ASPIRE®
THE CONCRETE BRIDGE MAGAZINE
FALL 2019
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WSDOT inspects 223-ft-long, 247-kip light-weight concrete girder

Prestressed Concrete Girders Achieve Record Lengths

Tacoma, Washington

BRIDGES OF THE FOOTHILLS PARKWAY
Great Smoky Mountains National Park

DWIGHT D. EISENHOWER VETERANS MEMORIAL BRIDGE
Anderson, Indiana

MARC BASNIGHT BRIDGE
Dare County, North Carolina

COURTLAND STREET BRIDGE
Atlanta, Georgia

Presorted Standard
Postage Paid
Lebanon Junction, KY
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PCI Precast/Prestressed Concrete Institute

CONTENTS

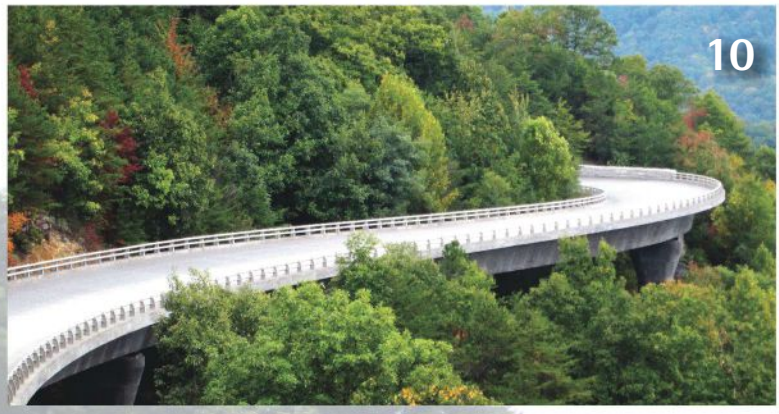

Photo: Eastern Federal Lands Highway Division of the Federal Highway Administration.

Photo: Beam, Longest and Neff LLC.

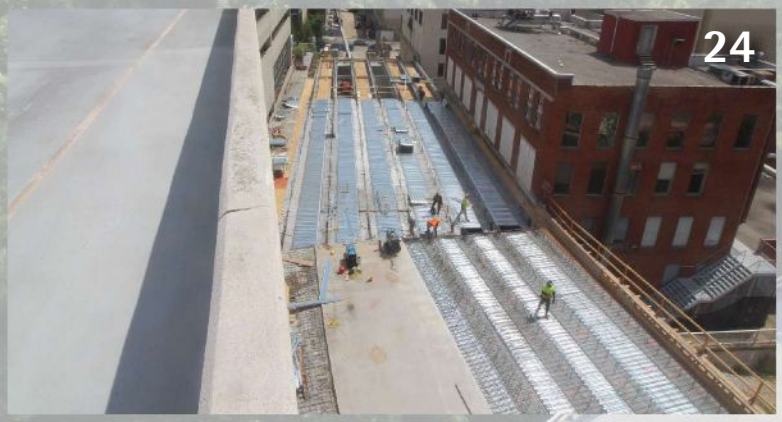

Photo: Georgia Department of Transportation.

Features

"A Shot in the Arm" 6
Armeni Consulting Services helps many clients address estimating and scheduling challenges as the array of delivery and construction methods grows.

Bridges of the Foothills Parkway 10

Dwight D. Eisenhower Veterans Memorial Bridge 14

Marc Basnight Bridge 18

Courtland Street Bridge 24

Departments

Editorial 2

Concrete Calendar 4

Aesthetics Commentary 20

CBT—Interative Substructure Design for the Marc Basnight Bridge Navigation Unit 28

CBT—Shasta Viaduct Arch Bridge: Improved by Pumping a Lightweight Concrete Solution 32

CBT—A Tool for Continuous On-Site Monitoring During Grouting of Post-Tensioning Tendons 36

CBT—Sweep in Precast, Prestressed Concrete Bridge Girders 38

CBT—Simplifying Bridge Deck Construction in Texas 44

Professor's Prospective 46

FHWA—A Synthesis Report on Concrete Bridge Shear Load Rating 48

State—Kentucky 50

Concrete Connections 54

CCC—Prestressed Concrete Girders Achieve Record Lengths 56

CCC—Fiber-Reinforced Polymer Reinforcement 58

Buyers' Guide 60

LRFD—Upcoming Changes to the AASHTO Bridge Specifications 62

Photo: Eastern Federal Lands Highway Division of the Federal Highway Administration.

Advertisers' Index

ALLPLAN Inc. 13

ASBI. 55

DYWIDAG-Systems International USA Inc. 59

FIGG. . . . . Inside Front Cover

Hamilton Form Company . . . . . Back Cover

HDR Inc. 31

Helser Industries. 3

Max USA Corp. 3

Modjeski and Masters Inc. . . . . Inside Back Cover

PCI 5, 22, 53

Poseidon Barge Ltd. 12

Stalite 23

Williams Form. 23

EDITORIAL

Photo: PCI.

A strong and bright vision of our future

William N. Nickas, *Editor-in-Chief*

In this editorial, let's peer into a kaleidoscope to see the future of versatile and resilient concrete bridges. That suggestion may seem whimsical, but let's imagine that all the choices we have for robust concrete construction are the pieces of glass that shift within the kaleidoscope (our industry). When viewed individually, these pieces may be attractive, but it is the visions they create when viewed together that are truly inspiring.

Recently, views within our kaleidoscope have been enhanced by adding new pieces to the mix, in the form of numerous technological advances, such as cement substitutes to densify concrete and improve the durability of bridges; new types of corrosion-resistant nonprestressed reinforcement and prestressing strand; higher strength strands (300 ksi); and larger strands (0.7 in. diameter). Strong evidence is helping us justify approaches such as migrating corrosion inhibitors for both old and new structures (see the Concrete Bridge Preservation article in the Winter 2019 issue of *ASPIRE*®). And the Federal Highway Administration is funding projects on the implementation of post-tensioning systems with electrically isolated tendons in the U.S. (see the Concrete Bridge Technology article in the Spring 2019 issue of *ASPIRE*).

As you acquaint yourself with organizational changes in Chapter 5 of the 8th edition of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*, notice the prominence of the strut-and-tie specifications. The changes demystify the use of an age-old tool, while using modern parameters. Let's embrace the mainstreaming of the strut-and-tie method without losing sight of durability advancements. The tube of the kaleidoscope may be a tunnel of sorts, but we must avoid tunnel vision—always look at the whole solution with multiple design features and limit state checks.

In 2019, we witnessed major milestones. Notably, record-breaking span lengths were constructed and some states used more concrete than ever before. This issue of *ASPIRE* features photos of the longest-known single-piece pretensioned, precast concrete girder (see the Creative Concrete Construction article on page 56). When I saw the pictures, I had to tip my hat to the team that created this "Oh my goodness moment." Meanwhile, in other states, shallow concrete spans are providing new efficiencies in bridge construction (see "Minnesota's MH Shape" in the Summer 2019 issue of *ASPIRE*).

As we develop thinner and longer beams, handling must be a bigger part of our design process. When all aspects of handling are designed into the solution, designers and builders gain an advantage. Eliminating overwater construction activities reduces labor costs, and these experiences teach us important lessons about connections and geometry. For example, I encourage you to consider the insights shared in the article on the construction of the Pensacola Bay Bridge in the November-December 2018 issue of *PCI Journal,* as well as lessons from"Calculations for Handling and Shipping Precast Concrete Deck Panels" in the Winter 2019 issue of *ASPIRE*.

In two years or so, when AASHTO ballots the first guide specification (materials, design, and construction) for bridge components built with ultra-high-performance concrete (UHPC), we can expect a renaissance for concrete bridges. Incremental changes in the concrete materials combined with reinforcement advancements have made high-performance concrete bridges the norm. In the next decade, UHPC specifications will provide the opportunity to again rethink the norm.

Let's start strategic discussions about using hybrid designs for concrete structures. In the November-December 2019 issue of the *PCI Journal*, there will be a story about long-span hybrid precast concrete bridge girders using UHPC and normalweight concrete. The authors share a critical look at many aspects of this innovative concept.

As we move forward, we must stay vigilant and verify that we're seeing the complete picture, not just snippets of data. Stay in touch with our industry partners as the colors of the kaleidoscope combine to create a strong and bright vision of our future.

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Concrete Technology Corporation (CTC) and the design-build team of Atkinson Construction and Jacobs have partnered in Tacoma, Wash., to produce the longest single-piece prestressed concrete girder made in the U.S. for the Washington State Department of Transportation (WSDOT). See page 56. Photo: CTC. Inset photo: WSDOT.

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Dr. Bruce W. Russell is director of the Bert Cooper Engineering Laboratory and an associate professor of civil and environmental engineering at Oklahoma State University. He has been a PCI member since 1990, and currently serves on the PCI Technical Activities Council and the PCI Bridges, Concrete Materials, and Fire Committees.

Dr. Henry Russell is an engineering consultant who has been involved with the applications of concrete in bridges for over 35 years and has published many papers on the applications of high-performance concrete.

CONCRETE CALENDAR 2019–2020

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

October 1–4, 2019
PTI Committee Days
Hilton Santa Fe Historic Plaza
Santa Fe, N.Mex.

October 20–24, 2019
ACI Fall 2019 Conference
Duke Energy Convention Center &
Hyatt Regency Cincinnati
Cincinnati, Ohio

November 4–6, 2019
ASBI 31st Annual Convention and Committee Meetings
Disney's Contemporary Resort and
Grand Floridian Resort
Lake Buena Vista, Fla.

November 13–15, 2019
Second International Conference on Transportation System Resilience to Natural Hazards and Extreme Weather
State Plaza Hotel
Washington, D.C.

November 20–22, 2019
PTI Level 1 & 2 Multistrand and Grouted PT Specialist Workshop
Austin, Tex.

November 23, 2019
PTI Level 1 & 2 Multistrand and Grouted PT Inspector Workshop
Austin, Tex.

December 11–13, 2019
International Accelerated Bridge Construction Conference
Hyatt Regency Miami
Miami, Fla.

January 12–16, 2020
Transportation Research Board Annual Meeting
Walter E. Washington Convention Center
Washington, D.C.

February 3–7, 2020
World of Concrete
Las Vegas Convention Center
Las Vegas, Nev.

March 3–7, 2020
PCI Convention
Fort Worth Convention Center
Fort Worth, Tex.

March 29–April 2, 2020
ACI Convention and Exposition
Hyatt Regency O'Hare
Rosemont, Ill.

April 6, 2020
ASBI Grouting Certification Training Course
J.J. Pickle Research Campus
Austin, Tex.

April 27–29, 2020
fib *Symposium on Concrete Structures for Resilient Society*
Shanghai, China

May 3–6, 2020
PTI 2020 Convention & Expo
Hilton Miami Downtown
Miami, Fla.

June 1–4, 2020
AASHTO Committee on Bridges and Structures Annual Meeting
Branson Convention Center
Branson, Mo.

June 8–11, 2020
International Bridge Conference
David L. Lawrence Convention Center
Pittsburgh, Pa.

June 22–25, 2020
Bridge Engineering Institute UHPC Symposium
National University of Singapore
Singapore

August 2–6, 2020
AASHTO Committee on Materials and Pavements Annual Meeting
Miami, Fla.

September 13–16, 2020
AREMA Annual Conference
Hilton Anatole
Dallas, Tex.

Errata

In the article, "Gilman Drive Overcrossing at the University of California San Diego," in the Summer 2019 issue of *ASPIRE*®, on page 20, right-hand column, the first full sentence should read, "The cable naturally adapts to the funicular shape as loads are applied. Under self- weight or other uniform gravity loads, it takes the shape of a catenary and all sections of the cable are in direct tension, with no shear or bending." We regret this error.

PCI Offers New Transportation eLearning Modules

Courses on Design and Fabrication of Precast, Prestressed Concrete Bridge Beams

The PCI eLearning Center is offering a new set of courses that will help experienced bridge designers become more proficient with advanced design methods for precast, prestressed concrete flexural members. There is no cost to enroll in and complete any of these new bridge courses.

The courses are based on the content of AASHTO LRFD and PCI publications. These include several State-of-the-Art and Recommended Practice publications, as well as the *PCI Bridge Design Manual*. These are available for free to course participants after registering with a valid email. While the courses are designed for an engineer with five or more years of experience, a less experienced engineer will find the content very helpful for understanding concepts and methodologies.

Where applicable, the material is presented as part of a "real world" example of a complete superstructure design so that students can see how actual calculations are completed according to the AASHTO LRFD specifications.

All courses on the PCI eLearning Center are completely FREE. **Go to: http://elearning.pci.org/**

PCI eLearning Precast, Prestressed Concrete Bridge Girder Series

Preliminary Precast, Prestressed Concrete Design (T110)

Materials and Manufacturing of Precast, Prestressed Concrete (T115)

Design Loads and Load Distribution (T120)

Flexure Service (T125)

Flexure Strength (T130)

Prestressed Losses (T135)*

Shear (T145)*

End Zone Design (T160)

Extending Spans (T310)

Bearing Pads (T450)*

In-Service Analysis Load Rating (T710)*

Full-Depth Precast Concrete Deck Panels Series

Introduction on Full-Depth Panel Precast Concrete Deck System and Its Advantages (T210)

Design and Detailing of Full-Depth Precast Concrete Deck Panels (T215)

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Case Studies and Emerging Developments of Full-Depth Precast Concrete Deck Panels (T225)

Lateral Stability of Precast, Prestressed Concrete Bridge Girders Series

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Stability of Transported Girders (T523)*

Seated Girders and Stability Issues from Bed to Bridge (T525)*

Stability Calculations and Sensitivity Analysis (T527)*

**These PCI eLearning Courses will be available soon.*

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AASHTO THE VOICE OF TRANSPORTATION

PCI Precast/Prestressed Concrete Institute

"A Shot in the Arm"

Armeni Consulting Services helps many clients address estimating and scheduling challenges as the array of delivery and construction methods grows.

by Craig A. Shutt

As the demands for bridge construction grow, and the array of available techniques, materials, and delivery methods expands, many in the industry look for help with pre- and postbid services, such as estimating and scheduling. Armeni Consulting Services has grown steadily since its founding in 2007 by filling that niche.

"Contractors aren't especially weak in these areas, but their staff can become stretched, and they can get into areas they aren't as familiar with," explains John Armeni, founder and president of the company based in Suwanee, Ga. "That can leave them feeling uncomfortable about their projections and calculations, so we provide an independent check on their work."

This type of consulting service is gaining importance as projects become more complicated, budgets tighten, and schedules compress. "Contractors don't want to over- or underestimate and get in trouble, so they want an experienced eye to review what they're planning. We're that 'shot in the arm' that lets them be confident going ahead."

The nature of the firm's work often leaves them in the background, which Armeni doesn't mind. "I like to say we're like BASF," he says, referring to the chemical manufacturer that ran ads in the early 1990s explaining they didn't make the products, they made the materials that made the products better. "We're behind the people on the frontlines who lead the work."

'We're behind the people on the frontlines who lead the work.'

Armeni Consulting Services helped develop cost estimates and constructability methods for the segmental concrete base being used by RUTE Foundation Systems for modular wind-turbine towers. The components are cast at a precasting plant and post-tensioned at the site. Photo: Armeni Consulting Services.

In some cases, partners on the projects don't realize Armeni's staff is involved until they arrive at a meeting. "We often don't get the credit, but that's fine with us. We're happy as long as the client's happy and they pay on time."

A Change in Direction

Armeni opened his consulting firm after his former company was bought out and he decided it was time to try something different. The original plan was to consult with local contractors in the Atlanta area, but he found they didn't need his help. So Armeni switched his focus.

As the past president of the American Segmental Bridge Institute (ASBI) and the chair of the publications committee for two editions of ASBI's *Construction Practices Handbook* (2005 and 2008), he was well known in the segmental bridge industry. "I had friends in ASBI and at departments of transportation, and I was surprised by the number of them who needed help with estimating and scheduling," he says. "I saw there was a need, so I focused there. It wasn't what I set out to do, but I found I enjoyed providing those services."

Armeni's segmental construction experience has made his firm especially popular for those types of projects, although the firm handles all types of designs. "Segmental designs are very useful for specific needs, and their use is growing," he says. "They offer solutions to a number of specific challenges."

For example, owners appreciate that segmental designs use concrete, which mean the structures are durable. "They require less maintenance over their lives. That's important when looking at costs, as those lower maintenance needs can overcome the potentially higher initial cost to make a concrete design the best option."

For Bridge 85851 in Winona, Minn., which opened in 2016, Armeni Consulting Services provided independent cost estimates. Photo: Armeni Consulting Services.

Recent Segmental Projects

In recent years, Armeni has consulted on a wide range of notable projects using segmental designs. In Portland, Ore., RUTE Foundation Systems is creating a precast concrete segmental system that reduces concrete use. The design is being used as the foundation for a modular wind-turbine tower, with the components cast in precasting plants and post-tensioned after erection. For this project, which won the 2019 Merit Award from the Post-Tensioning Institute, Armeni worked with the designer to develop detailed cost estimates and constructability methods. "Precast concrete segmental foundations that use bridge technology are new to the wind industry, and RUTE's patented technology allows wind-farm owners to reuse the foundation for multiple turbine repowering events," RUTE said.

'Precast concrete segmental foundations that use bridge technology are new to the wind industry.'

A more traditional segmental design project was Bridge 85851 in Winona, Minn., which opened in 2016. On this project, Armeni provided independent cost estimates. The goals for the project were to construct a new bridge parallel to an existing historic truss bridge, then rehabilitate the existing cantilever through-truss (the main three spans) and replace its approach spans. Challenges included an aggressive schedule as well as requirements to keep the river crossing open and preserve the original truss bridge's historic importance. The 2295-ft-long new structure comprises four spans of cast-in-place, post-tensioned concrete slab and five spans of 63-in.-deep precast, pretensioned concrete girders for the south approach. The structure transitions to a three-span, single-cell, segmental box-girder unit built using the balanced-cantilever method with form travelers, followed by four spans of 63-in.-deep precast, pretensioned concrete girders for the north approach. (Bridge 85851 is featured in an article in the Winter 2017 issue of *ASPIRE*®.)

Growing Demand for Scheduling Services

"The majority of our work has been in estimating, but we're finding that many clients also need help with scheduling," Armeni says. Providing this service to owners is an important component of the company's business. The firm provides critical-path method schedules, schedule monitoring and updating, and incorporation of resources and costs into schedules. Owners often need help handling revisions to the schedule as change orders arise. Armeni works to protect the owner's interests by reviewing the contractor's timeline, providing monthly updates on progress, and explaining details of changes.

'The majority of our work has been in estimating, but we're finding that many clients also need help with scheduling.'

"Owners like having [a consultant's] input, because it provides the contractor's perspective from an outside source," he explains. "Change orders and delays almost always occur, due to unknown obstructions or geotechnical obstacles. It's good for the owner to have people [other than the contractor]

A photo showing the nearly completed base of a wind-turbine tower. Photo: Armeni Consulting Services.

With no standard-bid cost history available for the Sixth Street Viaduct project in Los Angeles, Calif., Armeni helped with cost estimates as part of the CM/GC process. Rendering: Armeni Consulting Services.

who can help keep the game on a level playing field."

Clients don't always like what the consultants report, he notes. "We always tell them the truth and back that up. Change orders and claims can be contentious, and the client may not want to hear what we say about the validity of the delay and who's to blame."

Armeni has been involved in only a few contentious claims over the years, he adds. "We don't like being involved, because we often know people on both sides, and we don't want to have to take sides. We prefer working in partnership."

Scheduling complexities have been eased as software programs have become more sophisticated, he notes. About six years ago Armeni started to use TILOS linear scheduling software, which has been popular in Europe for some time.

"We're one of the few U.S. consultants using it, and we're introducing it to contractors on a regular basis," he says. "Most of our clients love it once they understand its benefits." It condenses the scheduling matrices into one page, with the bridge's length and stations along the x axis and the project timeline and structural components, such as pilings, footings, columns, caps, and superstructure, along the y axis.

"Some [clients] are resistant, but once they see how the software simplifies the key points, especially on complex segmental or cable-stayed bridges, they embrace it. They don't want to have to leaf through stacks of output if they can see the entire project in a nutshell on one page."

New Delivery Methods

Demand for Armeni's expertise is growing as delivery method options expand and more departments of transportation encourage the use of new techniques. "We're starting to pick up more construction manager/general contractor (CM/GC) projects," Armeni says. "Owners are seeing the need for independent cost estimates. It's a definite trend."

As it did on the 85851 Bridge project, Armeni Consulting provided independent cost estimates for the Sixth Street Viaduct, a prominent infrastructure project in Los Angeles, Calif. Planned for completion in 2022, the structure will replace a 1932 bridge spanning the Los Angeles River. The structure features 10 pairs of illuminated concrete arches, which create a dramatic nighttime effect along its 3500-ft length.

"The Sixth Street Viaduct project is unique and challenging to construct—and it was equally challenging to estimate," Armeni says. "There was no standard-bid cost history for a structure like this." The firm worked with engineering firm HNTB to create an engineer's estimate, which was developed in parallel with the independent cost estimate and the Skanska/Stacy and Witbeck (SSW) joint-venture cost estimate as part of the CM/GC process. For each construction package, estimates were compiled at the 30%-, 65%-, and 90%-complete stages, and for a guaranteed maximum price. "This gave city officials more confidence in SSW's budget costs and also helped stakeholders make design decisions."

Erection of precast concrete box-girder segment on the Cline Avenue Bridge in East Chicago, Ind. Armeni helped the design-build firm find the most cost-effective techniques for the project. Photo: FIGG Engineers.

Design-Build and Public-Private Partnership Opportunities

The firm also has been involved in more design-build projects and public-private

partnerships (P3s) as clients look for new ways to shorten project timelines and control budgets. "The trend toward more design-build projects began in the mid-1990s and has gained momentum ever since," Armeni says. "It's really taken off today."

Armeni participated in his first design-build project in 1995 and has completed a large number since then. One of the most recent success stories is the Cline Avenue Bridge replacement project in East Chicago, Ind. The new, 6236-ft-long elevated expressway crosses the Indiana Harbor and Ship Canal. The segmental superstructure consists of sustainable, post-tensioned concrete single-cell box girders designed with low-maintenance features to achieve a service life of more than 150 years.

