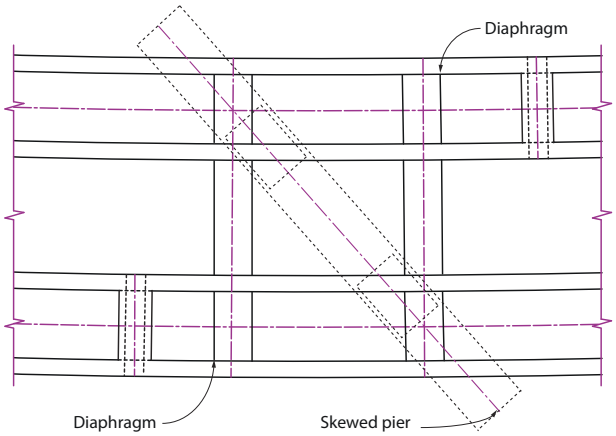

Figure 6. Diaphragm layout in plan for a skewed pier.

radial forces due to internal web post-tensioning along the curved alignment is also critical. This is addressed in AASHTO LRFD Specifications Article 5.10.4.3. The design radial force is calculated by dividing the ultimate tendon force by the curvature radius. The reinforcing steel should be sized and proportioned to limit the steel stress to $0.6f_y$. The reinforcement is typically made up of hairpin-shaped bars that wrap around the duct and adequately developed with hooks on the open end of the bar. Figure 5 shows a typical detail for the bars.

In addition to the horizontal curvature effects, additional torsion can be created when a pier support is skewed. In addition to designing for the added web shear, it is important to minimize the differential deflections that can occur in the box girders due to the non-radial orientation of the bearings from one girder to the next. It is recommended that two sets of radial diaphragms be used to connect the two box girders, one at each bearing location (Fig. 6). This also simplifies the constructibilty of the diaphragms by eliminating the skewed orientation of a single diaphragm, with respect to the precast concrete girders.

Tremendous progress has been made in the past 10 years regarding the development of plant produced horizontally curved precast concrete U-girders. By using standard sections that fabricators can purchase forms for, they are becoming a viable alternative to cast-in-place concrete and steel box girders for standard delivery design-bid-build projects. For this development to continue, it will be important for engineers in the industry to become familiar and proficient with the design and construction details of the girders.

Thomas W. Stelmack is west sector technical director with Parsons in Denver, Colo.

EDITOR'S NOTE

For more information on the history of the curved U-girders, see "Sharing New Technology through PCI Bridge Technoquests" in the Summer 2015 issue of ASPIRE *or the* ASPIRE *website for a longer version of the article.*

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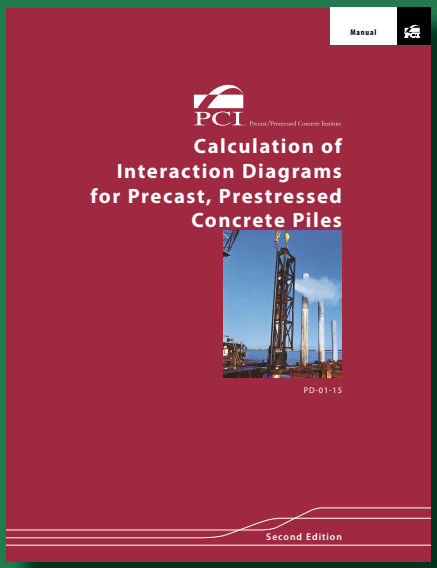

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CONCRETE BRIDGE PRESERVATION

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US 131 over Muskegon River

Harped external post-tensioning used to strengthen, rehabilitate bridge

by Corey E. Rogers, Michigan Department of Transportation

US 131 spans the Muskegon River on a northbound and southbound pair of 580-ft-long, concrete box-girder structures resting 55 ft above the river. The three-span bridges were constructed in 1982 and supported by a pair of fixed piers. Routine inspections by the Michigan Department of Transportation (MDOT) identified web shear cracking at pier locations and cracking in the bottom slab at post-tensioning (PT) anchorage locations.

Crack injection and structural monitoring were utilized until a 2013 structure study was completed. The shear reinforcement in the webs and diaphragms was found to be inadequate for the indirect support situation caused by the bearings not providing direct support to the webs of the box girders. Also, it was determined that the reinforcement around the anchorages was insufficient for the demands of the PT. A combination of harped external PT through the box girder, along with pier modifications to redistribute the shear forces, was recommended to strengthen the structure. MDOT proceeded immediately with a $5 million rehabilitation project.

The design incorporated pier corbel construction to support grout-filled flat jacks, diaphragm modifications, deviator diaphragms, anchorage blocks, and external PT. The original steel corbel design was value-engineered to a cast-in-place concrete option utilizing high-performance, 6.5 ksi concrete and transverse PT rods. The flat jacks were designed to accommodate a 1-in. stroke at 700 kips lock-off, and an ultimate load of 2100 kips.

Northbound US 131 over Muskegon River bridge after project completion. Photo: Michigan Department of Transportation.

The external PT comprised four full-length tendons and four additional tendons in the center span. The tendons were twelve 0.6-in.-diameter, 270 ksi, low-relaxation strands that were stressed to 560 kips. The diaphragm modifications, deviator diaphragms and anchorage blocks were designed for 6 ksi concrete and were attached to the structure with 1.25-in.-diameter PT rods.

In order to accommodate the 1.25-in.-diameter PT rods for the box girder modifications and tendon anchorages, approximately 400 holes were cored in the structures. To avoid damaging existing structural tendons and critical reinforcement during the coring process, ground penetrating radar, destructive testing (hand chipping and masonry drilling), and review of as-built plans were performed. Minor conflicts with existing tendons did occur initially, prompting process improvements such as early identification of reinforcement locations. Subsequent coring operations were successful. Much of the coring success is attributed to communication between MDOT inspectors, contractors, and designers.

Coordination with the city of Big Rapids, Mich., Ferris State University (Ferris), local businesses, and Michigan's notorious "up north" traffic was essential. US 131 is a major route to popular weekend destinations. Lane closures were permitted; however complete detours of 2- to 3-day durations were only allowed during Ferris's summer break. Diaphragm modifications, flat-jack inflation, and PT grouting were completed under structure closures to avoid locking in live-load stresses and effects from vibrations during grouting.

The project was completed in September 2015, ahead of schedule and under budget. The success of this project extends the service life of the structures and eliminates the need for complete structure replacements.

Corey E. Rogers is an engineer of Bridge Field Services in the Michigan Department of Transportation in Lansing, Mich.

Impregnation of Post-Tensioning Tendons

A solution for post-tensioning steel corrosion

by David W. Whitmore and Martin R. Beaudette, Vector Corrosion Technologies Ltd.

Post-tensioned concrete is a well-accepted construction technique that offers appreciable benefits in bridge design, construction, and appearance. Many signature structures across the country have been cost-effectively built with post-tensioned concrete. Post-tensioned bridge tendons are normally bonded with cementitious grout after tendon stressing. In addition to bonding the strands in place, the cementitious grout provides an important function in the durability of the structure by protecting the post-tensioning tendons from corrosion.

Grout quality for post-tensioning tendons has been an important consideration for many years because strand corrosion was noted in the presence of grout bleed water and bleed water voids. This concern prompted the development of proprietary cementitious grouts with anti-bleed characteristics.[1] Modern grouts have provided improved performance, but problems have been detected on recently constructed structures including grout voids, segregated grout, soft grout, presence of a high level of sulfates, and chloride-contaminated grout.[2]

In addition to the noted grout defects, an elevated level of relative humidity (RH) has been suggested as an environmental-condition factor that affects the risk of corrosion for post-tensioning systems, even without the presence of chlorides.[3] High RH influences the effectiveness of cementitious grout, particularly in the presence of voids, by reducing its electrical resistivity, an important material property for corrosion resistance. For example, RH levels in grouted post-tensioning tendons in Virginia have been measured as high as 94.4% as compared to ambient RH of 63.1%.[4]

According to the Federal Highway Administration, some post-tensioned concrete structures with multiple corrosion protection measures have experienced tendon failure within 6 to 17 years of service.[5] Due to the cost of inspection and the consequence of post-tensioning tendon failure, the Texas Transportation Institute states that "...bridge owners should do everything economically feasible to prevent the exposure of strands to high relative humidity levels, water, and/or chloride conditions."[3]

Example of a post-tensioned, box-girder bridge with external post-tensioning tendons. All Photos: Vector Corrosion Technologies Ltd.