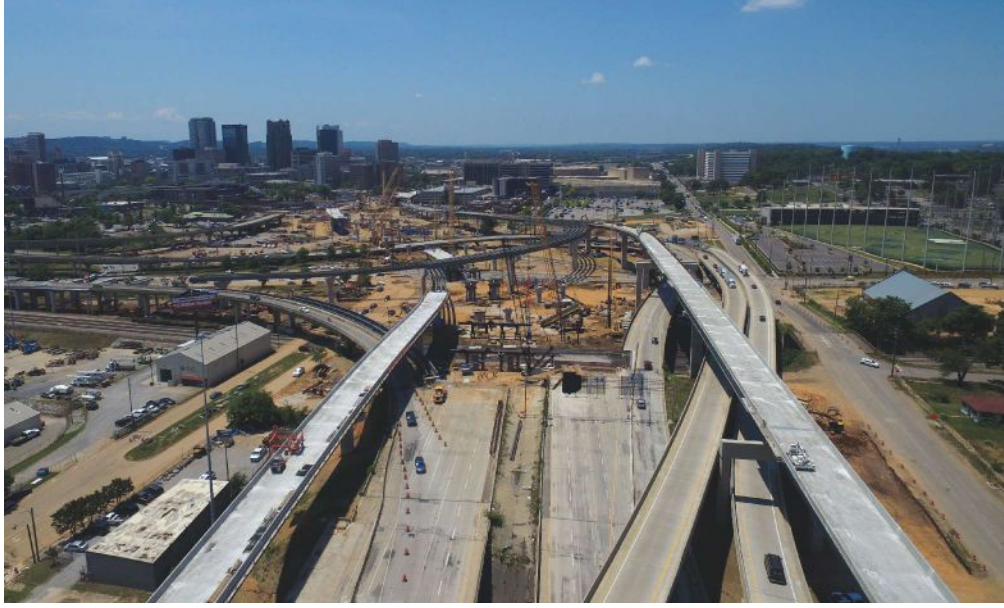
On this project, Armeni Consulting Services worked closely with bridge design-build contractor FIGG Bridge Builders from the earliest stages to find the most cost-effective strategy to resolve a variety of obstacles. The firm helped maintain cost estimates and construction schedules throughout construction.

"Design-build projects play to our strengths," Armeni says. "As a national company, we see many design ideas that states use and can help adapt them to new locations." Some concepts don't translate to new projects due to variations in soil conditions and other circumstances, he notes. "Often, the first thing a client asks is if we've seen anything that could help reduce costs or time. We present concepts we've seen that might work. They often lean on us to offer suggestions."

'Design-build projects play to our strengths.'

The firm's involvement in P3 projects has also grown. "From our perspective, P3s aren't really that much different from design-build, except for the financing aspect—and we stay out of that completely," Armeni says.

Design-build projects typically are more challenging than design-bid-build jobs, he adds, but that's what is appealing about them. "Design-bid-build projects have everything laid out, so they're not as interesting and we aren't as helpful to our clients."



On the Interstate 59/20 Central Business District Bridge replacement project in Birmingham, Ala., Armeni consulted on cost estimates, schedules, and options for construction methods. Photo: Armeni Consulting Services.

Still, Armeni does consult on design-bid-build efforts, such as the Interstate 59/20 Central Business District Bridge project in Birmingham, Ala. The state's busiest highway and its 36 bridges were replaced to handle vehicle demand that had tripled since the highway's construction in the 1960s. Armeni developed estimates, schedules, and construction means and methods for both pre-and postbid operations for the Alabama Department of Transportation. The firm also reviewed the contractors' monthly schedule submittals. Notably, a variety of segmental precast concrete bridges were cast offsite and delivered while other work was progressing at the site, which helped project stakeholders achieve their schedule goals. (For more details of the project, see the article about Alabama in the Spring 2019 issue of *ASPIRE*.)

What's Ahead

Armeni sees a strong future for concrete bridges, especially if tariffs drive up the cost of steel. However, the cost of reinforcing steel also will be affected by changes in international trade and other economic factors, he notes. "It's important for engineers to keep abreast of trends affecting reinforcing bar and materials such as galvanized steel, epoxy-coated, and stainless steel. The desire for 75- to 100-year service lives continues to grow, and every option needs to be examined."

The firm's own future looks bright. "Owners will continue to need help with estimating, as they can't award bridges that are overbudget to begin with." Requests for proposals often require independent estimates, which further increases the demand for the company's services.

"Every project is unique and can't use averaged estimates or schedules, and prices can be all over the place. Standard inflation increases aren't accurate enough."

Going forward, Armeni intends to retain the firm's independence. "We've had chances to get bigger and have had people look at us," he says. "But we like what we do and what we've got here. We're a small firm, but we do big work."

Armeni's History of Help

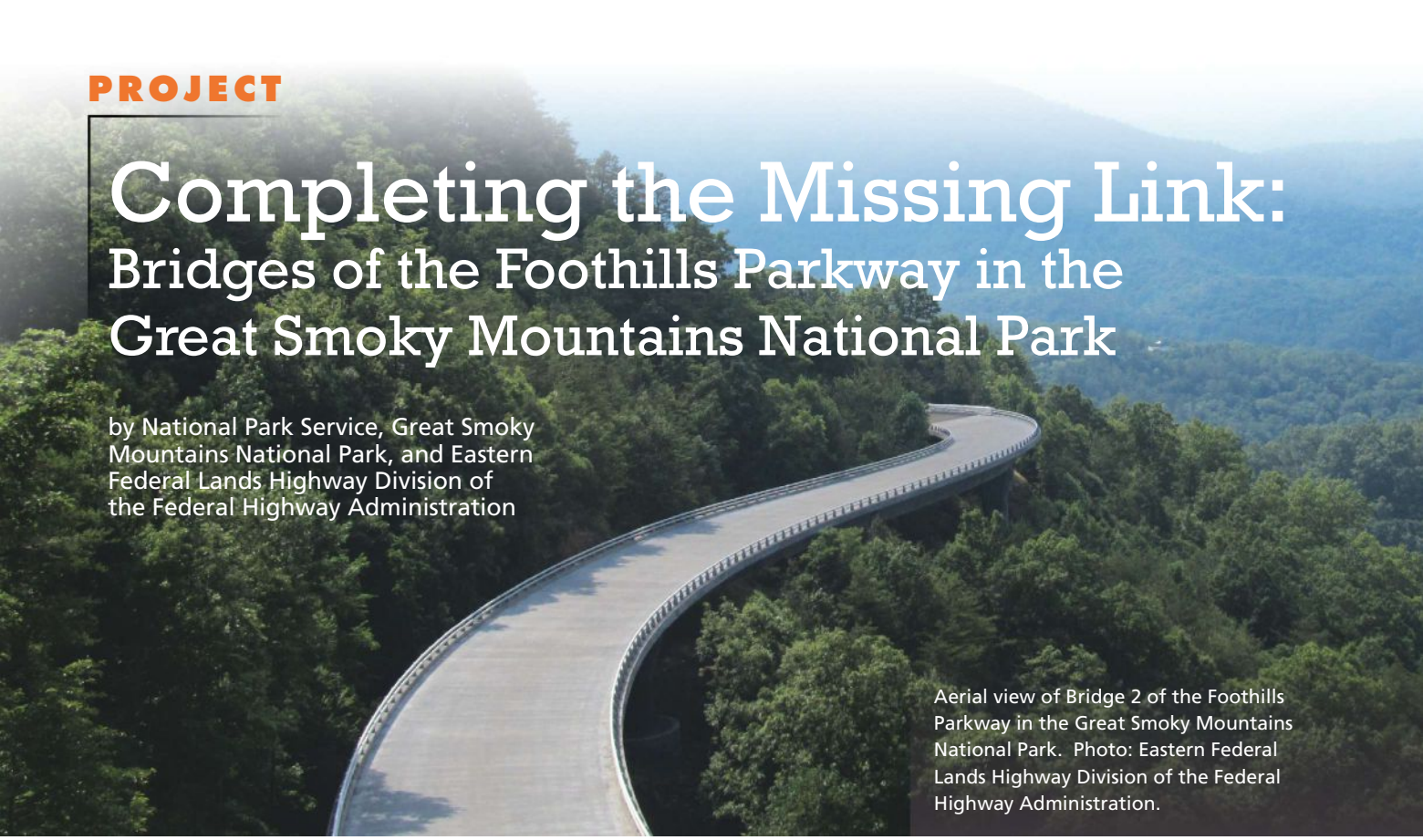
Armeni Consulting Services was founded in 2007 and quickly began helping owners and contractors with scheduling and estimates, including updates throughout the construction process. John Armeni's resume includes more than 25 years of experience with heavy civil/bridge engineering, 18 years of onsite management experience, and over 10 years of estimating experience.

Armeni Consultanting Services currently employs nine people, with most working from Suwanee, Ga., and one based in Gates, N.C.

PROJECT

Completing the Missing Link:
Bridges of the Foothills Parkway in the Great Smoky Mountains National Park

by National Park Service, Great Smoky Mountains National Park, and Eastern Federal Lands Highway Division of the Federal Highway Administration



Aerial view of Bridge 2 of the Foothills Parkway in the Great Smoky Mountains National Park. Photo: Eastern Federal Lands Highway Division of the Federal Highway Administration.

Winding along the northern side of the Great Smoky Mountains National Park is the Foothills Parkway. Authorized by Congress in 1944, the parkway was originally envisioned as a 72-mile scenic route with breathtaking views of the park. Currently, 37.2 miles are open to public traffic, including Sections E and F, which are 15 miles long. Within those sections is a 1.6 mile stretch of particularly challenging terrain. Because this part of the roadway was left incomplete after the last period of active construction in the 1980s, it became known as the "Missing Link."

The Missing Link passage includes nine bridges constructed on very steep terrain that cross ravines and environmentally sensitive areas. This construction project represents a major milestone toward completion of the Foothills Parkway from Wears Valley to Walland, Tenn. The Eastern Federal Lands Highway Division (EFLHD) of the Federal Highway Administration (FHWA) worked in partnership with the National Park Service to administer the design and construction of this section of the parkway.

Site Challenges

The nine bridges, numbered 2 through 10, in the Missing Link section include multispan prestressed concrete beam bridges as well as more complex, cast-in-place (CIP) post-tensioned box-girder bridges, and CIP and precast concrete segmental bridges. Because of the mountainous terrain, all the bridges have challenging horizontal and vertical alignments, which include horizontal curves as tight as 650 ft in radius, reverse curves, and transition spirals.

The project sites were narrow. Disturbance had to be limited as much as possible to the footprint of the parkway and could not extend past the drip lines of the bridges. On some of the steepest terrains, access was very limited and top-down segmental bridge construction was implemented. The use of design visualization, prepared by EFLHD, played a key role in the planning necessary to ensure that the Missing Link bridges were constructed using the most environmentally sensitive methods available.

Bridge 2 has a horizontal alignment composed of a reverse curve with transition spirals and a vertical alignment with a sag curve. These features contributed to the complexity of bridge construction. Photo: Eastern Federal Lands Highway Division of the Federal Highway Administration.

Pier and superstructure segments for Bridge 2 in the precast concrete yard. Photo: Eastern Federal Lands Highway Division of the Federal Highway Administration.

Bridges 2, 9, and 10

Bridges 2, 9, and 10 are noteworthy segmental, single-cell box-girder bridges built using top-down construction.

On the east end of the Missing Link, Bridge 2 has a length of 790 ft with five spans (125 ft, 180 ft, 180 ft, 180 ft, 125 ft) (see the article in the Fall 2012 issue of *ASPIRE*®). The geometry of this bridge includes a horizontal alignment with a reverse curve and transition spirals and a vertical alignment with an 8% change in grade sag curve. Ninety-two precast concrete segments were used for the superstructure and twenty for the piers. The foundations are supported on micropiles that vary in length due to varying geotechnical conditions.

The typical superstructure segment is a single-cell box girder with a constant depth of 9 ft 0 in. and a width of 36 ft 10 in.; each of these segments weighs 50 tons. The top slab is transversely post-tensioned. The substructure segments have widths from 10 ft 0 in. to 13 ft 8 in., are hollow, and are post-tensioned.

The specifications for the structure, prepared by EFLHD, required that the segments be cast at a Precast/Prestressed Concrete Institute (PCI)–certified facility to ensure quality fabrication of the segments.

Located in steep terrain, Bridge 2 was accessible from only one end during construction. Site constraints prevented the contractor from erecting the precast concrete segments with traditional cranes or overhead erection trusses. A temporary trestle/falsework system was used to support the segments and the elevated rail system for the gantry crane that was used to erect them.

On the west end of the Missing Link, Bridges 9 and 10 each have lengths of approximately 475 ft. Each of these bridges has three spans (approximately 138 ft, 194 ft, 138 ft). These bridges are also on reverse-curved horizontal alignments with spirals. The vertical alignment includes a crest curve with a 9.6% change in grade.

Bridges 9 and 10 were constructed using CIP segmental, balanced-

Due to the steep terrain at the site for Bridge 2, a temporary trestle/falsework system was constructed to support a gantry crane to erect the precast concrete segments. The gantry is lifting a pier segment in this photo. The access hole in the segment is visible. Photo: Eastern Federal Lands Highway Division of the Federal Highway Administration.

Segment erection as Bridge 2 nears completion. Photo: Great Smoky Mountains National Park, National Park Service.

Aerial view of the "Missing Link." Bridge 2 is visible in the center. Photo: Eastern Federal Lands Highway Division of the Federal Highway Administration.

cantilever techniques. Pier segments were used for both the piers and superstructure. The foundations are supported on spread footings. The superstructures consist of 13 segments each. The typical superstructure segment is a single-cell box girder with a constant depth of 9 ft 0 in. and a width of 36 ft 10 in. The top slabs are transversely post-tensioned.

Bridges 3 Through 7

Bridges 3 through 7, the last five structures constructed in the Missing Link project, are cast-in-place segmental box-girder bridges. The cross-sectional geometries of these bridges are like those of Bridge 2.

Bridges 4 and 5 are multispan structures, and Bridges 6 and 7 have single spans. Even the single-span bridges cross steep terrains and ravines and accommodate a variety of curves and spirals. The foundations of these bridges are supported with spread footings and micropiles. Multispan, prestressed concrete AASHTO-type girder structures were used for the parkway bridges that did not require spanning over steep terrains.

Aesthetics and Maintenance

All nine bridges in the Missing Link blend with their local surroundings. Aesthetic design elements include integrally colored concrete on the superstructures; integrally colored, fiber-reinforced high-performance concrete overlays; stone faces on the substructures; piers that blend into the superstructures; and aluminum bridge rails.

Additionally, the structures have been designed to anticipate bridge inspection and maintenance needs. For example, access doors are large enough to accommodate large equipment inside the single-cell segmental bridges, and abutments have openings to provide for proper ventilation during inspection. The bridges are designed to require minimal maintenance with a design life of 75 to 100 years.

Next Steps

Building on prior planning efforts and environmental studies for the Foothills Parkway, the NPS and FHWA propose to reinitiate the National Environmental Policy Act planning process for the next section of the parkway, the 9.8-mile-long Section 8D. Section 8D is the next logical section of the Foothills Parkway to construct because it will connect Section 8E in Wears Valley to the Gatlinburg Spur (U.S. Route 441) in Pigeon Forge, Tenn. This will create a new connection to the northern boundary of the Great Smoky Mountains National Park.

EDITOR'S NOTE

Bridge 2 received a 2013 ASBI Award of Excellence and a 2014 PCI Design Award in the category Best Bridges with Main Span More than 150 ft.

DWIGHT D. EISENHOWER VETERANS MEMORIAL BRIDGE

REPLACING AN URBAN GATEWAY IN INDIANA

by Jeffery Parke, Beam, Longest and Neff LLC

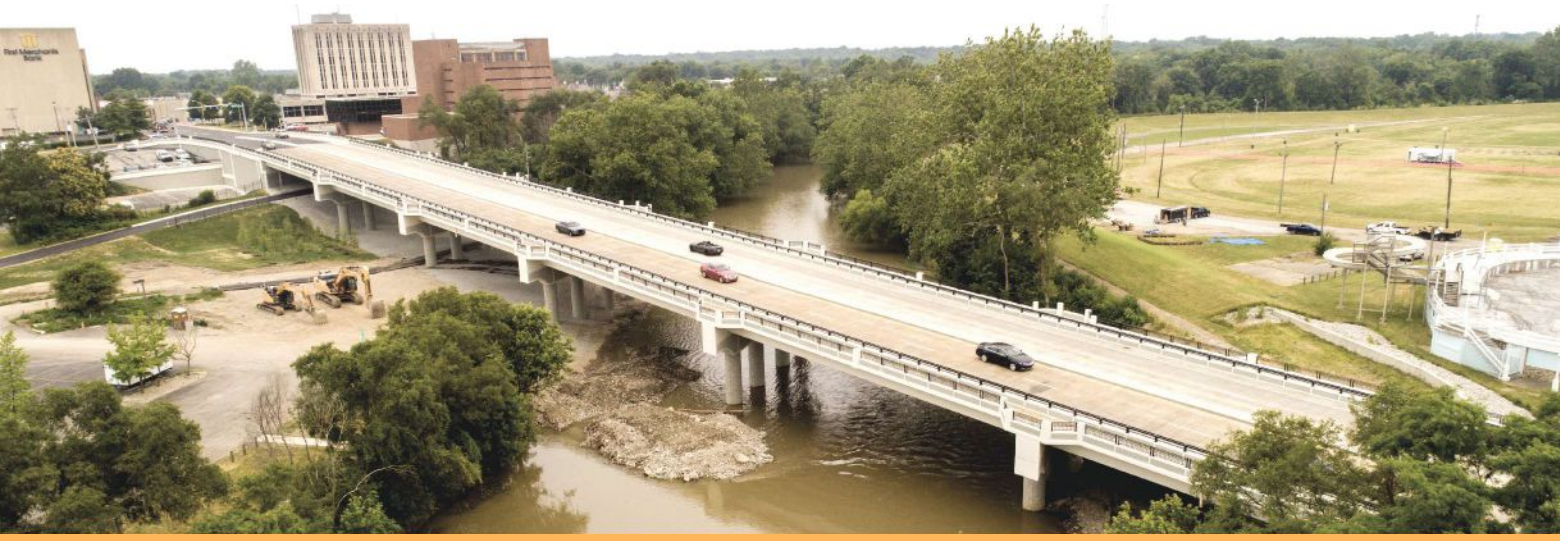
The Dwight D. Eisenhower Veterans Memorial Bridge in Anderson, Ind., carries Eighth Street over the White River. The most heavily traveled bridge in Madison County, it serves as the eastern gateway into downtown Anderson.

When the orginal Eisenhower bridge was built in the late 1960s, complex site conditions, including a railroad and utility building, led to the selection of a design with multiple skews, span lengths, beam depths, and substructure types. Unfortunately, some of these construction details eventually contributed to deterioration of structural elements at critical locations. In 2009, when the original bridge was about 40 years old, the owner, Madison County, initiated an alternatives analysis to determine the best course of action. The study found that the most cost-effective solution for the community was bridge replacement. Soon after, project development for the replacement bridge was initiated, with funding for construction secured in 2016 through the Transportation Investment Generating Economic Recovery grant program.

Project Specifications

The owner requested that the existing 729-ft-long, 10-span, continuous prestressed concrete I-beam bridge be replaced by a structure with an optimized span configuration; the owner also wanted to keep the existing cross-section geometry in a design that would be low maintenance and aesthetically pleasing. Additionally, permanent right-of-way acquisition was to be minimized while temporary right-of-way could be acquired for causeways used for beam placement or other construction activities.

The new Dwight D. Eisenhower Veterans Memorial Bridge over the White River. Debris walls for piers in the river were eliminated due to the lack of debris flow at this location. Photo: Beam, Longest and Neff LLC.



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DWIGHT D. EISENHOWER VETERANS MEMORIAL BRIDGE / ANDERSON, INDIANA

BRIDGE DESIGN ENGINEER: Beam, Longest and Neff LLC, Indianapolis, Ind.

PRIME CONTRACTOR: Primco Inc., Fort Wayne, Ind.

PRECASTER: Prestress Services Industries LLC, Decatur, Ind.—a PCI-certified producer

OTHER MATERIAL SUPPLIERS: Precast concrete panels and mechanically stabilized earth walls: SINE WALL, Wake Forest, N.C.

Epoxy-coated reinforcement was used in the 8-in.-thick deck. Photo: United Consulting.

The selected six-span bridge design was approximately 695-ft-long, with 114 ft 4 in. end spans and 115 ft 8 in. spans for the remainder of the bridge; all spans were constructed on a 30-degree skew, including the approaches and piers.

The designers eliminated the existing 34-ft-long span 1 on the west end of the bridge to lower the grade of Eighth Street by approximately 6 ft and shorten the bridge. Uniform span lengths were obtained by removing the final impediment, an abandoned utility building.

Much of the existing roadway geometry was maintained, except that on both sides an interior bridge railing was added to separate pedestrians from traffic and the sidewalks were widened to 6 ft. The selected cross section consisted of four 11-ft-wide travel lanes (two in each direction), 5-ft-wide outside shoulders, and 6-ft-wide flush sidewalks. A 4-ft-wide raised concrete median with 1-ft-wide flush offsets separates the eastbound and westbound traffic for an out-to-out coping width of 76 ft 8 in. Scenic overlooks were added at each pier, and ornamental streetlights were spaced along each coping.

The designers evaluated which superstructure materials and details would reduce future maintenance costs. Structural steel was eliminated because painted steel would require maintenance, and staining is associated with weathering steel. Traditional prestressed concrete I-beams and bulb-tee beams were eliminated because of the number of beams that would be required. However, 48-in.-deep, high-performance, high-strength (HPHS) prestressed concrete U-beams could be spaced at 13 ft and were selected because they would result in six beam lines and require minimal maintenance. The HPHS concrete allowed the spans and beam spacings to be increased, reducing project costs. For the precast, prestressed concrete U-beams, the HPHS concrete mixture proportions specified a water–cementitious materials ratio of 0.34 and silica fume to increase durability. The specified compressive strengths for the U-beams were 7000 psi at transfer and 9000 psi at 28 days.

Stay-in-place metal deck forms were used to form the 8-in-thick cast-in-place concrete deck, which used epoxy-coated reinforcement. Epoxy-coated reinforcement was also used in the barrier rails, moment slabs, approach slabs, and semi-integral abutments. Access hatches were placed at each end of every beam on the structure. The designer used semi-integral abutments and located the expansion

MADISON COUNTY, INDIANA, OWNER

BRIDGE DESCRIPTION: Six-span, 695-ft-long, continuous, prestressed concrete U-beam bridge with 114 ft 4 in. end spans and four 115 ft 8 in. interior spans, constructed on a 30-degree skew

STRUCTURAL COMPONENTS: Thirty-six 48-in.-deep prestressed concrete U-beams with stay-in-place metal deck forms and an 8-in.-thick cast-in-place concrete deck; cast-in-place concrete semi-integral end bents, pier caps, and columns founded on cast-in-place concrete foundations supported by steel H-piles

BRIDGE CONSTRUCTION COST: $13.8 million total; $10.1 million ($190/ft^2) for the bridge

Pier cap and footing construction. Photo: United Consulting.

joints at the ends of the reinforced concrete approach slabs because the 2009 analysis found that expansion joints located in the existing bridge had directly led to significant deterioration of the prestressed concrete beams and substructure. Cast-in-place concrete joint abutments were placed between the approach pavement and the approach slabs to support the expansion joints.

Minimalistic pier geometry was selected to reduce thermal load effects while also accommodating phased-construction requirements. The owner specified that the existing bridge could not be closed completely, so the design team developed a maintenance-of-traffic plan that reduced the existing eastbound traffic lanes to one across the existing bridge and through the project limits while detouring the westbound lanes around the construction site during phase 1 construction. Upon completion of phase 1, the eastbound traffic lane was diverted onto the newly constructed westbound portion of the structure while the westbound traffic continued to follow the established detour.

The pier consisted of a hammerhead cap on four 5-ft-diameter columns. The pier cap was designed using the strut-and-tie methodology, and an open joint was placed in the middle of the cap to reduce transverse thermal loads. Each of the columns was founded on a 4-ft-thick cast-in-place pile cap supported by steel H-piles.

Existing dissimilar span lengths between the westbound and eastbound lanes made it difficult to locate the substructure without conflicting with existing piling. The span arrangement was selected to minimize these conflicts; however, where conflicts did arise, specifications directed the contractor to remove the existing piles and backfill the remaining hole. All conflicting piles were successfully removed during construction.

Project Aesthetics

Because the bridge is the eastern gateway into Anderson, the owner stipulated that the bridge must have an appropriate, consistent aesthetic quality. The owner selected a concrete formliner to be used for all precast concrete wall panels and cast-in-place concrete pier columns. A different formliner was used for the cast-in-place concrete pier caps and bridge railing. A two-color (white/gray) palette was selected to be applied to all exposed concrete surfaces, including the U-beams, while metal portions of the bridge railing were painted black. Finally, black ornamental streetlights were selected to match similar units located throughout the city.

Project Challenges

Because of the project's location, numerous project challenges had to be overcome. The west end of the existing bridge was supported on a

On the west end of the structure, precast concrete panels were anchored to the existing retaining walls. The narrow 2 ft 6 in. gap between them was filled with lightweight cellular concrete fill to limit the additional load on the existing foundation. Photo: Beam, Longest and Neff LLC.

Prestressed concrete U-beams being erected at abutment number 1. The crane is sitting on crane pads within the mechanically stabilized earth fill. Photo: United Consulting.

full-faced concrete abutment with long, tall concrete retaining walls supporting Eighth Street. Given the limited right-of-way, precast concrete panels were anchored to the existing retaining walls, with a narrow 2 ft 6 in. gap between them. The gap was filled with lightweight cellular concrete fill to limit the additional load on the existing foundation. Railing and moment (overhanging) slabs were constructed on top of the wall panels without affecting the stability of the existing wall. The eliminated span was filled in and reconstructed using mechanically stabilized earth walls to support the abutment.

The Madison County Jail is located at the northwest quadrant of the project. Vibration monitoring was required during pile driving to decrease the likelihood of damage to the jail. Reports were prepared before and after the pile driving to document the condition of the facility. Pile driving was successfully completed without structural damage to the facility.

The numerous utilities located within the existing bridge and surrounding site required relocation. Fiber-optic lines were located on the bridge, and electrical lines were located underneath it. Also, 12-in.- and 24-in.-diameter water mains crossed under the bridge, river, and levee. The most significant utility relocation was the 24-in.-diameter water main, which was directionally bored beneath the river and levee. Coordination with the U.S. Army Corp of Engineers was required to develop construction details and specifications that would not endanger the stability of the levee during and after construction.

Relocating the thousands of copper and fiber-optic telecommunications lines located in both sidewalks of the existing bridge required significant coordination with the local utility. Relocation of all affected utilities was completed without delay to the project.

Finally, the existing bridge's flat and angular spaces were riddled with bird nests. These nests posed a health and safety hazard to bridge inspectors and residents who used the paths beneath the bridge. For the new bridge, prestressed concrete U-beams were selected in part because they do not have exposed bottom flanges or exterior diaphragms where birds can roost. Pier continuity diaphragms were detailed to the full pier cap width, further eliminating ledges. These design elements discourage birds nesting on the bridge.

Conclusion

From the in-depth alternative analysis through construction, clear and consistent communication among stakeholders was the key to success. The completed bridge, which was opened on time on June 18, 2019, will provide a safe gateway into downtown Anderson for years to come.

Jeffery Parke is a senior bridge engineer for Beam, Longest and Neff LLC, in Indianapolis, Ind.

Completed pier with architectural formliner. The open joint in the middle of the pier cap to reduce transverse thermal loads is barely visible. The clean lines beneath the bridge limit areas for birds to roost. Photo: Beam, Longest and Neff LLC.

Economy and Durability Achieved through Precast Concrete—the New Marc Basnight Bridge

by Domenic Coletti, R. Dominick Amico, Nicholas Burdette, and Mohit Garg, HDR

The new $252 million, 2.8-mile-long Marc Basnight Bridge carries traffic for North Carolina Highway 12, a hurricane evacuation route. The structure spans the remote Oregon Inlet of North Carolina's Outer Banks. The inlet is considered by some to be the most dangerous on the East Coast due to treacherous currents, constantly shifting bathymetry, and the potential for violent storms.

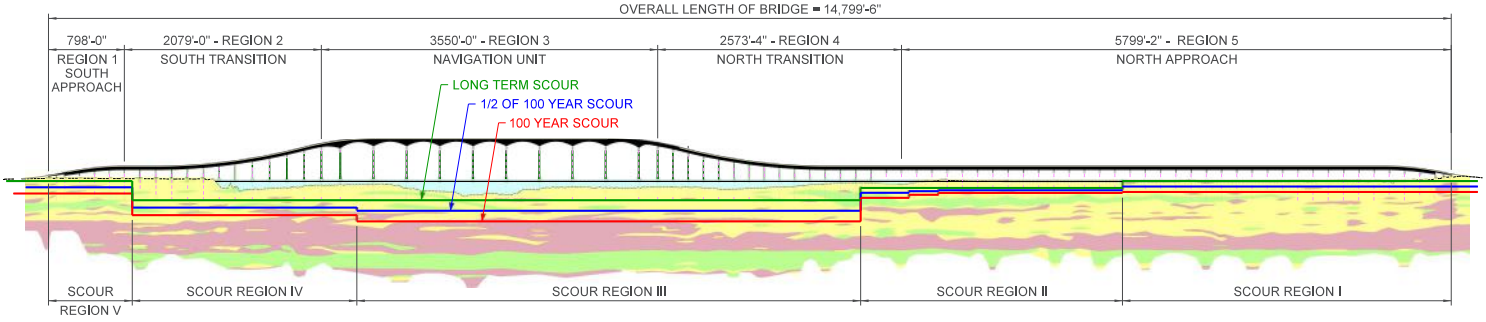
The new bridge replaces the Herbert C. Bonner Bridge, which had been designed for a 30-year service life and was a source of consternation for more than 55 years, requiring nearly continuous repairs after its completion in 1963. The Basnight Bridge is designed for a 100-year service life and to resist up to 84 ft of scour, 105 mph winds, and vessel collision forces.

Early in the design process, structural and geotechnical engineering teams for the Basnight Bridge created a color-coded longitudinal plot of the project illustrating the subsurface conditions, scour profile, vessel collision zones, and navigation clearance zones. Examination of this plot allowed the project to be partitioned into five regions: north approach spans, north transition spans, navigation unit, south transition spans, and south approach spans. These regions corresponded to both the scour profile and the vertical geometry of the bridge. Vessel collision forces, a primary loading consideration for the deep foundations of the bridge, also varied along the length of the bridge, with smaller forces in the north and south approach spans, larger forces in the north and south transition spans, and the largest forces in the navigation unit.

The vessel collision forces varied along the length of the bridge for a number of reasons. First, the vessel collision design vessel, the dredge Atchafalya, was only considered as operating within the navigation zone; in other regions of the bridge, only the American Association of State Highway and Transportation Officials (AASHTO) minimum impact vessel (an empty hopper barge) was considered.

Also, the current (velocities) varied along the length of the bridge; the design

Longitudinal profile of the Marc Basnight Bridge, showing subsurface conditions, scour depths, structure height, and design regions. All Photos and Figures: HDR.



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MARC BASNIGHT BRIDGE / DARE COUNTY, NORTH CAROLINA

BRIDGE DESIGN ENGINEER: HDR, Raleigh, N.C.

CONSTRUCTION ENGINEERING: Corven Engineering Inc., Tallahassee, Fla.

PRIME CONTRACTOR: PCL Civil Constructors, Raleigh, N.C.

PRECASTER: Coastal Precast Systems LLC, Chesapeake, Va.—a PCI-certified producer

POST-TENSIONING CONTRACTOR: Schwager Davis Inc., San Jose, Calif.

OTHER SUPPLIERS: Segmental formwork: Ninive Casseforme, Garbagnate Monastero, Italy; segment lifters: HCR Bridge Machinery, Kuala Lumpur, Malaysia

Simple precast concrete elements, such as these Florida I-beam girders and pile bents, were combined to accommodate a curved horizontal alignment in the south approach spans.

collision velocities were generally faster (with larger vessel collision forces) in the main part of Oregon Inlet and slower (with smaller vessel collision forces) near the ends of the bridge.

Advantages of Precast Concrete

Precast concrete elements were chosen as the optimum design solution for several reasons. First, each of the structure's regions lent itself to a tailored design approach, allowing for widespread use and detailing of repetitive elements in different configurations. Also, concrete resists corrosion from the saltwater environment, making it a durable material that extends the structure's service life and reduces maintenance.

Furthermore, broad use of prefabricated concrete elements and modular construction techniques was a practical and economical option given the project's remote location. Precasting Florida I-beam (FIB) girders, box-girder segments, bent caps, columns, and piles under controlled conditions at an off-site precast concrete facility resulted in the production of high-quality, extremely durable concrete components. These levels of quality and durability would have been difficult to achieve if the concrete had been cast in the harsh marine environment of the Oregon Inlet. Off-site fabrication was also less expensive than delivering and placing cast-in-place (CIP) concrete at the remote project site.

Additionally, because the use of precast concrete elements minimized field construction work from barges and work trestles, construction was faster and safer and had less impact on the environment than other options. It is estimated that 24 working months were saved because of the use of precast concrete.

Substructures and Foundations

The north and south approach spans represent approximately half of the total bridge length. Foundations in these regions consist of precast concrete pile bents, with three or four vertical 54-in.-diameter precast concrete cylinder piles for each bent. The cylinder piles connect directly to innovative precast concrete bent caps via reinforced CIP concrete infill. This infill extends 30 ft into the hollow piles to provide stiffness in cases of severe scour, transfers both moment and axial loads to each pile, and provides strength locally to prevent vehicle-collision damage to the pile wall.

The north and south transition spans represent approximately one-quarter of the total bridge length. Foundations in the transition regions include six to sixteen battered 36-in.-square precast concrete piles with a CIP waterline pile cap. The bents for the transition regions are match-cast, precast concrete solid 5-ft or 6-ft square, post-tensioned columns in a two-column bent arrangement with column heights up to 50 ft, supporting a precast concrete bent cap.

The 11-span navigation unit extends 3550 ft and includes 12 substructure bents. Foundations in this region include 18 to 30 battered 36-in.-square precast concrete piles, with square and octagonal CIP waterline pile caps. On top of the pile cap, a precast, post-tensioned, match-cast 16 ft by 11 ft hollow-box concrete column supports a rectangular precast concrete column cap.

NORTH CAROLINA DEPARTMENT OF TRANSPORTATION, OWNER

BRIDGE DESCRIPTION: A 14,800-ft-long bridge consisting of 71 spans of precast, prestressed Florida I-beam (FIB) girders (span lengths up to 182 ft) and 11 spans of variable-depth, post-tensioned segmental concrete box girders (span lengths up to 350 ft)

STRUCTURAL COMPONENTS: Two-hundred eighty-five 96-in.-deep FIB girders (46,379 ft total) and eighteen 45-in.-deep FIB girders (1508 ft total), both with 9-in.-thick sand-lightweight concrete decks; 238 single-cell precast concrete variable-depth (9 ft 0 in. to 19 ft 0 in.) segments in a 3550-ft-long continuous, post-tensioned box-girder unit; 44 pile bents with precast concrete bent caps and precast, prestressed concrete 54-in.-diameter cylinder piles; 25 two-column bents and 12 single-column bents with precast concrete caps supported on CIP pile caps and precast, prestressed concrete 36-in.-square piles

BRIDGE CONSTRUCTION COST: $252 million (total project cost), approximately $350/ft^2

AWARDS: American Road and Transportation Builders Globe Award in the >$100 Million projects category; Deep Foundations Institute Outstanding Project Award

Lateral resistance was a major element of the foundation designs. Multiple refined soil-structure interaction analysis models were performed for every bent on the bridge, considering pier stiffnesses, vehicle collisions, and various scour depths from no-scour to full-scour conditions. Design of the navigation unit included individual FB-MultiPier software bent models and a global three-dimensional LARSA model of the entire 3550-ft unit with superstructure and substructure. (The design process used for the piers is described in a related Concrete Bridge Technology article in this issue of *ASPIRE*®.)

Superstructure

The superstructure of most of the bridge (the north and south approach spans and the north and south transition spans) consists of a conventionally formed 9-in.-thick CIP sand-lightweight concrete (sand-LWC) deck supported by precast, prestressed concrete FIB girders. Stainless steel reinforcement is used in the deck to enhance corrosion resistance. The deck of the FIB spans was constructed using sand-LWC to reduce dead load on the girders, allowing the girders to span longer distances at the same girder spacing. This reduced both the total number of required spans and, more importantly, the total number of bents and foundations, thereby lowering costs substantially. The sand-LWC deck was not treated differently with regard to durability or corrosion protection; the North Carolina Department of Transportation permitted the use of sand-LWC in the deck without additional corrosion protection provisions.

Most of the bridge has a roadway cross section with two 12-ft-wide lanes and two 8-ft-wide shoulders, with a total out-to-out deck width of 42 ft 7 in. The most common span configuration features a four-girder cross section and a span length of 160 ft 10 in. In the south transition region, a number of spans feature a six-girder cross section and spans up to 182 ft, the longest simple-span prestressed concrete girders in North Carolina. The FIB girders are designed as simple spans for dead load and live load, but they are detailed as continuous for live load and are supported on steel-laminated elastomeric bearing pads with stainless steel sole plates and anchor bolts. In some cases, where designed to allow redistribution of vessel collision loads to adjacent bents, reinforced concrete shear keys are provided.

The use of precast concrete pile bents and vertical cylinder pile foundations minimizes the project's permanent impact on environmentally sensitive seagrass beds in the north approach spans.

To minimize the need for future dredging to accommodate Oregon Inlet's highly dynamic bathymetry, the design provides for a 2400-ft-wide "navigation zone." All spans within this zone provide at least 200 ft of

AESTHETICS **COMMENTARY**

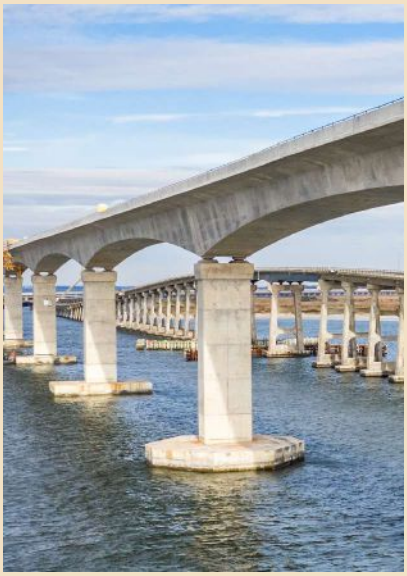
by Frederick Gottemoeller

For observers on the shore, long, relatively low bridges over water have an unfortunate aesthetic impact. Absent the bridge, water-level observers may have 180 degrees or more of seascape to admire, along with sunrises or sunsets and long-distance views of dramatic weather events. However, when a typical short-span causeway is viewed from shore at the usual oblique angles, the pile bents line up one behind the other to form a visual wall, cutting the visible water surface in half and destroying the sweeping, wide-angle exposures otherwise available.

The Marc Basnight Bridge impressively applies modern foundation technologies to the challenging conditions of the Oregon Inlet. Most of the major construction decisions emanated from those conditions, including the decisions to double the spans, raise the bridge height, and use haunched girders over approximately 25% of the bridge's length. A happy consequence of those choices is that the bridge's long spans and high clearances eliminate the most objectionable feature of typical causeway bridges—the way they block views of the water from the shore. With the new structure, long water-level views of the inlet are visible through the bridge.

The haunched girders also engage viewers by providing information about how the bridge works. The girders are thickest over the piers, where the forces are the highest. Finally, the disc bearings and their pedestals raise the girders above the piers just enough to provide a glimpse of sky between the tops of the piers and the bottoms of the girders, so that the girders appear to be floating in midair.

The aesthetic consequences of decisions made for technical reasons will make the Basnight Bridge a valued improvement to the Oregon Inlet seascape. With this example in front of them, perhaps other transportation agencies will now consider significantly longer spans and higher clearances for other water crossings with high scenic values, even if the technical issues are not as difficult as those faced in the Basnight project.



An innovative "leapfrog" approach to work-trestle construction for the north approach spans reduced the project's temporary environmental impact associated with shading of seagrass beds.

horizontal clearance and 70 ft of vertical clearance. By simply moving the navigation lights and buoys, the span identified for ship passage can be easily changed to accommodate migration of the natural channel. The resulting 3550-ft-long navigation unit consists of nine 350-ft-long main spans and two 200-ft-long end spans. The unit is a single, continuous post-tensioned concrete segmental structure with 238 single-cell precast concrete box-girder segments supported on post-tensioned precast concrete columns; this is one of the longest continuous precast, balanced-cantilever segmental concrete box-girder units in the United States. The variable-depth segmental superstructure provides a solution that is both aesthetically pleasing and economical.

Precast concrete segments were match-cast under controlled conditions at an off-site precast concrete facility, which resulted in high-quality, durable components.

Segments for the variable-depth, balanced-cantilever segmental box-girder superstructure were delivered by barge. In general, the north and south transition spans and the navigation span regions of the bridge were accessible by barge. Segments for the north and south approach spans regions were delivered either overland or in shallow water because barge access was not possible. The balanced-cantilever construction of the box-girder spans did require the use of temporary shoring towers to accommodate out-of-balance loading as segments were erected on either side of the piers.

The application of the balanced-cantilever construction method permitted the 350-ft-long spans. The superstructure segments range in height from 9 ft 0 in. at the midspan to 19 ft 0 in. at the interior piers. Longitudinal post-tensioning consists of 18- or 22-strand cantilever tendons and either 12-, 16-, or 20-strand continuity tendons encased in plastic ducts and high-strength grout. The cantilever tendons were installed in the top of the segment as each segment was erected. These tendons provide negative moment capacity for the box girder in the cantilever condition during erection as well as negative moment capacity in the completed multispan continuous-span structure. Once the closure pour was made at midspan, the continuity tendons were installed in the bottom of the segments. These tendons provide positive moment capacity in the completed continuous multispan structure. One pair of contingency ducts is provided for each cantilever. Segments are transversly post-tensioned using four-strand tendons spaced at 3 ft 6 in. All tendons are comprised of 0.6-in.-diameter, 270-ksi low-relaxation prestressing strands. The diaphragms use vertical 1¾-in.-diameter, 150-ksi deformed post-tensioning bars to resist vertical splitting induced by the longitudinal post-tensioning.

The superstructure is supported by two disc bearings at each pier, and the entire 3550-ft-long navigation unit is longitudinally restrained at the two middle piers. Bearings at other piers are designed to slide longitudinally to relieve thermal, creep, and shrinkage stresses. The construction sequence included midspan jacking prior to closure of the center span to counteract long-term creep and shrinkage effects

Creative pile templates were used to control the geometry during installation of battered 36-in.-square piles that were at least 130 ft long.

Balanced-cantilever erection of precast concrete segments in the 350-ft-long spans of the high-level navigation unit. Falsework towers, crane and materials barges, and two segment erectors (green and yellow hoists on ends of nearly completed cantilevers) are visible. The long spans allow the migration of the natural channel, and there is no need to constantly dredge to maintain a navigation channel.

on the two fixed piers (see the related Concrete Bridge Technology article in this issue of *ASPIRE*.)

Corrosion Protection

The 100-year design service life for the Basnight Bridge was achieved by incorporating a number of design and construction features, many of which were prescribed in the North Carolina Department of Transportation (NCDOT) request for proposals for the design-build project. NCDOT required concrete that would greatly enhance the durability and longevity of all concrete elements of the bridge, including the extensive use of fly ash or ground granulated blast-furnace slag and silica fume, a low water-cementitious materials ratio, and the use of a calcium nitrite corrosion-inhibiting admixture. In addition, stainless steel reinforcement was used for all CIP concrete except the barrier rails, which used epoxy-coated reinforcement. Stainless steel post-tensioning bars were used for post-tensioning applications below the splash zone, defined as 12 ft above the mean high-water level. Furthermore, stringent design criteria, such as a zero-tension stress requirement for prestressed concrete members under service load conditions, augmented the level of corrosion protection provided through material and construction requirements.