Salt spray testing on untreated and impregnation-treated strands.

Segregation of grout (light tan is soft and grey is solid) and void at top of tendon.

Impregnation of post-tensioning tendons, a recent advancement in corrosion mitigation of grouted post-tensioning steel, offers a promising solution to protect both new and existing post-tensioning tendons from the effects of high RH levels, voids, water, sulfates, and chloride-induced corrosion. The system utilizes the naturally occurring interstitial spaces between the wires of seven-wire strands to transport a formulated low-viscosity, dual-acting, hydrocarbon-silicon-polymer resin that displaces moisture, forms a protective barrier on exposed steel surfaces, and impregnates the surrounding grout to form a barrier to moisture and oxygen.[6]

Access to the interior of the tendon is made at the grout caps or by an installed port at an intermediate tendon location. After the tendon is air tested and any leaks are

Corroded strands in a bonded post-tensioning tendon.

Impregnation of post-tensioning tendon from end anchorage.

Grouted post-tensioning strand sample undergoing potentiostatic testing.

Removal of grout reveals impregnation material present on steel surface and penetration into surrounding grout.

repaired, the impregnation process begins. The impregnation material has been shown to travel as far as 250 ft along the length of a tendon through the interstitial spaces.

Impregnation of post-tensioning steel has been utilized on post-tensioned bridges in Florida and Virginia. These projects have successfully demonstrated that the impregnation process allows the impregnation material to

- flow through the interstitial spaces in post-tensioned tendons up to 250 ft from the end or up to 100 ft from an intermediate location,
- leave a corrosion resistant film on exposed steel surfaces, and
- penetrate into the grout for an additional layer of protection surrounding the strands.

The post-tensioning-steel impregnation system has been subjected to a great amount of corrosion testing. Laboratory testing completed to date has focused on determining the qualitative and quantitative corrosion-resistance benefits of the impregnation process. For example, salt spray testing on treated and untreated post-tensioning strand dramatically reveals the substantial corrosion resistance provided by the impregnation material.

Potentiostatic testing is a method to produce direct quantitative comparisons between treated and untreated samples in a laboratory setting. The potentiostatic testing completed on impregnated samples produced positive results with a 94.7% reduction in corrosion current in samples with 4.5% grout voids and a 93.1% reduction in corrosion current in chloride-contaminated grout samples. Interestingly, potentiostatic testing on void-free grout samples with and without chlorides shows that the impregnation treated samples had an 81.4% reduction in corrosion current versus the untreated samples.[6]

This testing demonstrates that the impregnation treatment improves the corrosion resistance of post-tensioning tendons with grout voids and chloride contamination. In addition, the treatment improved the corrosion resistance of properly grouted post-tensioning strands, a result that demonstrates the benefit of the impregnation process as a proactive corrosion protection treatment for newly constructed post-tensioned bridges.

Test Objective	Sample Description	Treated vs Untreated Samples Reduction in Corrosion Current
Grout Void Test	Chloride-free Grout with 4.5% Grout Void	94.7%
Chloride-contaminated Grout Test	Chloride-contaminated (2%) and Void-Free Grout	93.1%
Control/ Properly Grouted Strand	Chloride-free and Void-free Grout	81.4%

Potentiostatic test matrix and results.

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EMBRACING CHANGE

Innovate or live in the past?

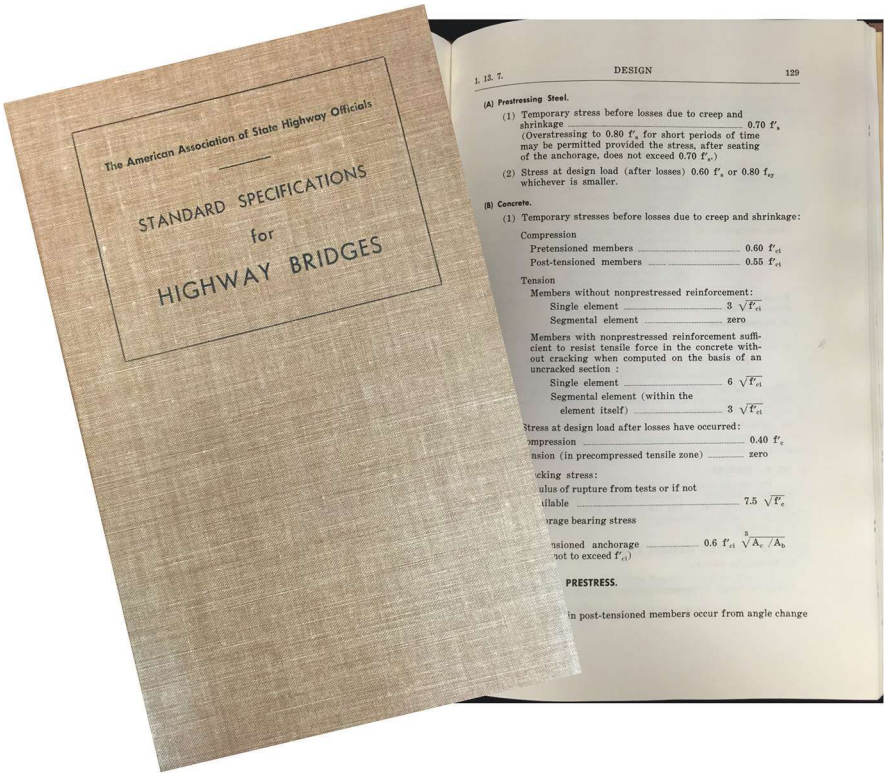
by Dr. Oguzhan Bayrak, University of Texas at Austin

As human beings, certainly as structural engineers, most of us are change averse. We may have all said "if it ain't broke, don't fix it" or "this appears to be a solution in search of a problem" at some point in time in our structural engineering careers. Even if we may not have said it, we certainly heard others say it in our workplace, committee meetings, conferences, construction sites, and fabrication plants.

Yet, there have been significant innovations in designing and constructing concrete bridges over the years. The decision of what change to embrace is a complex process, and one that can be emotional rather than rational. In this article, I will share some of my thoughts on embracing change.

Like most things in life, embracing change becomes easier if it can be put into its full context. That is to say, it is always important to understand the technical, financial, fabrication, and construction benefits of a solution as we work on a problem. Often, the involvement of the appropriate representatives from industry, design, construction, and fabrication professionals may help define the problem, and the potential benefits of any generated solutions.

To illustrate this process, I will use two research projects[1-4] in which the University of Texas researchers studied the feasibility of increasing the allowable compressive stresses at prestress transfer. This value has remained unchanged since prestressed concrete was introduced in the American Association of State Highway Officials (AASHO) *Standard Specifications for Highway Bridges* in 1961.



American Association of State Highway Officials *Standard Specification for Highway Bridges* (1961). Photo: Ozzie Bayrak.

Before getting into technical details, it is important to note that the research projects that are used as an example benefited from the participation of engineers from the Texas Department of Transportation's (TxDOT's) bridge and construction divisions, as well as the precast concrete fabrication plants in Texas. This participation helped provide context to the problem. Let's take a look at that context.

From the beam fabrication standpoint, an increase in allowable compressive stresses from $0.60f_{ci}'$ can reduce the cementitious material content in the concrete, the cycle time in precast concrete fabrication plants, and external curing costs. From the structural design perspective, an increase in this limit can increase the span capability of a given prestressed concrete section by allowing the use of a larger number of strands. Finally, an increase in this compressive stress limit may result in a reduction of the number of debonded or harped strands, and therefore may simplify design and fabrication of a prestressed concrete beam.