Conclusion

The Marc Basnight Bridge is a monumental structure built in a challenging marine environment and designed to achieve a 100-year service life with minimal maintenance. The extensive use of precast concrete elements—including 3.4 miles of precast concrete cylinder piles, 12 miles of precast concrete square piles, 0.58 miles of precast concrete bent caps, 0.3 miles of precast concrete columns, 8.75 miles of precast concrete FIB girders, and 0.67 miles of precast concrete segmental box girders—greatly enhanced the quality and durability of the structure, while simultaneously facilitating faster, safer, and more economical construction. In total, over 25 miles of precast concrete structural elements were used in the construction of the bridge; this represented over two-thirds of the 90,000 yd^3 of concrete in the structure, of which approximately 18,000 yd^3 was sand-LWC. The contractor's bid was $215.7 million, which was approximately $25 million below NCDOT's estimate. The new bridge opened to traffic on February 25, 2019, with a dedication ceremony held on April 2.

Domenic Coletti, R. Dominick Amico, Nicholas Burdette, and Mohit Garg are senior bridge engineers with HDR in Raleigh, N.C., Charlotte, N.C., Pittsburgh, Pa., and Tampa, Fla., respectively.

Courtland Street Bridge Replacement Project—Tight Space, Tight Schedule

by Al Bowman, Michael Baker International, and Mike Nadolski, C.W. Matthews Contracting Company

The partnering approach taken by the Georgia Department of Transportation, Michael Baker International, and C.W. Matthews ensured that the Courtland Street Bridge project could be executed rapidly despite a constricted urban site. All Photos: Georgia Department of Transportation.

A vital lifeline in Atlanta, Ga., the Courtland Street Bridge provides access to the state capitol and Georgia State University. This bridge replacement project was contracted by the Georgia Department of Transportation (GDOT) as a design-build collaboration between Georgia firms C.W. Matthews Contracting Company and Michael Baker International.

Between May and October 2018, the project replaced the 110-year-old Courtland Street Bridge in downtown Atlanta in just 155 days. The design-build team used micropiles, precast concrete beams, high early-strength concrete, and innovative phased-design packages to meet this aggressive schedule. While replacing this historic bridge, the team resolved challenges involving extensive utilities, a severely constricted right-of-way, and multiple safety and stakeholder coordination issues.

Historical Background

At the start of the 20th century, the railroad system that was helping Atlanta grow also divided the city, inconveniencing residents and prompting calls for a viaduct on Washington Street to help with traffic flow. Rail companies also had a stake in developing a viaduct because they wanted to close a nearby crossing to modify their railyard operations. In 1906, after several years of debate, the Atlanta City Council approved construction of the Washington Street Viaduct (as the Courtland Street Bridge was then called).

The original 28-span bridge was 1546 ft long and approximate 58 ft wide, with 8-ft-wide sidewalks on both sides. The existing bridge had multiple configurations; it included steel spans over the railroad and to the south and used reinforced concrete spans from the approaches to the abutments. The project, which became Atlanta's longest viaduct, took roughly 18 months and

profile

COURTLAND STREET BRIDGE / ATLANTA, GEORGIA

DESIGN ENGINEER: Michael Baker International, Norcross, Ga.

PRIME CONTRACTOR: C.W. Matthews Contracting Company, Marietta, Ga.

PRECASTER: Standard Concrete Products, Atlanta, Ga.—a PCI-certified producer

OTHER BRIDGE CONSULTANTS: Bridge deck drainage: Aulick Engineering, Atlanta, Ga.; lighting: Key Engineering, Decatur, Ala.; micropiles: Hayward Baker, Hanover, Md.

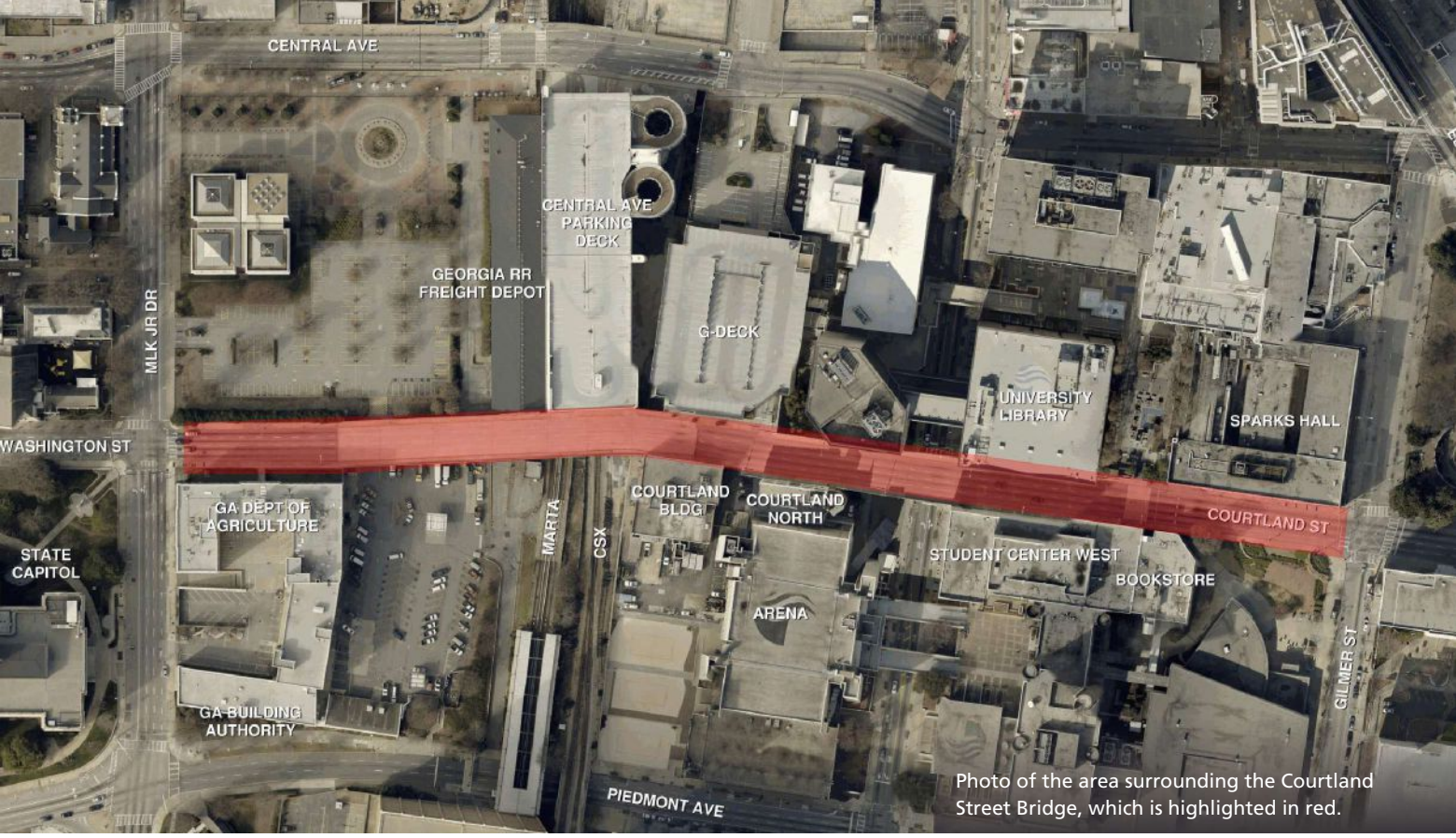


Photo of the area surrounding the Courtland Street Bridge, which is highlighted in red.

cost $126,180, with the railroads paying most costs and the city paying the rest.

The Washington Street Viaduct opened in September 1907. In 1909, the Auditorium and Armory was built nearby. This cultural facility, which hosted events such as the Metropolitan Opera's visits to Atlanta, helped make the thoroughfare a bustling center for Atlanta's social and business functions. In 1913, Georgia Tech (the predecessor to Georgia State University) established a presence next to the viaduct. Today, Georgia State University's urban campus is split by the Courtland Street Bridge; the Auditorium and Armory is now Georgia State's Dahlberg Hall.

Project Strategies

Before 2018, repair work on the Courtland Street Bridge had included reconstruction projects in 1950 and 1995. However, the increased volume and size of vehicles crossing the bridge prompted GDOT to seek a more permanent and lasting solution. To minimize the time the bridge would be out of service, GDOT opted to reconstruct the bridge using a design-build approach, including accelerated bridge construction (ABC) techniques.

From project inception, the design-build team recognized that utility conflicts and a restricted right-of-way posed major challenges. Conventional ABC techniques were not viable on this project due to the constricted right-of-way; therefore, the optimal way to meet project schedule goals was through an aggressive work schedule and innovative use of materials.

Using subsurface utility plans completed by GDOT, the team created


New bridge bents were constructed beneath the existing bridge while it remained open. This technique shaved several months off the road closure.

GEORGIA DEPARTMENT OF TRANSPORTATION, OWNER

BRIDGE DESCRIPTION: 1130-ft-long, 58-ft-wide, 12-span prestressed concrete girder bridge with cast-in-place concrete bents on micropile foundations

STRUCTURAL COMPONENTS: 142 micropiles, 13 concrete bents, 36 AASHTO-Type III beams totaling 2666 ft in length, 28 BT-54 beams totaling 2502 ft in length, eight BT-63 beams totaling 1015 ft in length, 14 BT-65 beams totaling 1871 ft in length, and an 8¼-in.-thick cast-in-place concrete deck

CONSTRUCTION COST: $21 million

AWARD: 2018 Georgia Partnership for Transportation Quality Preconstruction Award—Design-Build

Several prestressed concrete girders have been placed for the span in the foreground. Because of the confined site, the crane had to back out after all girders were set for a span so the cap for the next span could be constructed.

Prestressed concrete girders have been erected and forms set for placement of the deck.

The completed Courtland Street Bridge.

scaled-foundation cutouts from clear plastic stock to determine where and how foundations could be placed to avoid the most utilities and provide acceptable span lengths that could be accommodated by equipment and the limited access in the constrained workspace. In addition to the extensive utilities, a series of driveways and pedestrian access to two parking garages also had to be maintained.

Adding to the project's complexity, the existing Courtland Street Bridge had several different types of foundations that had been retrofitted over the years. Furthermore, while using ground-penetrating radar to determine the extents and types of foundations, the team discovered that a building had been constructed around one of the bridge's foundations.

To overcome the array of challenges, the team plotted all the driveways, utilities, and existing footings to determine the appropriate span lengths that would minimize site conflicts while allowing for material deliveries in the narrow right-of-way. This plan worked well, and all but five footings were built without conflicts to utilities and existing footings.

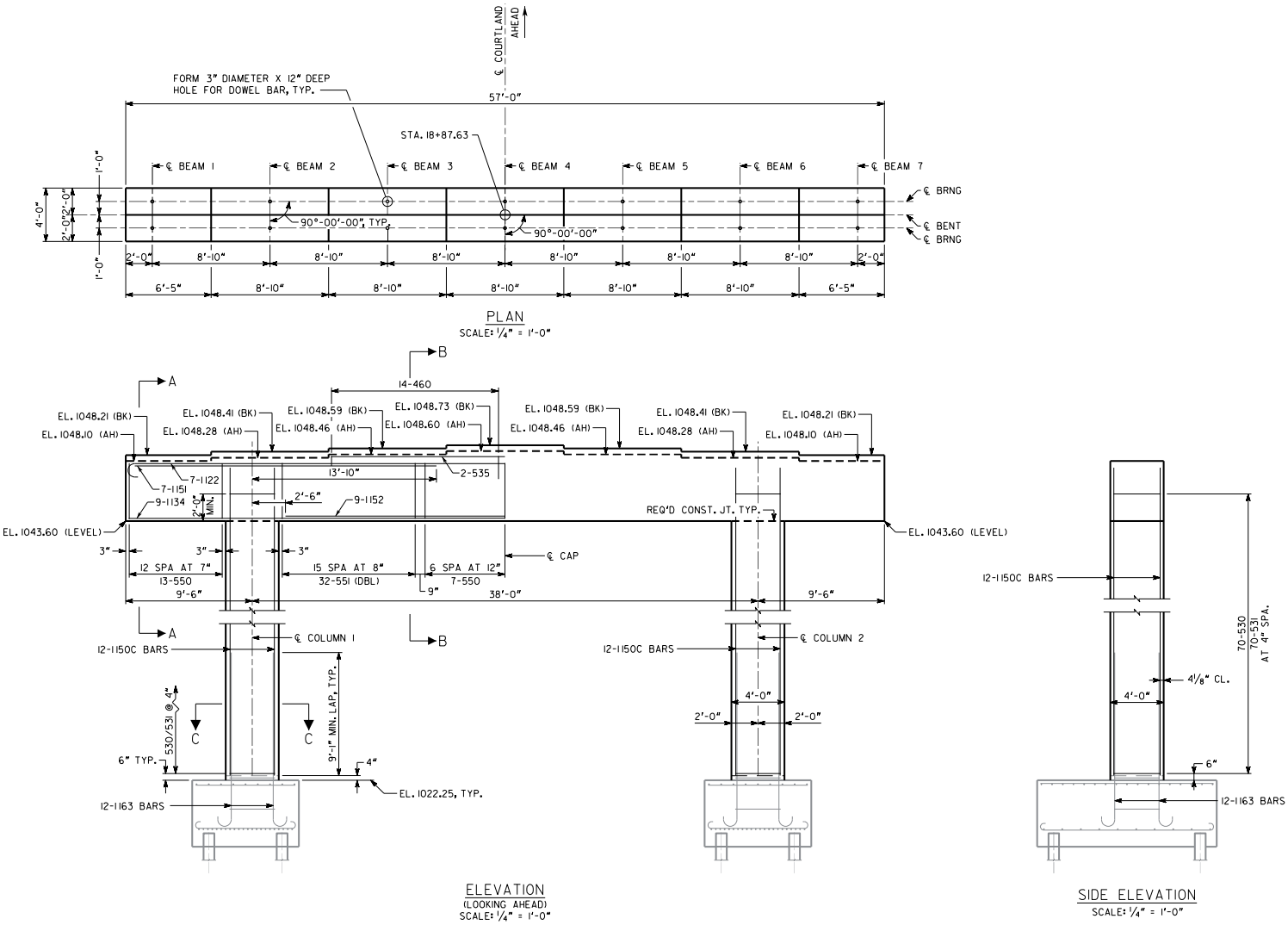
Micropile foundations were selected because they offered multiple advantages over other deep foundation types. First, they could be installed with low-overhead equipment beneath the existing bridge while it remained in service, which was important because the space between the ground and the old bridge was as low as 12 ft in some areas. Second, micropiles—with their drilled installation method—would not risk vibration damage to the surrounding structures. Third, micropiles generally have more capacity than traditional piles; therefore, fewer were needed, which helped to avoid utility conflicts.

The 9.625-in.-diameter micropiles were installed 80 to 90 ft deep in bedrock and bonded using high-strength grout. These piles carried a factored axial load of 555 kip downward and 125 kip in uplift. The uplift capacity allowed the designers to minimize the footing sizes, further aiding in the avoidance of utilities and other subsurface obstacles.
Because of the constricted construction space, beam-erecting equipment had to be placed within the footprint of the project. This limited most spans to less than 90 ft in the vicinity of the Georgia State campus. Multiple types and sizes of prestressed concrete beams were used because they proved to be an economical solution. Once a span of beams was placed, the crane would back out of that span and the substructure cap of the next span would be formed and placed.

Given the aggressive project schedule, waiting the typical 10 or more days for concrete to reach its required strength was not a viable option. Working with a concrete supplier, the design-build team developed high-early-strength concrete that could achieve the required strength within 24 hours. Using precast concrete bridge piers with a grouted connection was considered, but this option was not selected because the proposed bridge was wide and would have required thick bridge caps, which would have been difficult to transport given the restricted project site.

Using prefabricated steel diaphragms at midspan is a common practice in Georgia; however, to expedite this project, the team took the unusual step of using a steel diaphragm at the end of each span. A Georgia first, the team's use of prefabricated steel k-frames at each end eliminated the labor needed to form the traditional cast-in-place concrete end diaphragms, thereby reducing the time needed for each span placement by one week. This innovative approach helped ensure that the project stayed on schedule.

An important project requirement was limiting closure of the bridge to six months. The construction team had a 24/7 presence on the project once the bridge was closed. To meet the aggressive project schedule, an innovative phased approach to the design, as well as the construction, was used. The team was given its notice to proceed in September 2017. The goal was to have the contractor installing foundations by January 1, 2018, and setting the thousands of feet of precast, prestressed concrete beams by May 2018. To that end, the designers worked with GDOT to gain approval of the foundation and beam designs before the drawings for the entire



Intermediate bent details for the Courtland Street Bridge. High-early-strength concrete was used to achieve the design concrete compressive strength within 24 hours. Figure: Michael Baker International.

bridge were completed. Early approval of these phased packages allowed construction of the project to progress on schedule while other, minor details were still being discussed.

Demolition Challenges

Demolition began in May 2018, coinciding with the end of the spring semester at Georgia State. The construction team deployed two demolition contractors, which worked in opposite directions (north and south) starting at the railroad tracks.

During demolition, stakeholder engagement and safety precautions were vital. In some places, only ½ in. of expansion material separated the demolition area from nearby buildings, which included one building that had historic preservation status. The university's summer session was also underway; therefore, in-depth stakeholder coordination was required to maintain safe egress zones around the construction site. Demolition over the Metropolitan Atlanta Rapid Transit Authority and CSX railway corridor posed even more challenges because work could only occur there for 90 minutes a night.

Conclusion

The new, 12-span Courtland Street Bridge is 1130 ft long and 58 ft wide with a cast-in-place deck, 86 prestressed concrete bulb-tee and AASHTO-Type III girders, and cast-in-place concrete bents supported on micropiles. Its structural components include 142 micropiles, 13 concrete bents, 36 Type III beams, 28 BT-54 beams, 8 BT-63 beams, and 14 BT-65 beams. The design-build team's expertise, GDOT's flexibility, and Georgia State University's cooperation were all essential to overcome the logistical challenges of rebuilding this important historical connection through the heart of Atlanta. A

Al Bowman is the director of transportation for Michael Baker International in Norcross, Ga. Mike Nadolski is a project manager with C.W. Matthews Contracting Company in Marietta, Ga.

Iterative Substructure Design for the Marc Basnight Bridge Navigation Unit

by Nicholas Burdette and Mohit Garg, HDR

The navigation portion of the new Marc Basnight Bridge is an 11-span, 3550-ft-long continuous precast, post-tensioned concrete segmental box-girder unit, which is believed to be the third-longest continuous unit of its type in the U.S. (for more information on this project, see the Project article in this issue of *ASPIRE*®). For a segmental unit of this length, a primary substructure design challenge was determining the number of piers to have fixed bearings. This article outlines the unique approach the design team used to determine the number of fixed bearings and arrive at a pier design that would satisfy the array of requirements.

Design Process

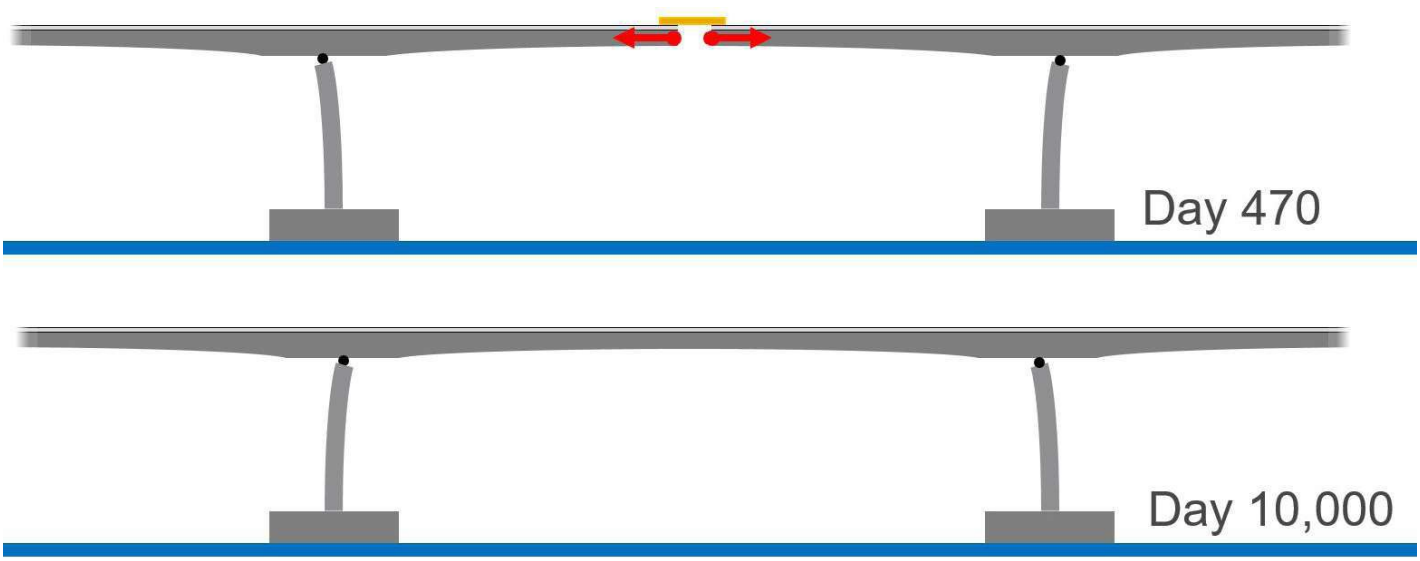
Fixed bearings are necessary to resist the significant longitudinal forces caused by wind and braking. While using several fixed-bearing piers would help distribute these forces, other forces due to creep, shrinkage, and thermal deformations accumulate and increase as a function of a fixed-bearing pier's distance from the center of the unit. On a long, continuous bridge these displacement-driven forces are substantial and increase over time.

To offset the effects of these forces, the design team decided to "preload" the structure by jacking the segmental cantilevers apart in the longitudinal direction between fixed-bearing piers before the cast-in-place (CIP) closure segment was placed. This process induced a shear force in the columns in a direction opposite to the column shear loading produced by creep and shrinkage of the superstructure.

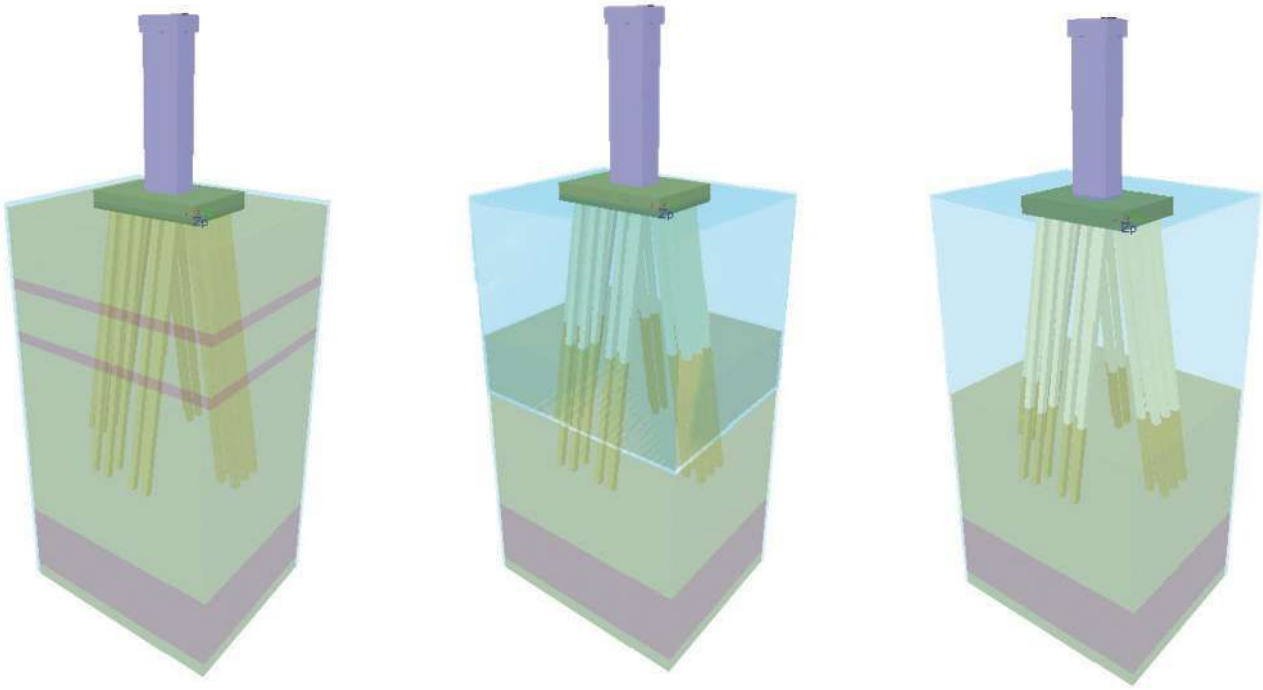
An added complication to the design was the potential for extreme scour, ranging up to 84 ft deep for the 100-year scour, which significantly changes the foundation stiffness. The structure had to be designed to resist longitudinal force-driven loadings due to wind and braking, and longitudinal displacement-driven loadings caused by creep, shrinkage, and thermal deformations of the superstructure at both the zero-scour and maximum-scour conditions. The zero-scour scenario represents the stiffest foundations and thus produces the largest displacement-driven longitudinal forces on the piers.

Over a dozen iterations of superstructure and substructure designs were performed to determine the optimal pier fixities, column and foundation designs, and superstructure jacking force.

The segmental superstructure was designed using a three-dimensional, time-dependent, staged-construction



At midspan of the Marc Basnight Bridge's navigation unit, the two 1825-ft-long halves were jacked apart with a 600-ton jacking force prior to midspan closure (top). This effectively "preloaded" the pier columns with a 600-ton shear force in the direction opposite to the anticipated creep and shrinkage deformations of the superstructure and helped offset the maximum design creep, shrinkage, and thermal contraction case (bottom). All Photos and Figures: HDR.



FB-MultiPier models of the foundation showing the range of design scour levels: zero scour, long-term scour, and 100-year scour of 84 ft.

model developed with LARSA 4D. This model included the pier columns down to the pile caps, where 6 × 6 stiffness matrices were used to represent the stiffness of the foundations. The analysis included steps during construction for the erection of the segments, post-tensioning, and longitudinal jacking of the segmental cantilevers between the fixed-bearing piers before placing the last CIP closure segment at the center of the unit. For design of the completed structure, this model considered two key time steps: Day 470, which was the completion of the last bridge span, and Day 10,000, when all creep, shrinkage, and post-tensioning relaxation are assumed to have occurred. The displacement-driven forces on the fixed-bearing piers increased by approximately 75% over this period.

The navigation unit substructure consists of precast, post-tensioned, 16-ft-wide, 11-ft-deep, and 70-ft-tall hollow concrete columns. These columns are on a CIP pile cap and battered, 36-in.-square precast concrete piles. The substructure was modeled using FB-MultiPier (see the Concrete Bridge Technology article in the Summer 2019 issue of *ASPIRE*) to assess the soil-structure interaction and nonlinear geometric and material effects. Each pier was modeled individually with appropriate design scour levels for strength, service, and extreme event (vessel collision) load combinations. A stiffness matrix representing the foundation stiffness of the pile pattern and a given scour level for each fixed pier was exported from FB-MultiPier and used to represent the foundation stiffness in the LARSA model for each design iteration.

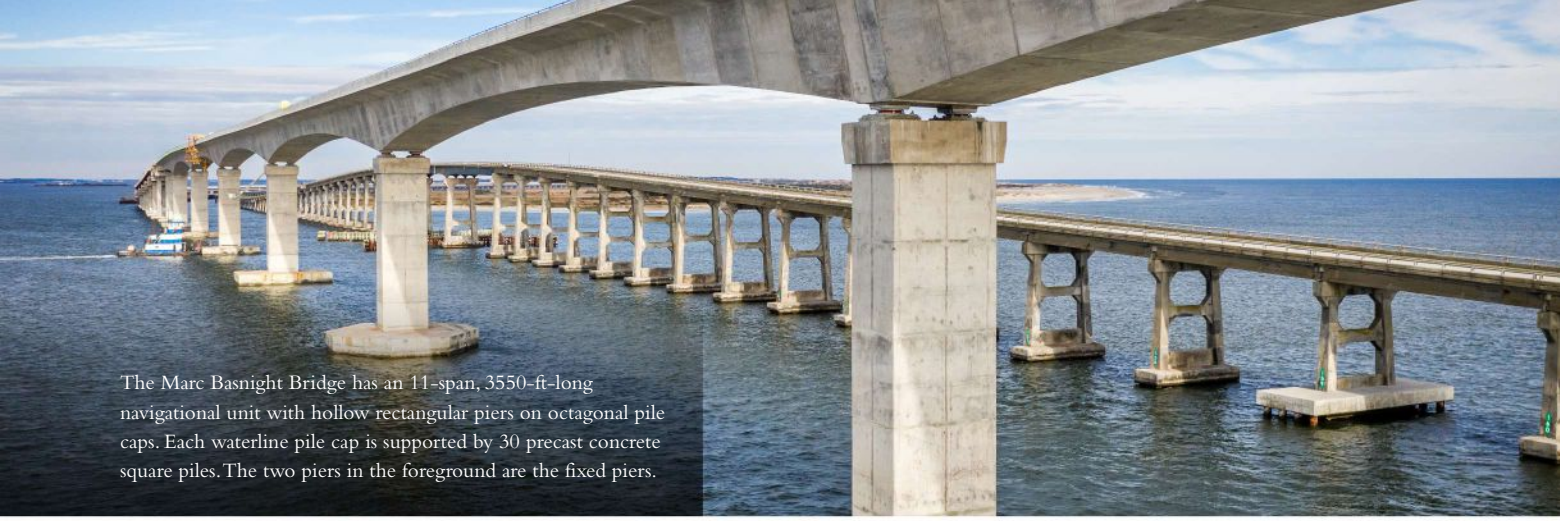
Each change in bearing articulation or substructure stiffness required a full design iteration of the global LARSA model. The bearing articulation in the model would be changed, pier sections updated, and new 6 × 6 stiffness matrices for the foundations provided. For substructure evaluation, the LARSA model was used primarily to evaluate bearing reactions for the stiffest substructure condition (zero scour) at Day 470 and Day 10,000. Following full, staged LARSA analysis, the resulting bearing reactions were summarized for incorporation in the FB-MultiPier models; this represented one design iteration.

The Optimal Number of Fixed-Bearing Piers

Initial designs of the 12-pier navigation unit considered the central six piers fixed, with the intent of distributing force-driven longitudinal loads among more piers to limit foundation sizes, especially for the 100-year-scour condition. For cases of zero scour, however, the foundations are extremely stiff and the box columns do not provide much flexibility. These stocky columns are further stiffened by post-tensioning designed to prevent tension in the columns under service loading. Given the lack of substructure flexibility, displacement-driven loads from the 1750 ft of superstructure between the outermost fixed piers were excessive.

The next design iteration was run assuming only four fixed piers. Resisting displacement-driven forces with the four fixed piers became difficult. Larger foundations (producing increased stiffness) were required at maximum scour, and several design refinements were attempted to address column overstress issues. A variety of pier column designs were investigated—including hollow rectangular sections with thinner walls, closer longitudinal pier spacings, and twin-wall pier configurations—in an attempt to reduce the longitudinal column stiffness for displacement-driven loading while maintaining adequate strength to resist the longitudinal force-driven column loads unaffected by column stiffness.

Also investigated was the construction sequence of jacking and closure segments for the three spans between the four fixed piers. The best jacking combination reduced loads on the interior fixed piers, but the outer fixed piers were heavily loaded and required very large foundations.



The Marc Basnight Bridge has an 11-span, 3550-ft-long navigational unit with hollow rectangular piers on octagonal pile caps. Each waterline pile cap is supported by 30 precast concrete square piles. The two piers in the foreground are the fixed piers.

In the end, none of these refinements proved satisfactory. It was not possible to reduce the column stiffness enough to decrease displacement-driven loads on the outer fixed piers without compromising their ability to resist these loads combined with the force-driven loads, which were divided evenly among the four fixed piers.

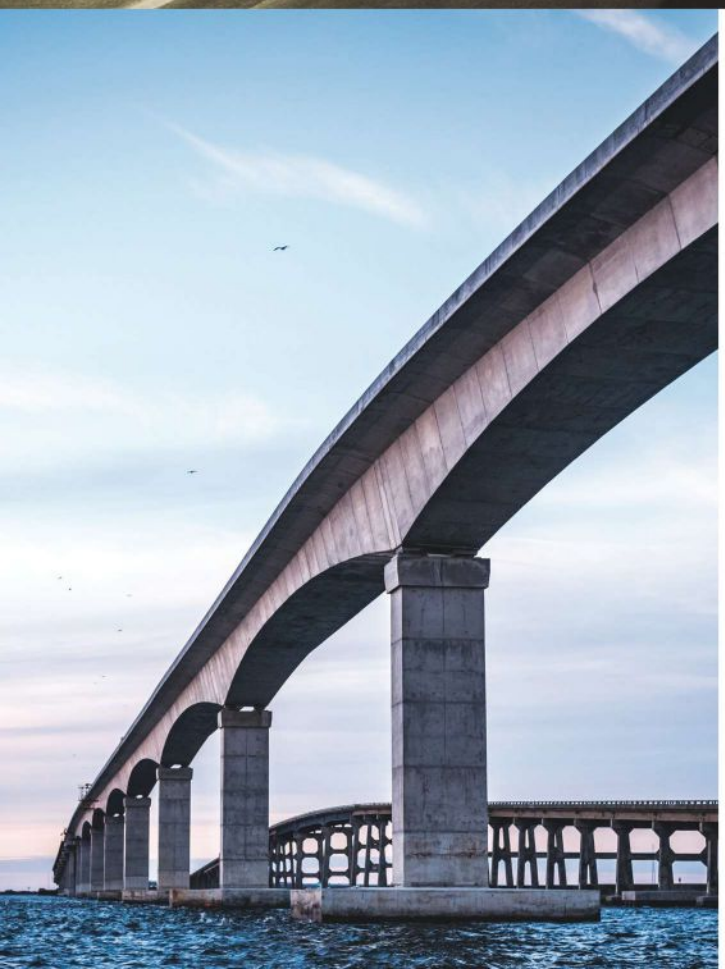
The optimal design solution incorporated only two fixed-bearing piers. This solution required that each of the fixed-bearing piers resist half of the longitudinal force-driven loads from the entire 3550-ft-long unit; as a result, larger foundations and significant column post-tensioning were necessary for those two piers. However, the two-fixed-pier solution significantly reduced the displacement-driven force effects by reducing the contracting distance to half the length of the 350-ft-long span between the two fixed piers. This meant that the increased substructure stiffness resulting from the larger columns and foundations required for these two piers to resist force-driven loading did not lead to excessive restraint forces under displacement-driven loading. This solution also limited jacking to one location between the fixed piers in the central span, where the application of a 600-ton jacking force was sufficient to offset a large portion of the displacement-driven forces occurring between Day 470 and Day 10,000.

Conclusion

This process of optimizing the number and type of fixed piers highlights the importance of close integration of superstructure and substructure design for longitudinal forces in segmental concrete bridges. Designers should understand how decisions made above and below the bridge bearings affect one another to produce an optimal design. A

Nicholas Burdette and Mohit Garg are senior bridge engineers with HDR in Pittsburgh, Pa., and Tampa, Fla., respectively.

Photo Credit: New York State Thruway Authority



Shasta Viaduct Arch Bridge: Improved by Pumping a Lightweight Concrete Solution

By Ric Maggenti, California Department of Transportation (retired), and
Sonny Fereira, California Department of Transportation

The Shasta Viaduct Arch Bridge is located on southbound Interstate 5 (I-5), in Shasta County, Calif., about 15 miles north of Redding and just north of Lake Shasta. This new bridge replaces the 78-year-old cast-in-place (CIP), conventionally reinforced concrete Sidehill Viaduct. The new arch bridge, which was partially opened to traffic in July 2019, is part of an improved horizontal alignment at that portion of southbound I-5. An arch support system was chosen to avoid conflict with the Union Pacific Railroad tunnel that passes beneath the bridge.

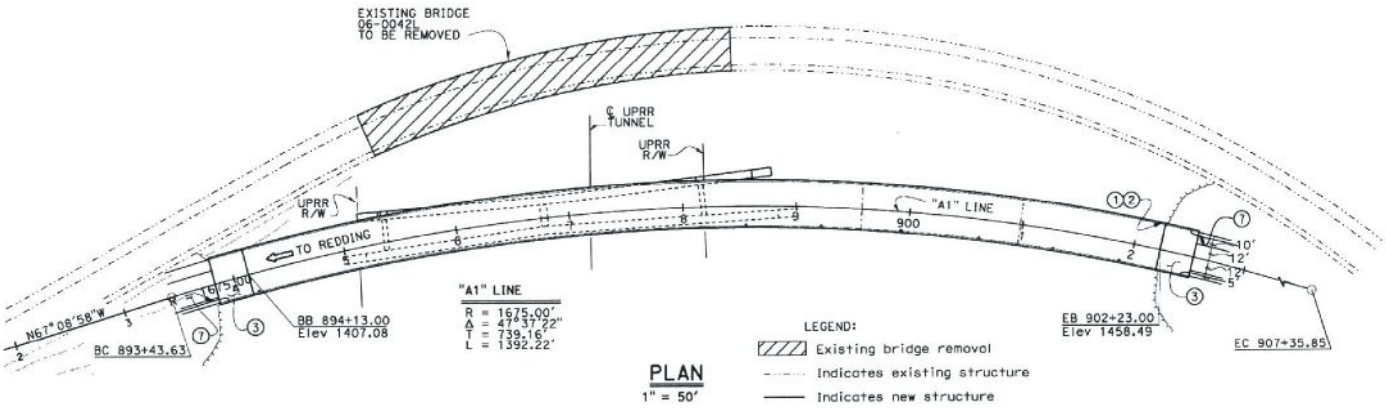
The Shasta Viaduct Arch Bridge is a six-span, post-tensioned, CIP, continuous concrete box-girder superstructure, with four of the spans supported by a two-ribbed arch. The arch support was designed to be within the perimeter of the superstructure. To accommodate the horizontally curved alignment, the ribs making up the arch are asymmetric, with each rib having a lateral kink near its apex. These kinks have different angles; therefore, four separate alignments make up the ribs and no two alignments are parallel. Given the steep hillside, each rib is footed on thrust blocks at differing elevations and the chord lengths of each rib vary accordingly. Thus, the left rib spans 366 ft while the right rib spans 400 ft.

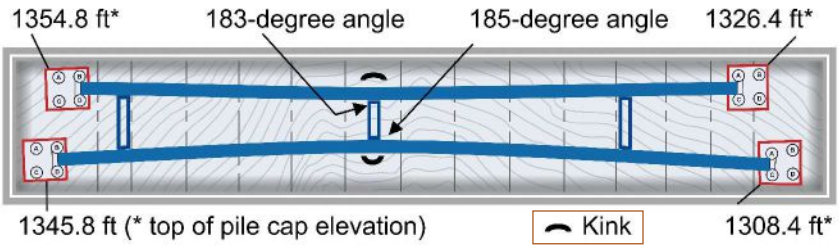
The asymmetric ribs complicated the design analysis. After the initial design and completion of the arch ribs and columns, the structure was evaluated using a nonlinear, second-order analysis with time-dependent effects. Following the evaluation, modifications to the initial design were made to ensure the structure's design life. The most significant modification was lowering the dead load on the arch ribs by using lightweight concrete (LWC) instead of normalweight concrete (NWC) in the superstructure box girder including deck, webs, bottom slab, intermediate and abutment diaphragms, and bent caps. All other bridge elements used NWC, including the bridge rail. Capacity of the ribs was increased by providing a 1-ft-thick NWC collar, placed by shotcrete, around the lower 17 ft of the 7-ft-square ribs. Struts connecting the two ribs were also carbon-fiber wrapped to improve the arch struts' performance.

Lightweight Concrete Mixture Proportions

The original NWC 28-day design compressive strength f'_c of 4000 psi was increased to 5500 psi for the LWC. The increased strength eliminated any need to adjust the shear, creep, modulus of elasticity, and modulus of rupture values used in the original design. In design codes, these empirically derived material properties based on the compressive strength are assumed to be higher for NWC than LWC. By increasing the compressive strength of the LWC, the values of these properties converge to those of the lower-strength NWC that were used in the original design. A target equilibrium density for the LWC of 120 lb/ft^3 was selected for the design; in the interest of time, an oven-dry density of no more than 112 lb/ft^3 was used for the trial batches. With a density of 120 lb/ft^3, there was no need for lightweight sand; therefore, local sand was used in the mixture. The same LWC mixture was used

Plan for the Shasta Viaduct Arch Bridge showing the horizontal curve with spiral transitions, as well as the existing horizontal alignment and the existing bridge. Figure: Ric Maggenti.





Plan view of the two asymmetrical arch ribs. Located on a steep hillside, the ribs, which span 366 and 400 ft, have lateral kinks near their apexes, and their ends are supported on thrust blocks at different elevations. Figure: Ric Maggenti.

for all LWC components except the deck, which was modified to include fibers.

The bridge structure is located in an area prone to freezing and thawing, so entrained air was to be no less than 5%. The concrete needed the capability of being pumped at a minimum rate of 60 yd^3/hr. Therefore, a maximum ¾-in.-diameter lightweight aggregate (LWA) sourced from Frazier Park, Calif., was selected. This LWA could be delivered fully saturated, which is a necessary condition for pumping the LWC. Proof pumping to a height of 100 ft was done at the bridge site.

Trial Batching

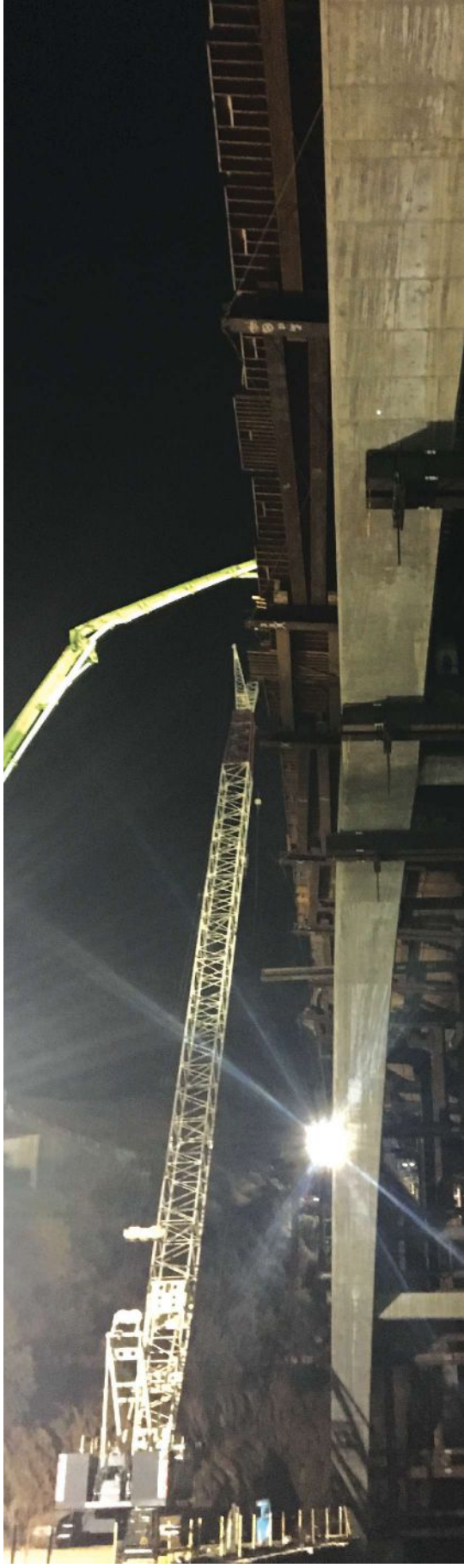
Iterative trial batches revealed that properly air-entrained mixtures with a maximum water–cementitious materials ratio (*w/cm*) of 0.35 achieved adequate strengths and met the 120 lb/ft^3 equilibrium density criterion. Because the batch plant was located more than an hour away from the bridge site, chemical admixture types and dosage rates were adjusted to provide a stable slump and, more importantly, consistent air content, which affects both strength and density.

Some of the first trial batches could not be pumped at even a 30 yd^3/hr rate. However, the pumping issues were eventually solved, and the final mixture could be pumped at a rate exceeding 100 yd^3/hr and to a height of 100 ft.