The interplay between the fabrication and design benefits can be complex and it is not possible to invoke all of the aforementioned benefits simultaneously. Having understood all of the aforementioned benefits, the University of Texas researchers considered the downside of increasing the allowable compressive stress at prestress transfer.

Service load testing of a beam in the lab. Photo: David B. Birrcher.

More specifically, the researchers focused on the behavior of the bottom flanges of the pretensioned girders. If the precompressed tensile regions of girders are subjected to excessively high stresses, there can be internal damage to the microstructure of concrete, and premature cracking under service loads may occur. This cracking may create a durability concern that would not otherwise be present.

Weighing the fabrication and design benefits against the potential of creating a durability problem, and primarily through full-scale testing, the University of Texas researchers concluded that an increase from $0.60f_{ci}'$ to $0.65f_{ci}'$ was possible. During the American Association of State Highway and Transportation Officials (AASHTO) Subcommittee on Bridges and Structures (SCOBS) meeting in April 2015, AASHTO Committee T-10 presented this change as an agenda item and their recommendation for this change was approved. The 2016 Interim Revisions of the 7th edition of *AASHTO LRFD Bridge Design Specifications* will include this change.

This example involves a methodical research approach in which participation of the precast concrete industry, structural designers, and inspectors was solicited and understood by the research team. Consequently, it was possible to establish proper context to the problem. This context helped the researchers establish priorities such that relevant answers were obtained.

With all of that said, the change from $0.60f_{ci}'$ to $0.65f_{ci}'$ seems small. In my view, that is the reality of structural engineering in the twenty-first century. Small refinements with big benefits or small adjustments with big pay-offs are our new reality. There will always be a few game-changing advancements to our knowledge base and design practices. However, a great majority of the advancements to our knowledge will be "incremental" at first glance but "profound" in consequences.

In regards to embracing change, I must say that, at least in Texas, the change that stemmed from the previous example was embraced readily. We found that the inclusion of all stakeholders in defining the problem was very helpful. Some of the stakeholders were hoping to see no change, others were hoping for a bigger change. All of them accepted the research results and what was possible, regardless of their original hopes and expectations.

Finally, we must all understand that change will happen one way or another. Understanding the history of a technical issue provides additional context to the problem we are trying to solve. Understanding that history helps us embrace change and it should never be taken as living in the past. Certainly, living in the past and resting on our laurels would stifle innovation and progress and should be avoided at any cost.

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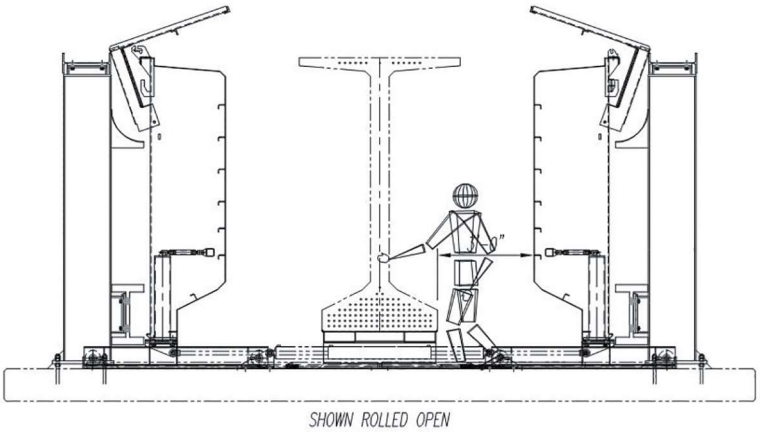
Steel Forming of Precast, Prestressed Concrete I-Beams

by Dr. Maher K. Tadros, e.Construct.USA LLC

Precast, prestressed concrete I-beams are the most common girder shape used for bridges in the United States. They can span between 30 and 350 ft, the widest range of spans of any precast concrete girder stringer bridge product. The American Association of State Highway and Transportation Officials (AASHTO) I-beam series has been in use in the United States since it was adopted in 1957.[1]

More recent developments are given in the *PCI Bridge Design Manual*[2] and various state departments of transportation websites. Developments include the AASHTO-PCI bulb tee and the deck bulb tee in the 1980s, the Nebraska (NU) I-beam and the Washington State wide-flange girder in the 1990s, and a number of other state shapes in the 2000s and 2010s. As the shape of the I-beam evolves with time, general trends are observed.

The new shapes generally have a large bottom flange to allow for placement of more strands and thus an increase in span capacity. The top flange is generally wider and thinner than the original AASHTO beams, with flange width as large as 10 ft, to reduce or eliminate cast-in-place concrete deck forming and thus provide accelerated bridge construction. A number of owners are still using older shapes due to the capital investment involved in creating new forms and the corresponding modifications that might be necessary in the prestressing beds. Yet, another group of owners are seriously considering updating their shapes with new forms. In both cases, it is helpful to understand the features of forming and producing prestressed concrete I-beams. This understanding would help with the decisions needed to modify existing forms and the features desirable for acquiring new forms.



Example illustration of I-beam forms. Figure: Helser Industries.

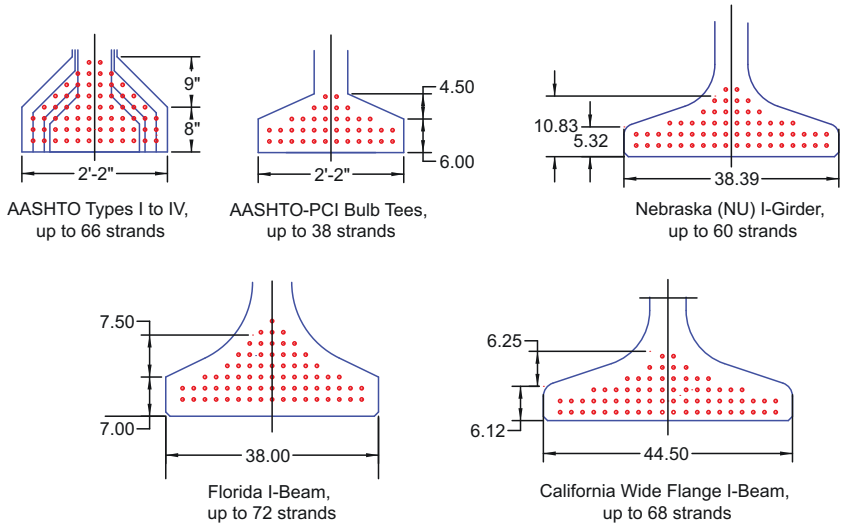
Two side-by-side beds with Nebraska (NU) beam forms. Note the stressing head in the foreground. Photo: e.Construct.USA LLC.

This article gives an overview of the historical development of I-beam shapes and the method used to produce I-beams. It discusses the desirable features in optimizing I-beam shapes. Finally, it presents options for possible modification of existing forms.

Understanding I-Beam Production

I-beams are often produced in prestressing beds that are 350 to 600 ft long. The forms are primarily two-sided forms that are moved in toward each other. They are tied at the bottom and at the top. At the bottom, a flat steel plate runs the full length of the bed. It forms the bottom face of the I-beam and is called the pallet. The pallet is elevated above the foundation to allow for tying of the side forms and for strand draping, if this option is utilized. The tops of the two side forms are tied together with ties at about 30 in. spacing, called the yokes.

Because the forms are continuous for the length of the bed, they are made of 40-ft-long sections that are bolted tightly together to ensure straightness for their full length



Bottom flange dimensions of selected I-beams. Note: All dimensions are in inches, unless marked otherwise. Figure: e.Construct.USA LLC.

and minimize the visibility of seams. The ends of each beam length are defined by steel or wood bulkheads that have holes to allow passage of the strands. Any special end reinforcement would require that additional holes be drilled in the bulkheads and would increase time and cost of production. The figure to the left shows a typical prestressing bed with the forms spaced out.

It is not desirable to have bars projecting from the concrete I-beam except at the top face, which is not formed and is expected to have composite action, or interface shear, reinforcement projecting from it.

The technology of forming for I-beams has evolved over the years to use hydraulics to move the forms and to react to demands for increasingly larger beams, which are nearing 10 ft in depth. The following sections illustrate the evolution of I-beam shapes, recommendations for future shapes, and, most importantly, the cost-effectiveness of various modification of existing forms to accommodate project needs

Evolution of I-Beam Shapes

Bottom Flange

The figure above shows the bottom flange dimensions for several types of I-beams. They are shown in a chronological order up to the most recent, which is the California I-beam that was introduced about 5 years ago. Also shown is the maximum number of strands that can be housed in the bottom flange.

Each shape corresponds to a series of beam sizes. For example, the AASHTO I-beams vary in depth from 28 to 54 in. Except for the early AASHTO shapes, the bottom flange dimensions are kept constant regardless of beam depth. The reason for keeping the bottom-flange dimensions constant is to simplify forming. The pallet over which the form is installed and that forms the bottom face (soffit) of the I-beam, is important for the design of the prestressing bed and the dimensions of the stressing head. It would be a significant capital expense to later increase it. It is also time-consuming to change pallets on a bed when changing beam types, so this is an added cost of production. Thus, it is preferable to keep it constant. Also, keeping the bottom flange geometry constant for the series of beams being considered would simplify forming for cast-in-place diaphragms, if needed.

Another observation is that the bottom flange size continues to increase with time. This development is the primary reason why I-beams recently have reached lengths of over 200 ft. At this time, the Florida I-beam has the largest bottom flange, which has a record length of 209 ft for a pretensioned I-beam.

It is possible to block out the outside edges of the bottom flange in order to create a smaller bottom flange, if necessary. However, this would be a modification that needs to be studied carefully. A narrower bottom flange is less stable during handling and shipping. The amount of concrete and weight saved may not justify the change in section and reinforcement details.

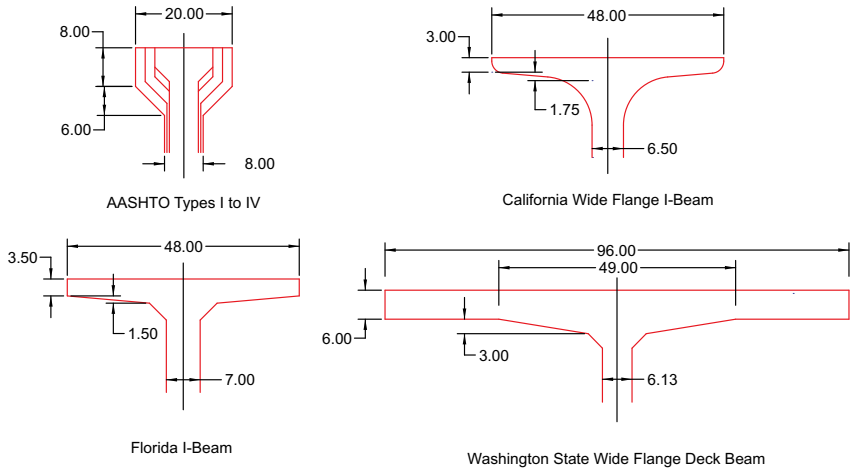
Web

The I-beam web is a significant part of the section area and beam weight. Using the smallest possible web width produces efficiencies, as long as there is room to place two layers of shear reinforcement on the two web faces with the minimum *AASHTO LRFD Bridge Design Specifications*-specified cover of 1 in. Shear strength is generally not a problem if enough shear reinforcement is placed. This is especially true since the adoption of the modified compression field theory in the AASHTO LRFD specifications. The AASHTO LRFD specifications allow as much shear stress as 0.25 f_c' in the web, which is nearly 250% of what had been allowed under the AASHTO *Standard Specifications for Highway Bridges* with a limit of 10 $\sqrt{f_c'}$.

In the early days of bridge I-beams, several states, such as Washington and Colorado, had I-beams with web widths of 5 in. However, these sections have largely been abandoned. Common widths are 6 in. (Nebraska and other states), 6.5 in. (California), and 7 in. (Florida and other states).

Reducing the web width for existing forms is a difficult process and would only serve the purpose of reducing the beam weight, which would not be recommended. Increasing the web width for the purpose of increasing shear capacity is seldom needed. However, it may be needed to allow a relatively shallow beam to span farther by creating more space for the pretensioning strands.

More commonly, the web width may have to be increased by 1 or 2 in. to allow for post-tensioning in addition to pretensioning. For example, Nebraska uses a 7-in. web width for I-beams with post-tensioning tendons containing up to fifteen 0.6-in.-diameter strands. California allows a web width of 8 in. with the same size tendon. As indicated earlier, the web is a significant part of the



Top flange dimensions of selected I-beams. Note: All dimensions are in inches, unless marked otherwise. Figure: e.Construct.USA LLC.

cross section. It significantly impacts the area and weight, but to a lesser extent the moment of inertia.

The most common way to increase the web width for existing forms is to space out the side forms by the required widening amount. This change requires modification of the set up at the bottom and at the top of the forms, and also a change in the bulkheads that define the end faces of the beam. At the bottom, steel spacers are provided. In the top, the "yoke," which holds the two side forms together and prevents spreading due to lateral pressure of the fresh concrete, is extended by the required web widening.

In most modern I-beam forms, it is allowed to have one set of bottom flange forms and one set of top flange forms for all beam depths. These two sets of forms add up to the depth of the shallowest beam in the series. For deeper beams, steel blocks are placed and bolted between the bottom and top flange forms. Alternatively, a separate set of the top forms is made for each of the beam depths in order to reduce the number of form parts, reduce the joint seams, and improve precision.

Top Flange

The top flange dimensions have varied considerably over the years. The figure above shows a sampling of top flanges of I-beams in use in the United States. The AASHTO Types I through IV represent old thinking, which is seldom used for newer products. The flanges are narrow and deep, and they change in size with the change in beam size.

Recently, the most popular flange width has been 48 in. and the flange is relatively thin. The flange needs to be adequate for resisting the fresh weight of the deck concrete, and it also helps stiffen the weak axis of the I-beam to allow for resistance to buckling during handling and shipping. This 4-ft width has been shown to be adequate for I-beams up to nearly 210 ft in length. After the deck concrete becomes composite with the I-beams, the value of the top flange in resisting subsequent loads diminishes. The deck becomes the "compression flange" of the beam/deck system in the positive moment zone of the span.

Integral precast concrete beams and decks have become more popular due to the desire for accelerated bridge construction, which lends itself to not having field-placed deck forms and cast-in-place concrete deck with the required labor and time for placement, finishing, and curing. An example of a deck beam is the recently introduced Washington State wide flange deck beam, the top flange of which is shown in the figure above. The bottom flange is similar to that of the Nebraska beam. A major disadvantage of a deck beam with a very wide top flange is the heavy weight. Thus, the system lends itself to relatively shallow depths with moderate piece lengths.

The value of having forms that will accommodate relatively wide flanges is the flexibility it offers. One of the simplest modifications that can be made on I-beam forms is to block out some of the top flange width with either wood or steel forming.

Summary

Precast, prestressed concrete I-beams are structurally efficient and popular for bridge applications. Bridge designers should be aware of the forms available in the vicinity of the bridge being designed. Costs and delays are caused by form modification, thus the decision to modify the beam shape should be based on the solution that produces the least time and cost for the project. It is most difficult to modify the bottom flange shape and least difficult to modify the top flange shape. The top-flange modification would involve placement of block-out forms to reduce the flange width.

Enlarging the section by making it wider is possible, however, the only feasible way to do this is to space the forms out by the necessary increase in form width. Such increase in width will have to be consistent for the full depth of the member including the flanges. If spacing out the side forms is desirable to increase member width, corresponding modification would be required for the pallet which forms the bottom face, the yokes that tie the two side forms together, and the bulkheads, which define the product length. These changes must be done in collaboration with the applicable precasters and their forming suppliers.

If an owner desires to create a new series of I-beams, the desirable features are a large bottom flange, a narrow web, and a wide top flange. This would allow for initial economy and future flexibility.