Pumpable Lightweight Concrete

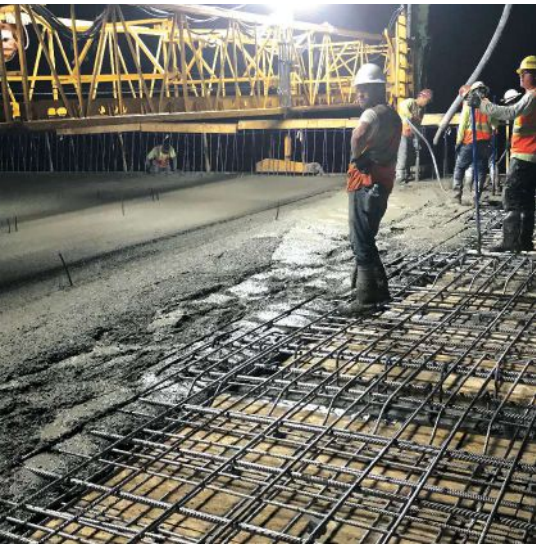
LWA must be fully saturated prior to pumping, which may be accomplished by vacuum or thermal methods for some LWAs. Aside from that requirement, fresh concrete properties necessary for pumping LWC are the same as for pumping NWC. Pumpability is related to dynamic behavior under pressure and the viscosity of the mixture as measured by slump. Slump is directly related to water content and admixtures. However, slump gained by admixtures changes the rheology of a mixture, making the "water slump" (the slump before admixtures are added) important. All things being equal, a higher slump enhances pumping as long as the mixture remains cohesive. The cement paste is the transport fluid that moves the concrete through the pump. The mortar conveys the coarse aggregate, keeping it in suspension and making the concrete pumpable, which is why the sand gradation is an important factor for pumpability. The coarse aggregate affects the plastic and cohesive qualities of fresh concrete, as well as the water demand to achieve slump values.

An early pumpability trial was a failure. The pump was overly strained before even a 30 yd^3/hr rate was achieved, and then the pump plugged. For a subsequent trial, the amount of cementitious material was increased from approximately 850 to 900 lb/yd^3, and the aggregate proportions were adjusted. Because of the high cementitious materials content, with 50% slag and no fly ash, the mixture required mass concrete cooling tubes at the bent caps. Although the LWC was prone to some countertendencies, such as a hotter or stickier mixture, increasing the cementitious content provided several pumpability benefits. First, it allowed an increase in water content without exceeding 0.35 *w/cm*. This increased water slump. Second, it increased the paste content, which is the fluid transporting the aggregate. Third, it indirectly enhanced aggregate gradation regarding pumping because the extra cementitious material could be treated as part of the fine aggregate.



Lightweight concrete being pumped from a working platform on a work trestle up to the bridge superstructure. Photo: California Department of Transportation.

Placing lightweight concrete for bent caps by pumping. Because of the high cementitious materials content, active (cooling tubes) and passive measures were used to control the concrete temperature to avoid thermal cracking and the potential for later cracking caused by delayed ettringite formation.



Nighttime placement of lightweight concrete with polyfibers for the bridge deck. Photo: Ric Maggenti.

Proprietary tables and graphs for optimizing aggregate gradations for pumpable mixtures were developed back in the days when a 700 lb/yd^3 cement content was considered very high. Such mixture resources were used as guidance for developing a pumpable mixture with a cementitious content of 900 lb/yd^3 by considering 200 lb/yd^3 of the cementitious materials as fine aggregate. The gradation of the local natural sand was within the gradation band of ASTM C33, *Standard Specifications for Concrete Aggregate.*[1] By adjusting the sand gradation to include a portion of the cementitious materials as sand, as suggested by ASTM C33, the fineness modulus dropped from 2.91 to 2.51. A lower fineness modulus allows for a larger volume of coarse aggregate, optimizing a pumpable mixture without adding water.

Production Rates—Pump Them Up!

On the Shasta Viaduct Arch Bridge construction project, work shifts began in the evening and continued until the early morning. After 1600 yd^3 of concrete for the box girders and bent caps were pumped in four shifts during three weekends in August 2018, 900 yd^3 of deck concrete were pumped in two shifts during one weekend in September.

There were no issues with workability and pumpability. Inspection after form removal showed good consolidation. The pump rates at times exceeded 100 yd^3/hr. The net placement rates for the stem and soffits, which included stopping to move the pump and intermittent placement as the crews moved from stem to stem, were greater than 50 yd^3/hr. The last deck placement rate exceeded 75 yd^3/hr, despite some logistical problems that led to production delays. The concrete mixture for the deck included 4 lb/yd^3 of polyfibers, one of the provisions for California Department of Transportation (Caltrans) "CRACK-Less" concrete deck mixtures.

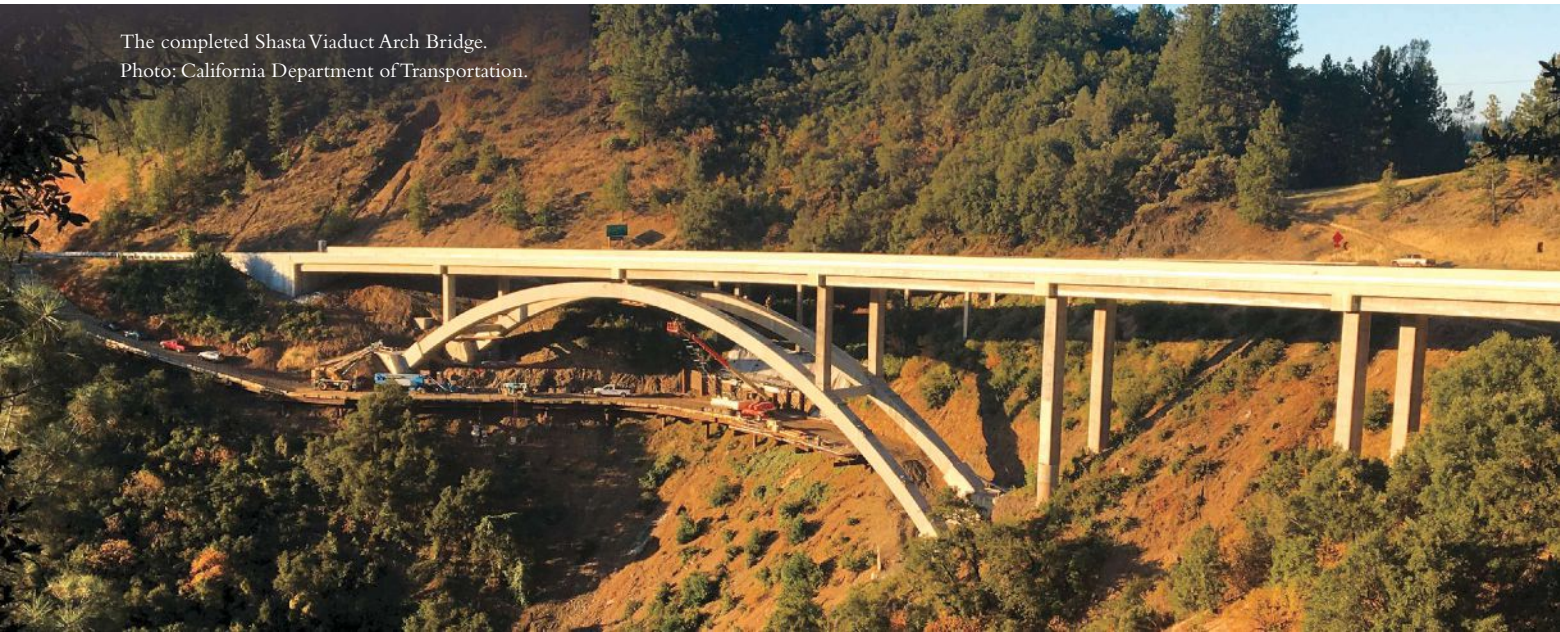
Conclusion

The LWC met or exceeded all expectations, including the capability to be pumped over 100 ft high. The Caltrans standard design life for a highway bridge is 75 years. Many design features and upgrades were included with service life in mind, such as stainless steel deck reinforcement, polyfibers in the deck concrete mixture, and inclusion of a 1-in.-thick protective polyester concrete deck overlay. With the reduced load resulting from the use of LWC for the superstructure, the support components should also see improved longevity. The resulting structure is anticipated to last as long and be as durable as the initially planned NWC bridge.

Reference

1. ASTM International. 2018. *Standard Specifications for Concrete Aggregate* (ASTM C33-18). West Conshohocken, PA: ASTM International.

Ric Maggenti was a senior bridge/materials and research engineer for the California Department of Transportation (stationed in Sacramento at the time of his retirement). Sonny Fereira is a senior area bridge construction engineer for the California Department of Transportation in Redding.



The completed Shasta Viaduct Arch Bridge. Photo: California Department of Transportation.

A Tool for Continuous On-Site Monitoring During Grouting of Post-Tensioning Tendons

by Dr. Andrea J. Schokker, University of Minnesota Duluth, and Jacob A. Bray, New Hudson Facades

This article describes the innovative use of an in-line density meter, a tool common to the chemical and food industries, to provide continuous information of the density of the grout during pumping. This tool can provide a significant indicator of post-tensioning (PT) grout quality as it enters and exits the ducts.

Background

For decades, PT grout has been successfully used to provide a highly alkaline and dense barrier for corrosion protection of PT strand in tendons while also bonding the tendons with the surrounding structure (in the case of internal tendons). Significant advancements have been made in material quality and the training of installers; however, notable quality issues, primarily related to bleed and soft grout, continue to be of concern. Overwatering of grout is the most significant factor related to poor field performance of PT grout. The prepackaged materials commonly used for grouting of PT tendons are tested within specific water ranges to achieve the desired performance. Therefore, it is critical to ensure that the grout placed in the tendon is within the same water range as the grout that was prequalified through testing. Overwatering can be caused by water in the tendons, water in the grouting line, or the addition of too much water to the mixture. Too much water may sometimes be added in an effort to have a less-dense mixture that is easier to pump, or an operator might make a measurement error. No matter the cause, the consequence of overwatering can be a compromised corrosion protective system.

According to the *Specification for Grouting of Post-Tensioned Structures*,[1] density measurements of the grout must be taken with the mud balance device at the inlet and the outlet of the tendon one time per day.[2] The mud balance is a straightforward test, but it has drawbacks:

- A discrete sample may not be representative of the rest of the grout, particularly if water is being picked up in the lines or tendon.
- If water is added after the initial mud balance testing, the change in density will not be recorded.
- There is potential for operator error, particularly with grouts that gel quickly and plug the weep hole in the mud balance device.

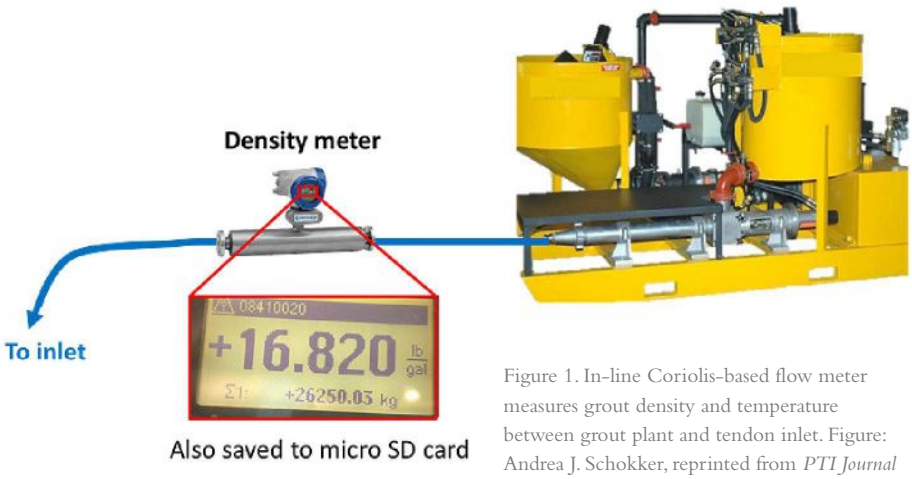


Figure 1. In-line Coriolis-based flow meter measures grout density and temperature between grout plant and tendon inlet. Figure: Andrea J. Schokker, reprinted from *PTI Journal* with permission.

Given these limitations of the mud balance test, the use of an in-line meter to continuously measure density could significantly improve the monitoring of grout quality. The meter is used commonly in industrial settings and is specified for slurry materials. It doesn't need to be calibrated, but it can be checked at any time by running water through it because water has a known density.

Test Plan

As described in detail in the *PTI Journal*,[3] a study sponsored by the Post-Tensioning Institute was conducted to evaluate the feasibility of using a commercially available in-line meter to monitor PT grouting. A schematic of the test setup is shown in **Fig. 1**. For this study, the Krohne OPTIMASS 1400C S25 Coriolis flow meter was selected. The density meter includes a digital readout as well as a data acquisition system that stores density and temperature measurements on a micro SD card for later analysis.

The test program included a comparison of data from the in-line meter and the current density measurement tool, the mud balance. Although data variation is expected, it is important to have confidence that the in-line meter values are consistent with the range of values obtained from existing practice. A wide range of grout densities was tested to provide a spread of data for comparison. Testing used two prepackaged grouts. The initial water contents were at the manufacturer's recommended value; then water was added in increments up to a value that was 65% over the maximum

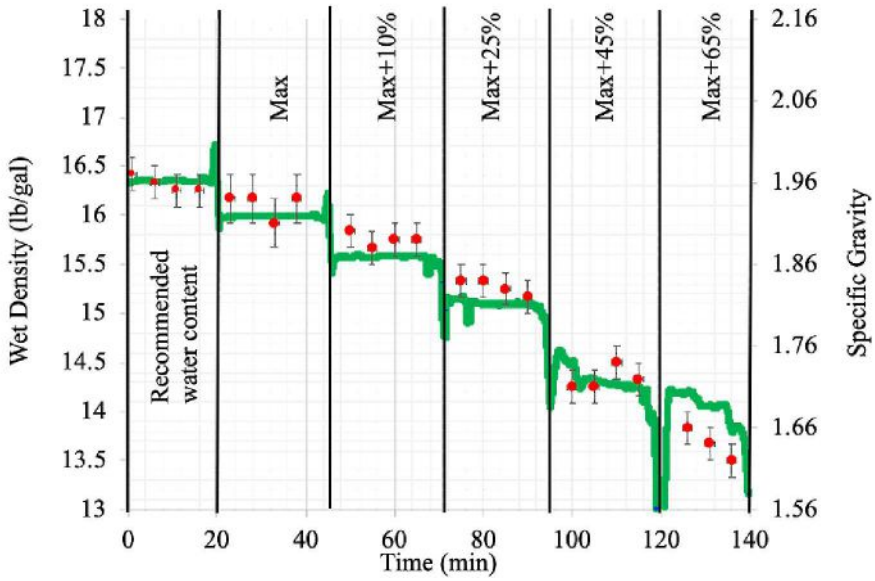


Figure 2. Comparison of continuous in-line Coriolis-based flow meter and individual mud balance density results for test with a single grout. In-line density meter data are in green, and mud balance data are in red. Figure: Andrea J. Schokker, adapted from *PTI Journal* with permission.

given by the manufacturer. Overwatering was done during testing to verify the performance of the device. However, it is important to note that, when used in the field, grouts should *never* have a water content above the maximum value stated by the manufacturer. Overwatered grouts can lead to significant bleed issues.

Grout testing involved mixing a batch in a commercial high-shear grout plant at the manufacturer's recommended water content, transferring the grout to the holding/agitator tank, and pumping it through a hose connected to the meter and then back into the holding tank. The grout was continuously recirculated for approximately 25 minutes. Mud balance tests were performed at 5-minute intervals during recirculation of the grout. After 25 minutes, additional water was added to the grout with a short remixing time. Test measurements were repeated for each water level.

Results and Discussion

Raw data from the in-line meter is easily exported to spreadsheet software for analysis and graphing. **Figure 2** shows plots of the in-line meter data (data points are so close together that they appear as the green line in the plot) as well as the results from the mud balance test. Mud balance data are shown in red with a bar indicating the amount of data spread between all readings taken at a given water content. The figure shows correlation between the mud balance test results and the in-line density meter readings, with distinct changes in density as water is added. The short downward spike of the in-line meter data between each water content change is due to a brief shutoff of the pump as the grout was transferred to the mixer and then back to the holding tank. As is shown in the figure, density changes are relatively small for significant changes in water content. The mud balance test has accuracy limitations in these ranges, and these limitations can be compounded by operator error or by acquiring a sample that is not representative of the batch. The in-line density meter provides accurate results and continuous monitoring of the grout as it is pumped into the tendon. **Figure 3** shows the temperature readings of the same grout mixture during pumping. Temperature data are captured by the in-line meter at the same time as each density reading.

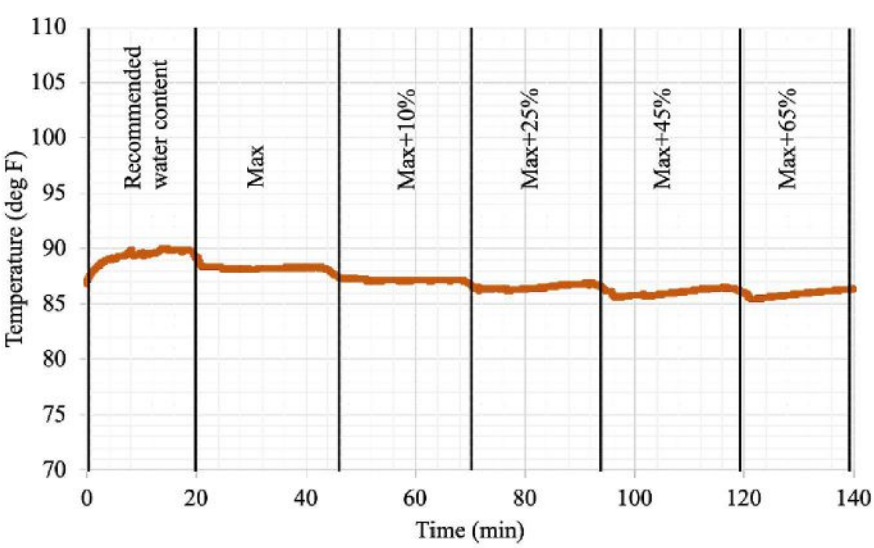


Figure 3. Temperature readings from in-line Coriolis-based flow meter corresponding to grout density results shown in Fig. 2. Figure: Andrea J. Schokker, adapted from *PTI Journal* with permission.

Summary and Recommendations

In this test, the in-line density meter (based on the Coriolis principle) showed clear potential for use in the field pumping of PT grout. Both the density and temperature data provide an electronic record of grouting from start to finish. Issues related to grout consistency (poor mixing and clumping), water trapped in the hose or pump piping, and other inconsistencies can be captured in data from the in-line meter. The use of an additional meter at the outlet would provide indications of water or debris that might be trapped in the tendon duct.

References

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2. American Petroleum Institute (API). 2015. *ANSI/API Specification 10A: Specification for Cements and Material for Well Cementing.* Washington, DC: API.
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Sweep in Precast, Prestressed Concrete Bridge Girders—Part II

by Dr. Bruce W. Russell, Oklahoma State University

This article is the second in a series of three articles on sweep, or bowing, in prestressed concrete bridge girders. The first article focused on potential causes of sweep and provided recommendations for actions that can be taken at the prestressing plant to mitigate its effects (see the Spring 2019 issue of *ASPIRE*®).[1] This article addresses the effects of sweep during the lifting, transportation, and erection of girders and describes actions that can be taken to increase factors of safety for lifting and hauling. The third article will focus on whether permanent sweep in girders should cause concern for long-term stresses, strength, or performance of bridges.

Design Example Describing the Effects of Sweep

To provide an example for discussion, a PCI-American Association of State Highway and Transportation Officials (AASHTO) bulb-tee BT-72 girder with a span length of 130 ft 6 in. is used. The girder cross section and material properties, prestressing, and initial camber are listed in **Table 1**. For this example, the girder is assumed to have a midspan sweep *f* of 1.631 in., which is equal to the PCI recommended sweep tolerance of ⅛ in. per 10 ft in length.[2] Calculations in this article assume that the girder is being lifted at the time of transport, so later-age material properties are used. Although the effect of wind loads is considered in PCI's *Recommended Practice for Lateral Stability of Precast, Prestressed Concrete Bridge Girders*,[3] it is not considered in this example.