References:

1. Zollman, Charles C. 1981. "The End of the Beginnings," In Reflections on the Beginnings of Prestressed Concrete in America, JR-H-81, Chicago, IL: Precast/Prestressed Concrete Institute (PCI).
2. PCI. 2014. *PCI Bridge Design Manual*, 3rd Edition, 2nd Release, Chicago, IL: PCI.

Dr. Maher Tadros is principal with e.Construct. USA LLC in Omaha, Neb.

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

www.sacatonabc.com/home.html
This is a link to the website for the Sacaton Bridge Project that was mentioned in the Authority article on page 50. A short video of the slide-in of the bridge is available on the Project Updates page.

www.youtube.com/watch?v=NttbEeAOpHA
This is a link to a video of the slide-in of the Lardo Bridge mentioned in the Project Profile on page 18.

www.slideinbridgeconstruction.com
This is a link to a website for a series of training webinars on lateral bridge slides developed by the Colorado Department of Transportation (CDOT) on behalf of FHWA and Every Day Counts (EDC). The webinars focus on the perspectives of owners, the engineer/designer, and the contractor/constructor. This website provides further information on bridge slide technology that is discussed in two articles in this issue.

Bridge Technology

NEW www.fhwa.dot.gov/bridge/lrfd/webinar.cfm
This is a link to the FHWA website that provides access to 17 recorded webinars on implementation of the Load and Resistance Factor Rating (LRFR) Method. A new webinar has been recently added: "Federal Bridge Formula Weights and State-Specific Legal Loads Web Conference Seminar."

www.youtube.com/user/TheESCSI
This is a link to a website with training videos produced by the Expanded Shale, Clay and Slate Institute (ESCSI). They include a new two-part video series on how to easily produce internally cured concrete. Part One focuses on lightweight aggregate preparation, moisture testing and mix design. Part Two focuses on ready-mix plant charging and batching considerations.

www.concretebridgeviews.com
This is a link to access the 79 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. The recently released issue focusses on durability of bridge decks and joints.

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.

http://abc-utc.fiu.edu/index.php/technology/monthly_webinar_archive/
This is a link to the archive webpage for monthly webinars presented by the Accelerated Bridge Construction (ABC) University Transportation Center at Florida International University. Many of the webinars address the use of concrete for building bridges using ABC methods.

Bridge Research

NEW www.dot.state.fl.us/research-center/Completed_Proj/Summary_SMO/FDOT-BDV31-977-11-rpt.pdf
This is a link to a recently released Florida Department of Transportation report that investigates the extent to which corrosion of steel can occur in submerged portions of reinforced concrete structures in marine environments through field studies of decommissioned pilings.

NEW www.intrans.iastate.edu/research/documents/research-reports/grouted_coupler_connections_for_ABC_w_cvr.pdf
This is a link to a recently released Institute for Transportation at Iowa State University report that evaluates the structural and durability performance of the grouted coupler connection details utilized between the drilled shaft and precast pier column and between the precast pier column and the precast pier cap on the Keg Creek Bridge.

NEW www.intrans.iastate.edu/research/documents/research-reports/grouted_coupler_connections_for_ABC_w_cvr.pdf
www.intrans.iastate.edu/research/documents/research-reports/negative_moment_reinforcing_w_cvr.pdf
These are a links to recently released Institute for Transportation at Iowa State University reports that 1) considers the effect of bridge width on deck cracking and other parameters including bridge skew, girder spacing, and abutment type, and 2) investigates negative moment reinforcement and the performance of bridge decks over intermediate supports.

NEW www.dot.state.fl.us/research-center/Completed_Proj/Summary_SMO/FDOT-BDV31-977-11-rpt.pdf
This is a link to a recently released Florida Department of Transportation report that evaluates the performance and usability of internally-cured concrete by using lightweight aggregates for bridge decks and concrete pavement slabs in Florida.

NEW www.oregon.gov/ODOT/TD/TP_RES/docs/Reports/2015/SPR750_Final_StengtheningGirders.pdf
This is a link to a recently released Oregon Department of Transportation report that investigates methods for strengthening deficient flexural steel anchorages on bridges by using supplemental surface bonded metallic reinforcing bars.

NEW www.roads.maryland.gov/OPR_Research/MD-13_SP309B4G_Stainless-Steel-Prestressing-Strands_Report.pdf
This is a link to a recently released Maryland State Highway Administration report that synthesizes critical information about stainless steel and other remedies that have been used to replace corroded prestressing steel strands and bars or prolong the corrosion rate and presents cases studies and life-cycle cost analysis studies of these alternate materials to conventional steel.

FHWA Partnerships to Advance Bridge State of Practice

by Reggie Holt, Federal Highway Administration

The Federal Highway Administration's (FHWA's) responsibility is to provide infrastructure policy, investment, management, stewardship, and oversight leadership as it expands and preserves the nation's multi-trillion dollar investments in highways with a priority on public safety. FHWA positions itself and the transportation community for the future by executing coordinated research efforts and promoting technologies that modernize project development, materials testing, construction practices, and the transportation workforce. This includes training the FHWA workforce to understand and embrace new and innovative ways to achieve the results that the public expects and deserves.

Need for Partnering

FHWA's Office of Bridges and Structures routinely partners with its stakeholders to fulfill the role stated previously. These stakeholders typically include bridge owners/departments of transportation, industry, academia, contractors, and design practitioners. Partnerships with these stakeholders are essential in identifying and developing technologies that can effectively advance the state-of-practice. FHWA typically engages its stakeholders by including them on technical working groups that give input to strategic planning and on technical review panels that guide and review progress on specific work initiatives. Cooperative agreements are another mechanism used by FHWA to facilitate partnerships with its stakeholders. Cooperative agreements are contractual agreements in which both parties agree to share the cost and technical development responsibilities for a particular project.

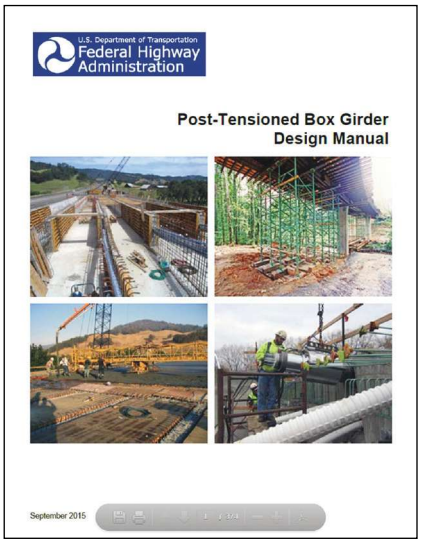
Recent Partnering Successes

FHWA's Office of Bridges and Structures has many technology deployment projects that are either under development or recently completed and that engage our stakeholders. A technology needs to satisfy many diverse requirements to be successful. It needs to

- address an important need,
- be technically sound,
- perform well, and
- be implementable with our nation's workforce.

These requirements cannot be met without interaction and input from stakeholders representing multiple groups in the bridge community.

The following sections list a few recently completed projects that benefited greatly from having stakeholder involvement. Using the four projects listed as a sample to gauge stakeholder involvement reveals that a total of 32 stakeholders were used with representation from bridge owners, academia, practitioners, and industry. This stakeholder outreach is actually greater than the numbers shown due to inter-relationships between many stakeholders, for example, industry organizations include multiple practitioners, owners, and academic members.



All Figures: Federal Highway Administration.

Post-Tensioned Box Girder Design Manual

Many within the bridge design community expressed the need for up-to-date design guidance on post-tensioned (PT) box girders. The last published guidance on this topic was the *Post-Tensioning Institute's (PTI) Manual* published in 1972. The design of this type of bridge has significantly changed since the publication of this design manual and new guidance on the current state-of-practice was greatly needed.

To address this need, FHWA developed a new comprehensive manual through a cooperative agreement with Lehigh University and Corven Engineering. Stakeholders were involved in the review and development of this manual through review panel participation and a cooperative agreement contract.

Another partnering feature for this manual will be its future maintenance. Two industry partners, PTI and the American Segmental Bridge Institute (ASBI), will be forming a new joint committee on PT bridges, which will maintain and expand the content in this manual.

An electronic version of this manual is available at http://www.fhwa.dot.gov/bridge/concrete/hif15016.pdf.

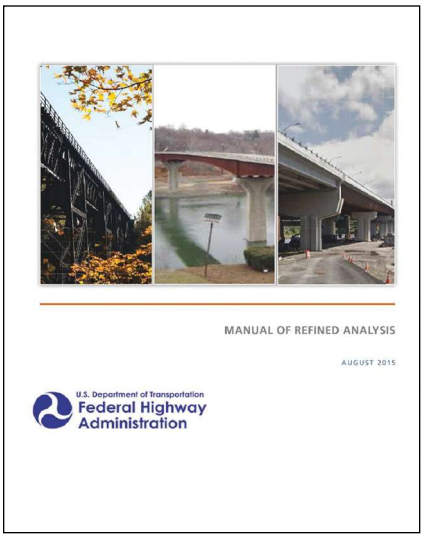
List of Stakeholders

Academia: Lehigh University

Departments of Transportation: California (CALTRANS) and Kansas (KSDOT)

Practitioners: Corven Engineering, Michael Baker, Buckland and Taylor

Industry: PTI and ASBI



Manual on Refined Analysis

A international scan (whose study team was made up of multiple U.S. stakeholders) determined that engineers in the United States lagged behind their European counterparts in the use of advanced modeling tools and procedures to design and assess bridges. In an effort to expand the use of refined analysis in the United States, FHWA is developing a new comprehensive manual on refined analysis through a cooperative agreement with Lehigh University and Modjeski and Masters.

Stakeholders were involved in the review and development of this manual through review-panel participation and a cooperative agreement contract. A draft version of the *Manual on Refined Analysis* is currently available online for review and comment https://www.fhwa.dot.gov/bridge/refined_analysis.pdf. For more information, see the article by Holt and Kozy in the Fall 2015 issue of *ASPIRE.* ™

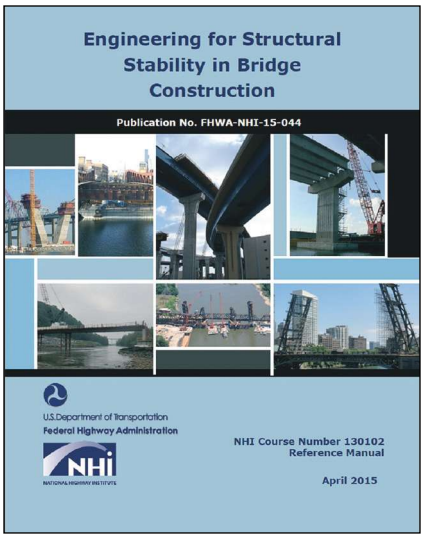
List of Stakeholders

Academia: Lehigh University

Departments of Transportation: California (CALTRANS) and North Carolina (NCDOT)

Practitioners: Modjeski and Masters, HDR

Industry: Precast/Prestressed Concrete Institute (PCI) and National Steel Bridge Alliance (NSBA)



Engineering for Structural Stability in Bridge Construction Manual and Training

There is large disparity in the standard of care that different engineering professionals will employ for the verification of bridge erection procedures. The bridge community identified a need to develop and disseminate guidance and criteria to improve structural safety in the construction process. The objective of this guidance and course is to educate engineers on the behavior of steel and concrete girder bridges during construction.

It also endeavors to teach them to identify vulnerabilities and engineering methods to investigate the structure's strength and stability at each critical stage. This is done within the practical context of engineering, development, verification, or review of erection plans. Stakeholders were involved in the review and development of this manual and training through review panel participation.

An electronic version of this manual is available at http://www.fhwa.dot.gov/bridge/pubs/nhi15044.pdf. In addition, training can be scheduled through the National Highway Institute (NHI) at http://www.nhi.fhwa.dot.gov/training/training_products.aspx (enter course 130102).

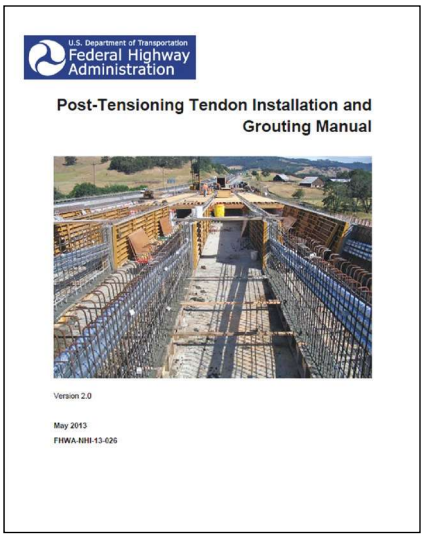
List of Stakeholders

Academia: University of Texas

Departments of Transportation: Pennsylvania (PENNDOT), Florida (FDOT), and Kansas (KDOT)

Practitioners: Collins, Genesis Structures, HDR

Industry: PCI, High Steel/Concrete, NSBA



Post-Tensioning Tendon Installation and Grouting Manual and Training

FHWA published the first version of this manual back in 2004, however many advancements have been made in PT systems and testing since this publication. In order to provide guidance on the most recent PT technologies, this manual was recently updated and released along with new web-based training.

This guidance and training deploys the latest in PT technology to individuals involved in the design, installation, grouting, and inspection of PT tendons in prestressed concrete bridges. Stakeholders were involved in the review and development of this manual and training through review panel participation.

An electronic version of this manual is available at https://www.fhwa.dot.gov/bridge/construction/pubs/hif13026.pdf. In addition, training can be scheduled at http://www.nhi.fhwa.dot.gov/training/training_products.aspx (Enter course number 130103).

List of Stakeholders

Departments of Transportation: Minnesota (MNDOT) and Texas (TXDOT)

Practitioner: Corven Engineering

Industry: ASBI, PTI, PCI, GTI, and VSL

Future Partnering Opportunities

A large number of stakeholders were involved with the previously listed projects. FHWA understands that in order to effectively advance the state-of-practice for our nation's bridge community it needs to maximize engagement and collaboration with its stakeholders. FHWA has had great success working collaboratively with its stakeholders on past projects and plans to continue to keep our stakeholders significantly involved in future projects.

STATE

OHIO

State funding focuses on repair and replacement of local bridges

by Pieter Wykoff and Tim Keller, Ohio Department of Transportation

The Rich Street Bridge was the perfect back-drop to Columbus' famed Red, White, and Boom Fourth of July celebration. Photo: Burgess & Niple.

With nearly 28,000 bridges, Ohio ranks second in the nation for the number of bridges connecting state and local roads. These are all critical to the state's transportation infrastructure. Most of these bridges are shorter-span bridges on local road systems, but the large Ohio inventory also contains a number of major, iconic structures. Several of the more-recently constructed concrete bridges in this category have been featured in past issues of *ASPIRE*.™ These include the Veterans' Glass City Skyway in Toledo (Summer 2007), the Rich Street Bridge in Columbus (Fall 2012), and the Jeremiah Morrow Bridge in Warren County (Winter 2014), all of which are owned by the Ohio Department of Transportation (ODOT). A brief description of each of these beautiful concrete bridges follows.

Veterans' Glass City Skyway—Toledo

The Veterans' Glass City Skyway Bridge carries Interstate 270 (I-270) across the Maumee River at Toledo, Ohio. The cable-stay supported main spans reach 612.5 ft on each side of the single tower. The entire structure is 8800 ft long and was constructed using 3050 precast concrete box-girder segments. In the main spans, the twin boxes are connected using delta frames that transfer loads from the box girders to the cable stays. The sides of the tower are encased in glass that is reflective during the day and is lit from behind with arrays of multi-colored LEDs at night. The bridge was opened to traffic in 2007.

Rich Street Bridge—Columbus

The Rich Street Bridge crosses the Scioto River in the heart of Columbus, Ohio. The new five-span bridge is 563 ft long and carries three lanes of traffic and two 10-ft-wide sidewalks. Precast semi-lightweight concrete arch segments support haunched precast concrete girders and a cast-in-place concrete deck. All elements of the structure were post-tensioned for structural efficiency and durability. The bridge was completed in 2012.