Table 1. Girder Design Details for Example

Girder Design:	Bruce W. Russell, PhD, PE	
Length:	130.5 ft	
Type of Girder:	PCI-AASHTO BT-72	
A =	767	In.^{2}
h =	72.0	in.
c_b =	36.6	in.
c_t =	35.4	in.
I_{xx} =	545,113	in.^{4}
I_{yy} =	37,543	in.^{4}
S_t =	15,420	in.^{3}
w_b =	0.799	kip/ft
f'_c =	10.0	ksi
f'_{ci} =	7.0	ksi
E_{ci} =	4820	ksi
E_c =	5760	ksi
Width of bottom flange	26	in.
Width of top flange	42	in.
N =	38	0.6-in.-diameter Gr. 270 ASTM A416, low-relaxation strands
Strand eccentricity =	29.02	in.
Δ_{camber} =	2.72	in. ↑ estimated at time of handling and includes girder self-weight

Lifting Girders with Sweep and the Calculation of Secondary Effects Related to Roll Equilibrium

When a bridge girder exhibits sweep, the center of mass of the girder (CG) is eccentric to the roll axis.[3] While other effects can also contribute to this eccentricity, only sweep will be considered in this article. A generalized representation for sweep is shown in **Fig. 1**, where the sweep at midspan is defined by the symbol *f*. The initial eccentricity e_i of the center of mass of the girder with sweep, which can be modeled as a circular arc, is located at approximately ⅔*f* from the roll axis of the girder and given by:

$$e_i = \frac{2}{3}f = \frac{2}{3}(1.631\text{ in.}) = 1.087\text{ in.}$$

If we assume that the girder is lifted at its ends to simplify analysis and is lifted at the top surface (lifting loops are assumed to be flexible, so y_{lift} = 0 in **Fig. 2**), then y_r, the height of roll axis above the CG of the hanging girder, is defined as[4]:

$$y_r = c_t - 0.64\Delta_{camber} = 35.4 - (0.64)(2.72)$$

$$= 33.66\text{ in.}$$

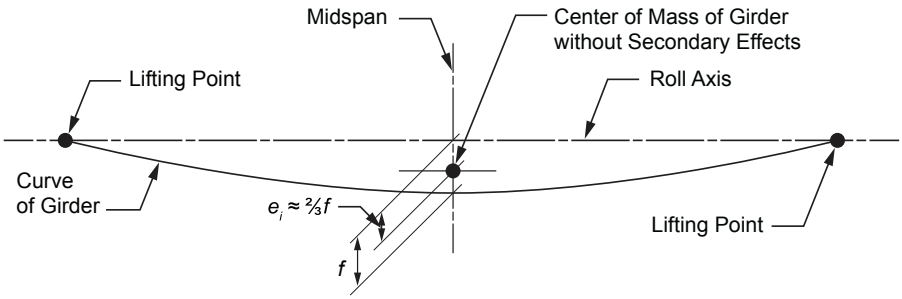

Figure 1. Schematic plan view of girder with sweep when supported at its ends. Figure: PCI.

where

c_t = distance from the CG to the top surface of the girder

Δ_{camber} = girder camber at midspan including self-weight

0.64 = coefficient that accounts for the CG of a polynomial shape describing the cambered girder

Once the girder is lifted, the initial angle of inclination θ_i (using small-angle theory [θ < 0.400 rad] and not considering any secondary effects), as shown in **Fig. 2**, is given by:

$$\theta_i = \tan\theta_i = \frac{e_i}{y_r} = \frac{1.087 \text{ in.}}{33.66 \text{ in.}} = 0.0323 \text{ rad}$$

When lifted, the girder rotates by an angle θ_i, causing a secondary effect—bending about its weak axis. This effect results in additional eccentricity caused by the deflection of the self-weight component of the girder bending about its weak axis and more rotation. The final eccentricity e is:

$$e = e_i + 0.64\left(\frac{5w_bL^4}{384E_cI_{yy}}\right)\sin\theta$$

After tilting, the girder's final angle θ at its "roll equilibrium" is derived directly from equilibrium and is expressed by:

$$y_r \tan\theta = y_r \tan\theta_i + z_0 \sin\theta$$

The term z_0, which is the lateral deflection of center of gravity of the girder with the full self-weight applied laterally, is computed by the following equation (see explanation in Mast[4]):

$$z_0 = 0.64\left(\frac{5w_bL^4}{384E_cI_{yy}}\right) = \frac{w_bL^4}{120E_cI_{yy}}$$

$$= \frac{\left[\frac{(0.799 \text{ kip/ft})}{(12 \text{ in./ft})}\right]\left[(130.5 \text{ ft})(12 \text{ in./ft})\right]^4}{120(5760 \text{ ksi})(37{,}543 \text{ in.}^4)}$$

$$= 15.43 \text{ in.}$$

By rearranging these equations and using small-angle theory, the angle of inclination at roll equilibrium can be determined by the following equation[4]:

$$\theta = \theta_i + \frac{z_0}{y_r}\theta$$

Solving for θ yields:

$$\theta = \frac{\theta_i}{\left[1 - \left(\frac{z_0}{y_r}\right)\right]}$$

The equation shows that the final angle of inclination is dependent solely on the initial angle of inclination θ_i, whether caused by sweep or other imperfection, and the ratio of z_0 to y_r. Furthermore, the inverse of the denominator quantifies directly the magnification of the initial angle of inclination.

For the BT-72 example girder, the rotation at roll equilibrium is:

$$\theta = \frac{0.0323 \text{ rad}}{\left[1 - \left(\frac{15.43 \text{ in.}}{33.66 \text{ in.}}\right)\right]} = 0.0596 \text{ rad}$$

Table 2 shows the same calculations for varying amounts of sweep from no sweep to a sweep equal to twice the sweep. The design example is represented by the highlighted row, where sweep f is 1.631 in. and the final roll angle θ is 0.0596 rad (which, for reference, equals 3.4 degrees of tilt).

Roll Stability

Roll stability of girders directly impacts the lifting, transportation, and erection of prestressed bridge girders. It should not be ignored or omitted in a discussion regarding treatment of girders with initial imperfections such as sweep because these imperfections are multiplied by the effects of girder instability. Roll instability is created when the deflection of the center of mass of the girder bent about its weak axis z_o is greater than the distance from the CG of the girder to the roll axis y_r.

A factor of safety (FS) against roll instability becomes an important concept when discussing improvements to lateral stability when lifting, hauling, and transporting girders. For the example BT-72 girder, which is 130.5 ft long but without initial sweep or other

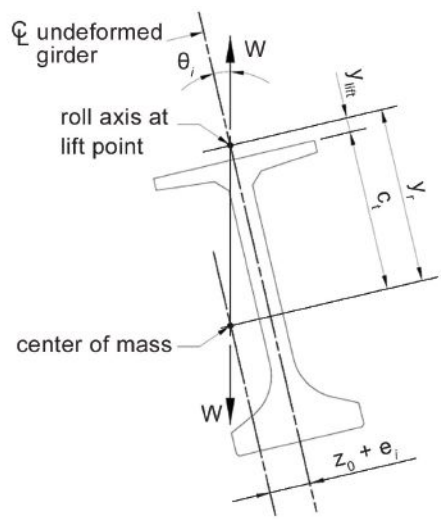

Figure 2. Schematic end view of rotated girder with sweep when lifted. Figure: PCI.

Table 2. Effects of Varying Sweep when Lifting for Example Girder

f	f, in.	e_i, in.	θ_i, rad	θ, rad	FS_θ
0.00	0.000	0.000	0.0000	0.0000	2.18
0.50 f_{tol}	0.816	0.544	0.0162	0.0298	1.89
1.00 f_{tol}	1.631	1.087	0.0323	0.0596	1.60
1.50 f_{tol}	2.447	1.631	0.0485	0.0895	1.31
2.00 f_{tol}	3.263	2.175	0.0646	0.1193	1.02

Notes:

1. Calculations are based on a BT-72 with length L = 130.5 ft.
2. Sweep tolerance f_{tol} = 1.631 in. is computed from a PCI standard practice[1] where sweep tolerance is ⅛ in. per 10 ft of length.
3. FS_θ is the factor of safety against cracking and is computed from θ_{max} = 0.1218 rad, which is the angle of tilt that is expected to cause cracking.

imperfections, and therefore has no secondary effects, the *FS* is:

$$FS = \frac{y_r}{z_0} = \frac{33.66 \text{ in.}}{15.43 \text{ in.}} = 2.182$$

Any steps to improve the roll stability of a girder without sweep—such as increasing y_r by raising the lifting point above the top of the girder or increasing the moment of inertia in the weak direction (see the Creative Concrete Construction article on long girders in this issue of *ASPIRE*)—help mitigate the secondary effects of sweep. Note that typical lifting loops are flexible, and the stability of the girder is not improved by increasing the length or height of typical lifting loops. The factor of safety against roll instability is not affected by initial sweep because the girder's theoretical roll stability is independent of initial imperfections.

Additional Stresses and Possible Cracking Caused by Sweep in Long Girders

As the girder is lifted, sweep will cause the girder to tilt, which creates new stresses from bending about its weak axis that increase the tensile stress at one corner of the girder's top flange. Considering this added stress and typical patterns of prestressing, the top of a girder is more likely to be affected by cracking than the bottom. Small-angle approximations are used to simplify the equations. The additional stresses (tension is positive) are computed as follows:

1. Calculate the top fiber stress considering the girder without tilt. For this computation, it is estimated (from normal design procedures) that the effective prestressing force at erection F_{se} is 1197 kip after all losses, and that the eccentricity of the prestressing force e_{ps} is 29.02 in. For purposes of this article, $f_{t,m}$ is defined as the stress at the top fiber at midspan due to prestressing and self-weight:

$$f_{t,m} = -F_{se}\left(\frac{1}{A} - \frac{e_{ps}}{S_t}\right) - \frac{M_B}{S_t}$$

$$f_{t,m} = -1197 \text{ kip}\left(\frac{1}{767 \text{ in}^2} - \frac{29.02 \text{ in.}}{15{,}420 \text{ in.}^3}\right)$$

$$-\left(\frac{(1700.9 \text{ kip-ft})(12\text{in./ft})}{15{,}420 \text{ in}^3}\right) = -0.632 \text{ ksi}$$

where

M_B = the self-weight moment of the girder at midspan when supported at the ends = 1700.9 kip-ft

2. Calculate the concrete stress at the extreme fiber caused by bending about the weak axis. This is the additional stress that can be anticipated when the girder with sweep is lifted by the lifting loops. For this article, $f_{t,\theta}$ is the stress at the outside edge of the top fiber due to weak axis bending:

$$f_{t,\theta} = \pm\left(\frac{M_B\theta}{I_{yy}}\right)\left(\frac{b_f}{2}\right) =$$

$$\pm\left[\frac{(1700.9 \text{ kip-ft})(12 \text{ in./ft})(0.0596 \text{ rad})}{37{,}543 \text{ in.}^3}\right]$$

$$\times\left(\frac{42 \text{ in.}}{2}\right) = \pm 0.680 \text{ ksi}$$

3. The calculated tension (or compression) can be determined by adding the two computed stresses together. In this example, the top corner of the BT-72 will have a combined stress of 0.680 ksi – 0.632 ksi = 0.048 ksi (tension). In this particular case, the sweep in the girder coupled with secondary effects creates only a slight tension in the top corner of the BT-72. However, these calculations should be performed for other locations along the length of the girder. For harped strand designs, the harp location should be checked. A check should also be made of maximum compressive stresses at the bottom fibers.

When Is Sweep a Serious Consideration?

To summarize the conditions computed for the example, the initial sweep was taken to be equal to the sweep tolerance of 1.631 in. These results demonstrate that the additional roll from sweep is unlikely to cause cracking in the girder. Therefore, the engineer could, and perhaps should, conclude that the 1.631 in. of sweep is acceptable when lifting and handling the girder, and similar computations can be made by the specialty engineer for each case where an owner or other stakeholder raises concerns about observed sweep.

While the reduced cross-sectional properties associated with cracking of the prestressed concrete bridge girder are considered in both Mast[5] and the PCI Recommended Practice,[3] this author recommends that cracking be used as a performance limit for stability calculations. This can be implemented by computing the rotation θ_{max}, which is the tilt angle at which cracking is expected. A suitable factor of safety similar to that described in the PCI Recommended Practice can be applied to θ_{max}.

Cracking in the top flange of the girder can be avoided by satisfying the following equation, where cracking is assumed to occur when the tensile stress in concrete is equal to the modulus of rupture from the *AASHTO LRFD Bridge Design Specifications*[6] Article 5.4.2.6, which is $0.24\lambda = \sqrt{f'_c}$. Note that compressive stresses are taken as negative, f'_c is measured in ksi, and λ is taken as 1.0 for normalweight concrete.

$$f_{t,m} + f_{t,\theta} \le 0.24\sqrt{f'_c} = 0.24\sqrt{10 \text{ ksi}} = 0.759 \text{ ksi}$$

For the example girder, the 0.048 ksi tension computed earlier satisfies this inequality.

The inequality can also be used to determine θ_{max}, the tilt angle at which cracking would theoretically occur, by substituting θ_{max} into the expanded form of the equation as follows:

$$\left[-F_{se}\left(\frac{1}{A} - \frac{e}{S_t}\right) - \frac{M_B}{S_t}\cos\theta_{max}\right]$$

$$+\left[\frac{M_B \sin\theta_{max}}{I_{yy}}\left(\frac{b_f}{2}\right)\right] \le 0.24\sqrt{f'_c}$$

Using small-angle approximations of $\cos\theta_{max} = 1$ and $\sin\theta_{max} = \theta_{max}$, and substituting the values for $f_{t,m}$ and $f_{t,\theta}$ the equation can be solved for θ_{max}:

$$\theta_{max} = \frac{\left[F_{se}\left(\frac{1}{A} - \frac{e}{S_t}\right) + \frac{M_B}{S_t}\cos\theta_{max}\right] + 0.24\sqrt{f'_c}}{\left[\left(\frac{M_B}{I_{yy}}\right)\left(\frac{b_f}{2}\right)\right]}$$

$$\theta_{max} = \frac{-f_{t,m} + 0.24\sqrt{f'_c}}{\frac{f_{t,\theta}}{\theta}}$$

$$= \frac{\left[-(-0.632 \text{ ksi}) + 0.759 \text{ ksi}\right]}{\frac{0.680 \text{ ksi}}{0.0596 \text{ rad}}}$$

$$= 0.1218 \text{ rad}$$

From this result, the eccentricity of the CG of the girder associated with cracking and, by inference, the sweep that would be expected to cause cracking can be computed (again, note that compressive stresses are negative):

$$\theta_{i,\max} = \theta_{\max}\left(1 - \frac{z_0}{y_r}\right)$$

$$= 0.1218 \text{ rad}\left[1 - \left(\frac{15.43 \text{ in.}}{33.66 \text{ in.}}\right)\right]$$

$$= 0.06629 \text{ rad}$$

$$e_{i.max} = y_r\theta_i = 33.66 \text{ in.}(0.06629 \text{ rad})$$

$$= 2.231 \text{ in.}$$

$$f_{max} = \left(\frac{3}{2}\right)e_i = \left(\frac{3}{2}\right)2.231 \text{ in.} = 3.346 \text{ in.}$$

These simple calculations show that a BT-72 with a length of 130.5 ft could possess a sweep at midspan prior to lifting of 3.346 in. without cracking when lifted by rigging located at the girder's ends. Note that this sweep of 3.346 in. represents more than twice the sweep tolerance of 1.631 in. Most importantly, these computations show that girders with sweep exceeding the tolerance published by PCI[2] may be lifted and erected without damage to the bridge girders. By way of comparison to standard practice, the 2014 *PCI Bridge Design Manual*[7] states that sweep equal to one-half the tolerance should be used during girder design for considering the effects of lifting; however, for many cases, it is apparent that sweep equal to or exceeding the published sweep tolerance can be accepted.

Maximum Tilt Angle and Factors of Safety

For the purpose of this article, we have adopted a factor of safety that assigns θ_{max} to be equal to the tilt angle that causes cracking. Accordingly, the term FS_θ is used in this article to distinguish it from the factor of safety related to roll instability FS. The limiting value of FS, or the critical ratio as defined by Mast,[4] is computed by:

$$\left(\frac{y_r}{z_0}\right)_{cr} = \frac{1}{\left(1 - \frac{\theta_i}{\theta_{max}}\right)} = \frac{1}{\left[1 - \left(\frac{0.0323}{0.1218}\right)\right]} = 1.361$$

The FS_θ against cracking for this example is computed using this equation, which is equivalent to one developed by Mast[5]:

$$FS_\theta = \frac{\frac{y_r}{z_0}}{\left(\frac{y_r}{z_0}\right)_{cr}} = \frac{FS}{\left(\frac{y_r}{z_0}\right)_{cr}} = \frac{2.182}{1.361} = 1.600$$

Table 2 shows computations for FS_θ for varying amounts of sweep. The design example with sweep equal to the sweep tolerance is included with its FS_θ equal to 1.600, which corresponds to the computation shown in this article. The changes in FS_θ as initial sweep increases are also reported, showing that as the initial sweep increases, the likelihood of cracking the beam upon lifting also increases. Nevertheless, the results in **Table 2** show that a BT-72 girder with sweep equal to twice the tolerance can be lifted safely and, theoretically, without the girder cracking.

Note that the calculation of FS_θ is dependent on both the initial sweep and secondary effects. While analysis in this article is limited to uncracked sections, industry experience, and testing performed by Mast[5] demonstrate that precast, prestressed concrete girders can provide additional safety for roll or tilt beyond that computed for first cracking. The engineer should work with the owner and contractor to determine acceptable factors of safety.

Considerations for Hauling Girders

The transportation of bridge girders requires special attention or special considerations when transportation involves superelevation and curvature of roadways, which is almost always the case. During transport, girders are supported at the bottom on specialized hauling equipment that has different equilibrium conditions and resists rotation differently than when girders are lifted. Analysis methods developed and presented in Imper and Laszlo,[8] Mast,[5] and the PCI Recommended Practice[3] can be used to evaluate the effects of stability during the transportation of a girder. An article by Brice, Khaleghi, and Seguirant[9] provides a detailed design example that gives an approach for incorporating lateral stability considerations into the design of a bridge girder rather that checking lateral stability after the design is completed. Information on actual girder sweep, initial imperfections, and hauling equipment are required for the analysis. The reader is referred to these other references for details on the inputs required and analysis methods used for stability of girders during transportation. The differences in analysis apply, however, whether the bridge girders exhibit sweep or not.

The standard of care for girder transportation calls for the bracing of the ends of the girder during transportation and secure attachment of the girder to the truck, tractor, or hauling platform. Three of the four stability improvement strategies described in the following section can be used to mitigate the effects of sweep in girders during transport.

Mitigating Effects of Sweep During Lifting, Transportation, and Erection

There are four primary ways to decrease the effects of sweep and other imperfections that affect the lifting and transportation of prestressed concrete bridge girders:

- Move the lifting location toward the center of the girder.
- Increase the height of the lifting point.
- Incorporate fully tensioned top strands.
- Provide bracing along the length of the bridge girder.

The following discussion can be augmented by the works by Mast[4,5] and Imper and Laszlo[8].

Move the Lifting Location Toward the Center of the Girder

The engineer and owner should be concerned about adding tension to the top of the girder during lifting and erection, but they must address this concern while also considering the need to lift and transport the bridge girder safely and without damage. Computations for this BT-72 girder show that by moving the lifting locations inward by 6% of its length, the magnification of the tilt angle is reduced by 50% and factors of safety are increased.

Mast[4] also discusses the effect of moving the lifting locations. His calculations demonstrate that moving the lifting location as mentioned above for the BT-72 will give the same result for any type of girder and will also double the factory of safety. This method must be

Figure 3. Example of extended rigid lifting system. Photo: The Lane Construction Corporation.

considered during initial design because lifting devices must be located during fabrication and the design must consider additional tension in the top of the girder.

Increase the Height of the Lifting Point

This technique requires a frame sufficiently stiff (that is, rigid) to raise the hinge point for lifting above the top of the cross section. The frame must maintain its horizontal location with respect to the girder cross section during lifting. The frame will increase y_i, the distance between the roll axis and the centroid of the girder.

The photograph shown in **Fig. 3** shows one method that can be used to raise the lifting point for a long bridge girder. Other methods are available. **Table 3** reports the results of computations where the girder is lifted by rigid devices that have pick points at varying heights above the top of the cross section. All calculation results in **Table 3** are based on an initial sweep equal to the sweep tolerance of 1.631 in. **Table 3** shows that by raising the lifting point (hinge point) for a BT-72 from the top of the girder cross section to a height 24 in. above it, the *FS* against roll instability increases from 2.18 to 3.74 and the FS_θ increases from 1.60 to 3.16. The tabulations demonstrate that raising the lifting point is an effective means of improving girder stability for a girder that exhibits sweep.

Incorporate Fully Tensioned Top Strands

These strands can be internal and fully bonded as shown by Russell,[1] internal and debonded over the center portion of the girder (temporary top strands), or external and not bonded. The fully tensioned top strands increase precompression in the top fibers of the girder and effectively improve the girder's resistance to cracking if and when tilting occurs.

Analysis results in **Table 4** indicate modest improvements in FS_θ for the example BT-72 by adding fully tensioned top strands 4 in. below the top of the girder. In the event that cracking occurs at the top fibers of a girder, for any reasons, fully tensioned top strands effectively limit both the width and number of cracks that may occur.

Provide Bracing Along the Length of the Bridge Girder

Several bracing arrangements can be employed, such as a stiffening truss connected to the top or bottom flange of the girder or the web, or a post-tensioned king- or queen-post arrangement. Mast[4] also discusses the issue of king posts for bracing girders.

A king-post arrangement is shown in **Fig. 4**.[8] In this figure, the king post supports fully tensioned strands as outriggers that effectively add stiffness to the cross section and resist lateral bending. Note that for the arrangement shown, strands are not required to be fully tensioned. Instead it is sufficient to post-tension external tendons to the level where they remain in tension through all possible girder deformations.

For girders with sweep, a fully tensioned, one-sided king-post arrangement can be employed to help straighten the girder and reduce the amount of sweep.

Table 5 includes computations for the BT-72 example girder where a king-post with external post-tensioning is provided on one side only. The table shows the effects of adding external tendons in a king-post arrangement where the post deflects the tendons 36 in. Increasing the number of tendons provides a modest increase in lateral (weak-axis) stiffness. This has the direct effect of improving the roll stability of the girder, which is shown in the table as increasing *FS*. Perhaps more important in this discussion about lifting and transporting girders with sweep is the computation showing the straightening effect of

Table 3. Effects of Raising the Lifting Point Above the Top of the Example Girder Using a Rigid Frame

Distance Above Top Fiber, in.	y_i, in.	$FS = y_i/z_o$	θ_i, rad	θ_{max}, rad	FS_θ
0	33.66	2.18	0.0323	0.1218	1.60
4	37.66	2.44	0.0289	0.1218	1.86
8	41.66	2.70	0.0261	0.1218	2.12
12	45.66	2.96	0.0238	0.1218	2.38
16	49.66	3.22	0.0219	0.1218	2.64
20	53.66	3.48	0.0203	0.1218	2.90
24	57.66	3.74	0.0189	0.1218	3.16

Table 4. Effects of Fully Tensioned Top Strands (Internal or External) Located 4 in. Below the Top of the Example Girder

No. Top Strands	e_{top}, in.	ΔF_{pe}, kip	Δf_{top}, ksi	$f_{t,m}$, ksi	θ_i, rad	θ_{max}, rad	FS_θ
2	-31.40	0.0	0.000	-0.632	0.0323	0.1218	1.60
4	-31.40	62.9	-0.046	-0.678	0.0323	0.1258	1.62
6	-31.40	125.9	-0.093	-0.724	0.0323	0.1299	1.64
8	-31.40	188.8	-0.139	-0.770	0.0323	0.1340	1.66

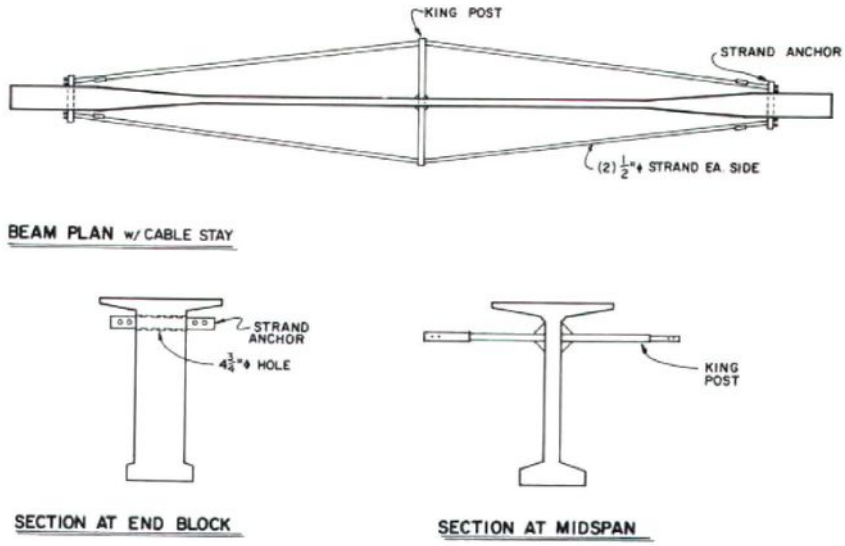

Figure 4. Example of king post bracing system used for shipping long-span bridge beams. Figure: Reference 8.

eccentric post-tensioning of the king post. The king-post arrangement with four strands tensioned to 30 ksi deflects the center of the girder 0.89 in., which is a sizable amount of the original sweep used in this example. Moreover, largely because of the straightening effects, the factor of safety against cracking FS_θ is increased from 1.60 (without the king post) to 2.35 with eight 0.6-in.-diameter strands tensioned to 30 ksi. These computations assume the external post-tensioned strands are located at the vertical centroidal axis, so the benefit of increasing top fiber precompression is only a result of axial compression.