Jeremiah Morrow Bridge—Warren County

The Jeremiah Morrow Bridge is the tallest bridge in Ohio and is the first cast-in-place segmental concrete box-girder bridge to be constructed in the state. The six-span twin bridges carry Interstate 71 (I-71) nearly 240 ft above the scenic Little Miami River in Warren County, Ohio. The two main spans are 440 ft long.

The single-cell box girders have a top flange width of 55 ft and vary in depth from 12 ft deep at midspan to 25 ft deep at the piers. The northbound bridge was opened to traffic in 2013; the second bridge is still under construction with completion expected in the fall of 2016.

Improving Local Bridges

The Buckeye State's bridge inventory is better than most when it comes to nationwide reports on statewide bridge conditions, but the ODOT continues to rehabilitate and repair bridges on most interstates, and U.S. and state routes. Many of the bridges on local routes, which are owned and maintained by counties and cities, are in need of more-costly repairs.

To address the condition of these local bridges, Governor John R. Kasich launched the Ohio Bridge Partnership Program in 2013 to help cities and counties repair 220 local bridges. The three-year, $120-million program started the following year by fixing the first 40 structurally deficient bridges on the local transportation system. Reductions in original cost estimates allowed program planners to add 10 more bridges to the list in 2014, bringing the total number of bridges receiving repairs to 230. The next transportation budget provided an additional $10 million to the program, extending the program through 2017. In 2015, 84 local system bridges in Ohio were slated for reconstruction as part of the program.

"It's been a much-needed shot in the arm for

Nighttime shot of the mainspan cantilever segment erection for the Veteran's Glass City Skyway Bridge from the north shore of the Maumee River. Photo: FIGG.

a lot of Ohio counties. It's helped some counties get caught up and get through the replacement of deficient bridges. This is a program where you can really see the results," said Fredrick B. Pausch, executive director of the County Engineers Association of Ohio (CEAO). "For any county engineer, their number one job is the safety of the traveling public so this fits perfectly into what they try to do every day as they try to make roads and bridges safer for everybody."

To qualify for the program, local bridges had to meet the federal bridge definition of 20 ft or longer, be identified as structurally deficient, and be open to and carrying vehicle traffic. "It wasn't anything political; it was based on engineering numbers and based on the worst county bridges in the state of Ohio. So that process was very well done," said Pausch.

Investing in local bridges is not new in Ohio. ODOT already spends about $35 million each year to help repair local bridges, but counties and cities are required to match at least 20% of the costs. The state funds the new Ohio Bridge Partnership Program at 100%, which means local governments share none of the costs.

"This new program complements the existing program through a much larger effort. We work with the CEAO all the time, but this is going at a very fast pace and we are funding it 100%," said Andrea Stevenson, ODOT's Bridge Partnership Program manager.

"This is going to be very fast paced. Typically in the past, bridges might go to construction about 3 to 4 years after they are selected for repairs. But under the new program, bridges are going to construction in some cases within 6 months," said Stevenson.

Background

The new bridge program almost did not happen. Just a few years prior, ODOT announced a $1.6 billion transportation funding deficit. Transportation projects that had been promised to communities for years were pushed back and forced to wait for new funding, in many cases, for more than a decade.

ODOT immediately began a top-to-bottom review for improved operational efficiencies and better ways to spend scarce resources. Through these efforts, ODOT was able to identify $600 million in operating costs that could be redirected to the transportation construction program. Improved department efficiency, workforce and vehicle fleet reductions, higher than projected gas tax receipts, and the elimination of federal earmarks allowed ODOT to move even more money into the budget for new construction projects throughout the state.

Then, the Ohio General Assembly passed Governor Kasich's "Jobs and Transportation Plan" in 2013. The plan authorized the Ohio Turnpike and Infrastructure Commission to sell up to $1.5 billion in revenue bonds, backed by future toll revenue. When combined with federal and local money the state will receive, the Jobs and Transportation Plan is expected to result in an investment of more than $3 billion in Ohio's transportation infrastructure over 6 years.

The innovative plan, along with ODOT's careful management of resources by reducing overhead costs, meant the department could look at new ways to invest in infrastructure spending and assist Ohio's counties and cities with their deficient bridges. The Ohio Bridge Partnership Program was born.

Seeing Results

The first bridge completely reconstructed as part of the new program re-opened in August 2014 in Meigs County. The bridge on Tornado Road (or County Road 124) over the Yellowbush Creek just outside of Rutland, Ohio, took 6 weeks to repair and cost $492,570.

Allen County in the northwestern part of Ohio is in the process of benefiting from a state investment of nearly $5 million to repair or replace seven bridges. Allen County Engineer

Tim Piper says that with a total county transportation budget of $6 million, any assistance to bridge repairs is tremendously helpful.

"We had a total of about 23 bridges that were posted with weight limits and now seven of them will be opened up to traffic. This means trucks using the local routes from stone quarries, or from businesses, will no longer have to detour around these bridges. So, for Allen County, we're actually wiping out about one-third of our bridges that were posted with weight limits," said Piper. Posted weight limits on local bridges also impact schools and fire and rescue departments where, in some cases, buses and fire trucks are forced to detour for miles.

"This is certainly going to save a lot of time and money for our schools and our local emergency management organizations who can't go across many of these bridges due to the weight limits that are posted on them," said Piper.

Jeremiah Morrow Bridge, looking downstation from Pier 5 Southbound, towards the Little Miami River. Photo: Ohio Department of Transportation.

Repairing and replacing more than 230 bridges is a reduction of nearly 12% in the state's structurally deficient bridges.

Repairing and replacing more than 230 bridges is a reduction of nearly 12% in the state's structurally deficient bridges. Additional investments are reducing the statewide total of structurally deficient bridges even further.

"Anytime you can take 230 structurally deficient bridges off the inventory, that's a very positive thing and that frees up money for county engineers to spend on other priority bridge projects," said Tim Keller, ODOT's administrator in the Office of Structural Engineering.

"Concrete is a vital material for Ohio's bridge inventory. We have a significant inventory of conventional concrete bridges as well as prestressed concrete box beam and prestressed I-beam bridges. In a typical year, we will build about 300 new bridges in Ohio with around 1,000,000 ft^2 of deck area. Typically, between 70 to 80% are concrete structures," he says. During the heyday of Interstate construction, Ohio's bridges were built primarily with steel. In the past 30 years, Ohio's new bridge market has transformed from predominantly steel bridges to predominantly concrete bridges due to lower costs and concrete's versatility.

Pieter Wykoff is the public affairs officer and Tim Keller is administrator for the Office of Structural Engineering with the Division of Communications for the Ohio Department of Transportation in Columbus.

GILA RIVER INDIAN COMMUNITY

Gila River Bridge Replacement

by Shawn Sheble, FNF Construction Inc., and Mark Chase, AZTEC Engineering

Completed Gila River Bridge. Photo: Greg Harasba.

Replacement of the Gila River Bridge on the Gila River Indian Community in Arizona was a construction manager/general contractor (CM/GC) project administered by the Gila River Indian Community Department of Transportation (GRIC DOT). The purpose of the project was to replace the existing Sacaton Road (Route 7) Bridge over the Gila River to improve roadway safety and hydraulic capacity. The previous 140-ft-long bridge was built in 1961 with a cast-in-place concrete slab on precast concrete rectangular beams, over a mostly dry river channel. The existing two lane bridge with a clear roadway width of 28 ft was replaced with a new two lane bridge with a clear roadway width of 44 ft.

A Strategic Highway Research Program (SHRP2) grant helped direct how this project was delivered through the use of the SHRP2 toolkit and its emphasis on accelerated bridge construction (ABC) techniques. The GRIC DOT selected a contractor through a qualifications-based selection to work with their designer. The team worked together for several months during preconstruction to revise the design and select a bridge type using ABC methods to minimize traffic disruptions. Much of the success of this project can be tied to a partnering session held after the contractor was brought on board. At this meeting, the project team set project goals and rules of engagement that drove the bridge preconstruction and construction.

During construction of the new bridge, the older span remained open to traffic. The new bridge was constructed in halves, one on each side of the existing structure. This approach was chosen to strategically address two of the project's key goals: reducing temporary works and implementing a simple, effective bridge sliding system. To find a more cost-effective way to replace the Sacaton Road Bridge, the bridge wingwalls were reoriented to run parallel to the abutments instead of perpendicular and they were designed simply as an extension of the abutment caps. In this configuration, they served initially as the temporary supports for half of the bridge and then were re-used as the permanent wingwalls; therefore, no temporary works were required at either abutment and the incremental cost of the abutment cap was less than the cost of a temporary abutment system.

Last segment of bridge sliding into place. Photo: Shawn Sheble.

Another benefit of constructing the new bridge in halves was that the slide system could be much smaller. The Sacaton Road Bridge used a very basic slide system consisting of greased abrasion resistant steel shoes under each beam and a transverse plate to slide on.

One last benefit to constructing the bridge in halves was that one of the halves was actually more than half of the bridge width and therefore heavier than the other piece. This larger half actually served as the anchor from which the smaller half was pulled into final position against its stop, which was the permanent horizontal shear key at the pier and the abutment. Once the smaller piece was set, the larger half was pulled into position using the smaller piece and stop as the anchor. The last steps in the slide process involved lifting the bridge vertically with single acting hydraulic jacks to remove the slide plate, set the elastomeric bearings and place the closure pour in the diaphragms and bridge deck.

The project team was able to successfully design and construct a bridge replacement with just a single weekend closure plus a 9-day full closure as opposed to the estimated 6-month closure required in a conventional construction approach. A very successful SHRP2 Showcase was held where bridge engineers from across the country were able to see the final bridge slide component being moved into place.

From early in the process, all team members had to be on the same page and maintain trust in the other for this to work. This was the case and lead to the completion of the first bridge slide by an Indian community in the country. None of this would have occurred without the guidance and support provided by the Federal Highway Administration Federal Lands Transportation staff, the efforts put out by GRIC staff, their designer, and their contractor.

Shawn Sheble is vice president-structures manager for FNF Construction Inc. in Tempe, Ariz. Mark Chase is a principal with AZTEC Engineering in Phoenix, Ariz.

2016 Interim Changes Related to Concrete Structures, Part 1

by Dr. Dennis R. Mertz

At their 2015 annual meeting, hosted by the New York State Department of Transportation (NYSDOT) in Saratoga Springs, N.Y., in April, the American Association of State Highway and Transportation Officials (AASHTO) Subcommittee on Bridges and Structures (SCOBS) considered and adopted seven agenda items specifically related to concrete structures. Technical Committee T-10, Concrete Design, developed agenda items 4 through 9 and moved them to the subcommittee ballot for consideration in Saratoga Springs. In conjunction with Technical Committee T-5, Loads and Load Distribution, Technical Committee T-10 also developed agenda items 2 and 4 over the past several years and moved them to the subcommittee ballot. The agenda items represent revisions and additions to the 7th edition of the *AASHTO LRFD Bridge Design Specifications*. This column reviews the 2015 concrete-structures agenda items, which are the 2016 Interim Revisions, that have been published and are now available from AASHTO.

Agenda Item 2

Agenda item 2 represents a major revision to the wind-load provisions. It begins by making revisions to the descriptions of the limit-state load combination dealing with wind, and the load factors for wind load, based upon new wind-load provisions applying a 3-sec. wind gust speed with 7% probability of exceedance in 50 years (mean return period of 700 years). Wind load provisions in previous editions of the AASHTO LRFD specifications are based upon fastest-mile wind speed measurements.

The previous provisions allowed the use of a 100-mph fastest mile base wind speed. This wind speed was used for the Strength III limit-state load combination. The Strength V, Service I, and Service IV limit-state load combinations were based upon constant wind speeds. Instead of calculating the wind pressure for the specified constant wind speeds, the 100-mph wind speed is used and the Strength V, Service I, and Service IV limit-state load factors for wind loads on the structure are adjusted to scale the resulting factored wind load to the implied load factor and constant wind speed specified for each load combination.

In the revised provisions, the load factor for wind is 1.0 for all load combinations applied to the wind pressure calculated for the wind speed specified for each limit-state load combination. In addition, Article 3.8 titled "Wind Load: WL and WS" is replaced in its entirety by a new article defining the new wind-load provisions. These provisions provide consistent reliability across different regions and locations unlike those that they replace. Although many typical bridges will not see a change in design due to wind, those structures that fall between the typical range and those needing site-specific considerations will be more reliable through the application of the more robust wind provisions.

Finally, Article 5.14.2 is revised to make all of the load factors for wind during construction of segmentally constructed concrete bridges in Table 5.14.2.3.3-1 equal to 1.0 (to be consistent with the previous discussion). The revision further leaves the specification of the wind speed for the various service limit-state load combinations to the owner, with the only exception being a specification for the minimum wind speed of 70 mph for erection-stability analysis of cantilever construction in lieu of a better estimate by analysis or meteorological records.

Agenda Item 4

Load factors for the Service III limit-state load combination—the check of tensile stress in prestressed components—are addressed in agenda item 4. The calibration of the service limit states for concrete components[1] concluded that typical components designed using the refined estimate of time-dependent losses method, which was incorporated in the specifications in 2005 and includes the use of transformed sections and elastic gains, have a lower reliability index against flexural cracking in prestressed components. This is true when compared against components designed using the prestress loss calculation method specified prior to 2005 based on gross sections and not including elastic gains. For components designed using the currently specified methods for instantaneous prestressing losses and the currently-specified refined estimates of time-dependent losses method, an increase in the load factor for live load from 0.8 to 1.0 is required to maintain the level of reliability against cracking of prestressed concrete components inherent in the system. Agenda item 4 inserts a table into Article 3.4.1 specifying a live-load load factor of 1.0 for prestressed concrete components designed using the refined estimates of time-dependent losses as specified in Article 5.9.5.4 in conjunction with taking advantage of the elastic gain and 0.8 for all other prestressed concrete components. The corresponding appropriate revisions to the *Manual for Bridge Evaluation* were also included in this agenda item.

Agenda Item 5

Agenda item 5 integrates lightweight concrete (LWC) into the entirety of Section 5 in a more consistent and accurate manner based upon the work of Greene and Graybeal[2-4] who presented these changes in greater detail in an article in the Summer 2015 issue of *ASPIRE*.™ A revised definition of LWC is provided to include concrete with lightweight aggregates up to a unit weight of 0.135 kip/ft^3, which is considered the lower limit for normal-weight concrete. Also the terms "sand-lightweight concrete" and "all-lightweight concrete" are removed in the proposed definition to allow other types of LWC mixtures. The concrete density modification factor, which has been in earlier editions of the specifications, is now defined as λ and is introduced to modify various traditional resistance equations, stress limits, and development lengths. Finally, the shear strength reduction factor, ϕ, for LWC has been set equal to the factor for normal-weight concrete.

The remaining concrete agenda items from the 2015 SCOBS meeting, agenda items 6 through 9, will be discussed in a future column.

References

1. Wassef, W. G., et al. 2014. *Calibration of LRFD Concrete Bridge Design Specifications for Serviceability*, NCHRP Web-only Document 201, Transportation Research Board, National Research Council, Washington, DC.
2. Greene, G. G., and B. A. Graybeal. 2013. *Lightweight Concrete: Mechanical Properties*, Report No. FHWA-HRT-13-062, Federal Highway Administration, Washington, DC.
3. Greene, G. G., and B. A. Graybeal. 2014. *Lightweight Concrete: Development of Mild Steel in Tension*, Report No. FHWA-HRT-14-029, Federal Highway Administration, Washington, DC.
4. Greene, G. G., and B. A. Graybeal. 2015. *Lightweight Concrete: Reinforced Concrete and Prestressed Concrete in Shear*, Report No. FHWA-HRT-14-15-022, Federal Highway Administration, Washington, DC.