The king post is effective in accomplishing two important things at once: increasing the *FS* against roll instability and cracking and removing sweep from the girder during transportation and erection.

Conclusion

In most bridge girders, sweep, if it exists, is likely to remain small and unnoticed. However, as longer bridge girders are fabricated, sweep may become more noticeable. This article presents simple calculations that can be performed by the specialty engineer and shows that the angle of inclination, or the angle for roll equilibrium, can increase dramatically for longer girders. The calculations demonstrate that for girders possessing modest amounts of sweep within the tolerance, the lifting and transportation of the bridge girders can usually be performed safely and without damage to the girder. If the design of the girder considers an allowance for sweep, it is unlikely that a problem will arise during lifting, transporting, and erection. Where measured sweep exceeds the PCI tolerance, engineers and owners will want to retain a qualified engineer to help with rigging, transportation, and erection.

Table 5. Increasing Lateral Stiffness and Straightening the Girder with a One-Sided, External King Post at a 36 in. Standoff

No. of External Strands	A_{sx}, in.^{2}	Lateral Stiffness Factor	z_0, in.	*FS*	F_{px}, kip	Δ_x, in.	θ_i, rad	FS_θ
0	0	1.00	15.43	2.18	0.00	0.00	0.0323	1.60
1	0.217	1.01	15.26	2.21	0.60	0.22	0.0281	1.69
2	0.434	1.02	15.09	2.23	1.20	0.44	0.0239	1.79
4	0.868	1.05	14.76	2.28	2.39	0.89	0.0155	1.99
6	1.302	1.07	14.45	2.33	3.59	1.33	0.0070	2.19
8	1.736	1.10	14.16	2.38	4.79	1.77	-0.0014	2.35

Notes:
1. External 0.6-in.-diameter strands are tensioned to 30 ksi each.
2. The lateral stiffness factor reflects the relative stiffness at midspan of the girder with the king post to the bare girder.
3. F_{px} is the lateral force transmitted to the bridge girder at the midpoint through the king post.
4. The straightening effect is the computed lateral deflection Δ_x.

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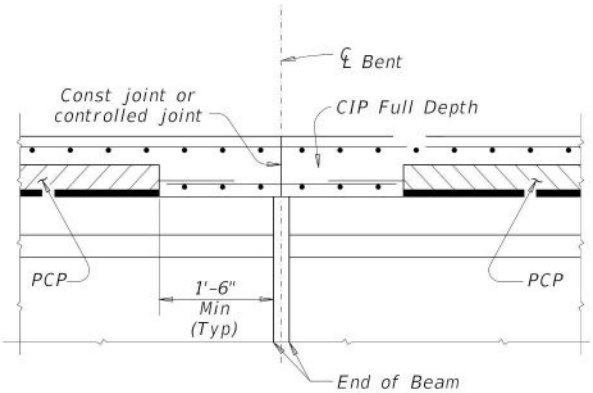
EDITOR'S NOTE

Some of the approach and notation in this article differ from those in the PCI Recommended Practice.[3] This article will be considered by the PCI committees as they develop the 2nd edition of the Recommended Practice. While the author proposes a different factor of safety against cracking, the PCI Recommended Practice also requires consideration of the factor of safety against failure.

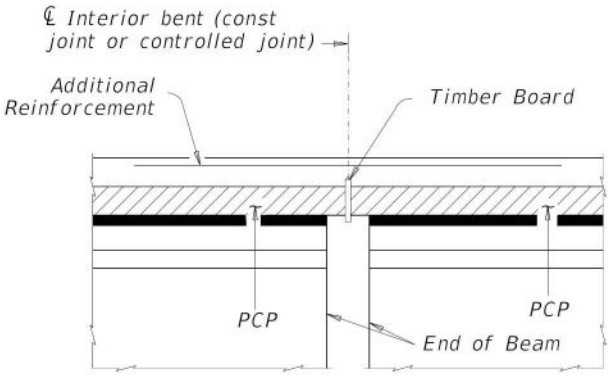
After the original article was published an error was found in the calculations for the girder braced using a king post. The author has revised the text in the first column on page 43 and the values in Table 5. The revised values do not alter the author's conclusion.

Simplifying Bridge Deck Construction in Texas

by John Vogel, Harris County Toll Road Authority; Kevin Pruski, Texas Department of Transportation; and John Holt, Modjeski and Masters



Traditional Texas Department of Transportation detail for termination of a prestressed concrete panel (PCP) used as a stay-in-place deck form at the interior support of a multiple simple-span unit. Figure: Texas Department of Transportation.



New Texas Department of Transportation detail for termination of a prestressed concrete panel at the interior support of a multiple simple-span unit. Figure: John Vogel.

Texas Department of Transportation (TxDOT) bridge deck construction heavily relies on 4-in.-thick prestressed concrete panels (PCPs) as stay-in-place (SIP) forms between prestressed concrete girders. Cast-in-place (CIP) concrete is later placed, and the PCPs become part of the bridge deck's structural system. TxDOT allows PCPs as a contractor option; their use is not tracked, but they are usually used whenever possible, which clearly suggests their efficacy.

Until a few years ago, PCPs were held back a nominal distance from all beam ends (with or without expansion joints), creating "boundaries" of full-depth CIP concrete deck that needed to be formed with SIP metal forms or timber forms.

Galveston Causeway

In some circumstances, TxDOT restricts the use of SIP metal decks forms for this closure at the beam ends and instead dictates the use of conventional forming, which is tedious and poses scheduling and safety complexities. Such was the case for the Galveston Causeway, a large bridge replacement project near Houston. TxDOT and the contractor worked together, with the aid of a TxDOT research program, to extend the panels from end to end of the span, thus eliminating the need for any conventional deck formwork between the beams. This subtle yet substantial change has become the new go-to option for bridge deck construction in Texas.

The Galveston Causeway carries Interstate 45 and connects Galveston Island with the Texas mainland. This bridge was designed in 2003 using AASHTO-type prestressed concrete girders and the standard PCP forming system for 7852 ft of approach spans. TxDOT's detailing practice required full-depth CIP thickened slabs at the ends of each span. SIP metal-form standard details were omitted from the plans for this area of the deck as part of the durability design of the bridge, but they were not explicitly prohibited. However, the contractor intended to use SIP forms because barge access in shallow water was limited and demolishing wood forms from an under-bridge platform for a 148-ft-wide bridge would be difficult. Based on work done by TxDOT researchers, the contractor proposed the use of PCP forms to improve the existing TxDOT detailing practice for bridge decks. From a static load standpoint, the researchers confirmed the adequacy of the concept of using PCPs for the full length of the girders, so the plans were revised to use PCPs for the full length of each span.

Lessons were learned on the Galveston Causeway project as bridge construction progressed and details were modified to improve constructability and performance. In Texas, decks of simple-span bridges are placed continuously over interior bents, creating multispan continuous deck units, and the decks are expected to crack at the centerline of the bent. On the Galveston Causeway, various techniques to produce a neat crack were tried with mixed results. The first few deck units cast had jagged cracks at the interior bents. Ceramic tile backer

Old detail at the girder end showing the stay-in-place metal formwork on each side of the girder. Photo: John Holt.

board was used to form the gap over the beams where there were no panels, but no provisions except the use of preformed polyvinylchloride (PVC) crack inducer were made to force the crack.

Modifying Details

Details were revised to better align PCP joints over interior bents and keep the panel gap uniform across the bridge. Allowance of some gap between panels is necessary to align panels over the centerline of the bent. Additionally, the area between beam ends, where there are no panels, needs some type of vertical joint form to induce a crack at the desired location. Vertical and straight cracks are desirable to avoid spalling at the crack. A ¾-in.-thick lumber board was specified over the beams to match the panel gap at the centerline of the bent. The timber board is not required between the panels because backer rod or foam board would suffice, but full-width timber board would be acceptable. The contractor troweled a groove at the joint location to make it possible to install the PVC crack inducer in a neat, straight line directly over the panel gap, because large aggregate prevented good alignment of the joint former. The preformed crack inducer simply deformed around the aggregate because the concrete stiffens before the joint former can be installed. Cracks at interior bents were 0.016 in. wide without the crack former and 0.005 to 0.007 in. wide with the improved crack-control joint details.

On occasion, when the PVC crack inducer has been left out, contactors have used a saw cut as a "crack creator," which has not been successful. The saw cut cannot be made in a timely fashion, and, with age, the previously hidden cracks become visible, meandering back and forth across the saw cut, creating slivers of concrete that spall. Currently, the method outlined in the TxDOT standard is to use the PVC crack inducer at the top as well as a timber board that is glued on the vertical faces of the PCPs at the centerline of the interior bent and extends the full width of the bridge between edges of exterior beam flanges. Attached to the board is a timber strip with a triangular cross section to drive the crack from below.

Another lesson learned from the Galveston Causeway project is that there will be small gaps below the expansion joint rails that need to be plugged to contain the CIP concrete. A sheet metal angle was used to bridge the gap between the expansion joint rail and the PCP with the flat of the angle on top of the PCP. However, this detail breaks the bond between the panel and overlay, potentially inducing delamination. A better solution is to adhere sheet metal to the steel expansion joint flush with the edge of panel. TxDOT's expansion joint detail on its PCP standard drawing shows the PCP edge flush with the edge of the steel expansion joint. This is believed to be important to allow CIP concrete to fill the space and provide support for the steel expansion joint rail. Thus, placement of the end PCP at expansion joints requires precision—the contractor needs to plan the placement of the other PCPs to maintain the maximum allowable gap of 1 in. between any adjacent panels. When extending PCPs to the full length of the span, gaps become necessary to accommodate tolerances of prefabricated elements. It is not typical to seal the gaps between abutting panels, but filling/sealing is required when there is a significant gap. When CIP end closures are required, panel joints must be tight.

The use of PCPs for the full length of the girders on the Galveston Causeway project was a success that has been replicated elsewhere with other girder types or with skewed bents. The detail is now part of the TxDOT standard drawing for PCPs and has been widely embraced by Texas bridge contractors. The details can be found at https://www.dot.state.tx.us/insdtdot/orgchart/cmd/cserve/standard/bridge-e.htm.

John Vogel is the deputy engineering manager at the Harris County Toll Road Authority in Houston, Tex.; Kevin Pruski is the bridge construction and maintenance branch manager at the Texas Department of Transportation in Austin; and John Holt is the senior project manager/office manager at Modjeski and Masters in Round Rock, Tex.

New detail used at the Galveston Causeway project with PCPs extending to the end of the girder. The expansion joint rail at the end of a unit was installed abutting the edges of the PCPs. The last PCP was lowered—taking advantage of the increased haunch depth at the end of the girder used to accommodate the beam camber—to thicken the cast-in-place deck to add capacity and provide space needed for shear studs on the expansion joint. Photo: John Vogel.

A PROFESSOR'S PERSPECTIVE

Precast Bridge Studio Strengthens Connection

between Academia and the Bridge Industry

by Dr. Eric Matsumoto, California State University, Sacramento

One of the bridge community's most remarkable accomplishments is that design codes worldwide continue to incorporate rigorous provisions to prevent connection failures of concrete bridges, even for high-seismic regions. For the past two decades, academics and industry have partnered to develop innovative concrete connections, including those for precast concrete bridges under the banner of accelerated bridge construction (ABC).

During that same time, however, the connection between students and the concrete bridge industry has remained weak. While design professionals have justifiably obsessed over maintaining a continuous load path for our bridges, we have paid scant attention to ensuring continuity in the relationships between our students and industry. The consequence is a shortage of adequately prepared young engineers, construction managers, and others who can readily enter the workforce and contribute to its future. This issue is particularly disconcerting as we face ever-increasing challenges in the transportation sector at the same time that a large wave of Baby Boomers are transitioning into retirement, leaving the bridge industry without their wealth of experience.

Precast Bridge Studio students meet with bridge mentor, Bob Fish, T.Y. Lin International. Photo: Eric Matsumoto.

What can we do to improve this situation? Although there is no quick fix, ABC has shown us that success requires leaders, innovation, and, yes, precast concrete "connections"! For over a decade, the PCI Foundation (pci-foundation.org) has sponsored dozens of "precast studios" at universities as part of a curriculum development grant. These innovative immersion experiences in precast concrete connect students in architecture and other disciplines to the precast concrete industry. Many participating students have been inspired to enter the precast concrete industry and related industries.

Based on this successful track record, the PCI Foundation took a further step in fall 2018 to fund the first curriculum grant focused solely on bridges, dubbed the "Precast Bridge Studio" (PBS), at California State University, Sacramento (Sac State). This four-year grant has provided a new pathway for the precast concrete bridge industry to develop a long-term partnership with academia to directly support the education and training of the next generation of industry professionals and leaders. PBS aims to connect civil engineering and construction management students to members of the precast concrete bridge industry—including bridge designers, fabricators, construction managers, contractors, suppliers, and others—and thereby produce a new generation of students who are employable in the industry immediately upon graduation. Specific connections are made by joining industry to the students on campus and linking students to industry in the workplace.

During PBS's first semester, its students were immersed in the precast concrete bridge industry primarily through a precast concrete design class reorganized to focus on bridge structures rather than buildings. Fifteen civil engineering students were grouped into five-person teams, and each team was tasked with designing a 300-ft-long, multispan, precast, prestressed concrete girder bridge located in the Sacramento region. The teams had to adhere to American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications* and focused on either the California wide-flange girder, the bathtub (U) girder, or the I-girder. Deliverables consisted of plans, design calculations, hand checks, construction sequencing, working-day schedules, and cost estimates.

This task—a tall order for any group of civil engineering undergrads—was especially challenging because one-third of the students were simultaneously taking their first reinforced concrete design class and all students lost two weeks of the semester due to the northern California Paradise Fire smoke days. Nevertheless, students embraced the challenge, calling it "the experience of a lifetime." Teams were motivated and strengthened by continual engagement with industry, which included team mentoring by consultant bridge engineers from T.Y. Lin International, Mark Thomas & Co., and MGE Engineering and oversight by the California Department of

Transportation (Caltrans). In addition to learning bridge design and construction management, students treasured the life lessons derived from a realistic project with deliverables, office visits, and accountability to a practicing engineer.

Precast concrete fabrication was foremost in the minds of students during the entire process. In fact, they unanimously ranked the visit to the Con-Fab precast concrete plant in Lathrop, Calif., as the highlight event of the semester. One student recalled, "Lecture could not prepare me for the surreal experience of seeing the over 100-ft long bridge girders at the Con-Fab plant. It completely blew me away." The students were eagerly anticipating the plant tour because Con-Fab chief engineer, Brent Koch, gave a class lecture, "A Day in the Life of a Precast Plant," the previous day. During the visit, students directly experienced the magnitude and intricacies of bridge girders, details for composite action and continuity, deck panels, precast concrete pavement, steel forms that matched their own girder design, tensioning strand on a prestressing bed, materials testing, and much more.

During a tour of the Sumiden Wire plant in Stockton, Calif., the students also witnessed production of plain and epoxy-coated strand and tensile testing

The semester was filled with interaction with many industry members, including lectures by individuals from Dura-Stress on long-span girders, Viking Construction on precast concrete versus cast-in-place bridge construction and construction sequence and schedule,

A student team presents its final bridge design before the Precast Bridge Studio judging panel and audience. Photo: Mike Kealey, Owl Productions.

Precast Bridge Studio students learn to use bridge design software during a PGSuper™ workshop led by BridgeSight Software president, Richard Pickings. Photo: Eric Matsumoto.

and Caltrans on bridge design specifications and cost estimating. Additionally, students took part in two precast concrete bridge design software workshops that equipped them to perform bridge design using PGSuper Design Software I/II. To make the precast connection more "concrete," every classroom event or field trip included a 10-question quiz.

By the end of the semester, the teams were ready to showcase their hard work through a formal presentation, "Design and Construction of a Multispan Precast/Prestressed Concrete Bridge," before a packed audience, half of whom were bridge industry members. The industry mentors and supervising faculty served as the judging panel.

The atmosphere was electric. Teams were initially anxious to present before so many industry experts, but they eventually relaxed and ended up doing such a great job that bridge firms began to offer jobs to students that very night. One of the members in attendance, PCI Foundation Board of Directors member Glen Switzer of Durastress, noted, "Right now we have 15 students in one semester that can come into our industry and become a vital part, especially in the bridge arena. They're familiar with the technology advantages and already have the ability to design precast concrete bridges."

Because of an active PCI student club at the university, undergraduates who were not enrolled in the PBS class were still able to connect with the precast concrete industry. These students participated in PBS field trips and workshops, plus other events such as a new Precast Bridge Seminar series, which incorporated both civil engineering and construction management students—the first collaboration between these departments in the history of the college. Caltrans kicked off the series with "Accelerated Bridge Construction in California," and other industry partners presented at club meetings. Such events drew dozens of students because of the interesting topics, student officer enthusiasm, and, of course, lots of desserts and other treats.

The generous industry support of time and resources during fall 2018 has paved the way for further development of the PBS program for the next three years. In fall 2019, a new joint civil engineering/construction management bridge design and construction project is being offered, including engineering and construction mentors aided by assistant mentors.

With continued vision, guidance, and support from the PCI Foundation, PBS at Sac State can be a model for other universities to develop a strong, reliable "precast connection" between students and industry that will supply the fresh faces, innovative ideas, and vigor of a new generation needed for the growth of the precast concrete bridge industry.

EDITOR'S NOTE

Students compiled a 15-minute video capturing the excitement of their PBS experience, which is accessible at https://youtu.be/IKg1d-YEg1k.

A Synthesis Report on Concrete Bridge Shear Load Rating

by Dr. Lubin Gao, Federal Highway Administration

Of the 615,000 bridges in the National Bridge Inventory, approximately 42% are reinforced concrete and 25% are prestressed concrete structures, based on the material of the main span superstructure. Load rating concrete bridges for shear allows the owner to determine how to manage and operate them; however, doing this appropriately is challenging. One of the significant challenges is how to apply modern design standards to bridges that were designed to past design specifications.

History of Specifications

The history of bridge design standards includes both gradual evolution and periodic significant changes. In 1931, the American Association of State Highway Officials published the first edition of the *Standard Specifications for Highway Bridges* (ASD). In 1973, the organization, renamed the American Association of State Highway and Transportation Officials (AASHTO), incorporated load factor design (LFD) into the 11th edition of the specifications. In 1994, AASHTO published its first reliability-based load and resistance factor design (LRFD) bridge design specifications. In 2017, the 8th edition of the *AASHTO LRFD Bridge Design Specifications* was released.

Over this period, the design live loading, primarily truck loadings, has changed from the earliest notional truck loading, identified as H20 and H15 for most roadways or H10 for low-volume roadways, to the current HL-93 notional truck loading used in the AASHTO LRFD specifications. In addition, the design processes for application and distribution of live loads to supporting members have evolved from the simplified girder spacing–to–partial lane dimension ratio (S/D) to the current sophisticated distribution formulas in the AASHTO LRFD specifications.

Shear design provisions have also undergone significant changes through the years. **Table 1** summarizes those changes and the applicable

Table 1. Evolution of concrete shear design requirements in American Association of State Highway Officials (AASHO) and American Association of State Highway and Transportation Officials (AASHTO) bridge design specifications

Year	Notable Concrete Shear Design Change	Specifications
1931	Upper shear unit stress limits implemented.	*AASHO Standard Specifications*, 1st ed.
1935	Reinforcing steel and bond unit stress limits added.	*AASHO Standard Specifications*, 2nd ed.
1941	Shearing unit stress formula and required shear reinforcement formula introduced.	*AASHO Standard Specifications*, 3rd ed.
1944	Allowable concrete and bond unit stresses modified to be a function of the concrete ultimate strength.	*AASHO Standard Specifications*, 4th ed.
1961	Prestressed concrete design provisions using LFD methods introduced.	*AASHO Standard Specifications*, 8th ed.
1965	The required shear reinforcement formula adjusted to optimize reinforcement capacity at an angle of inclination of 45 degrees.	*AASHO Standard Specifications*, 9th ed.
1971	Minimum shear reinforcement area modified to accommodate high-strength (60 ksi) reinforcement.	Interim revisions to *AASHTO Standard Specifications*, 10th ed.
1979	Refined equations introduced to more accurately calculate shear stress in concrete members.	Interim revisions to *AASHTO Standard Specifications*, 12th ed.
1980	LFD shear design equation for prestressed concrete introduced.	Interim revisions to *AASHTO Standard Specifications*, 12th ed.
1989	Shear provisions for concrete segmental bridges introduced.	*AASHTO Guide Specifications for Design and Construction of Segmental Bridges*, 1st ed.
1994	Strut-and-tie method included in concrete bridge design specification.	*AASHTO LRFD Bridge Design Specifications*, 1st ed.
1998	Updated concrete shear strength equation provided.	Interim revisions to *AASHTO Manual for Condition Evaluation of Bridges, 2nd ed.*
2005	Segmental concrete bridge shear provisions adopted.	Interim revisions to *AASHTO LRFD Bridge Design Specifications*, 3rd ed.
2007	Simplified shear calculation procedure for prestressed and non-prestressed concrete sections provided.	*AASHTO LRFD Bridge Design Specifications*, 4th ed.
2008	Sectional design model revised to provide a noniterative method for the determination of β and θ factors.	Interim revisions to *AASHTO LRFD Bridge Design Specifications*, 4th ed.
2016	Strut-and-tie procedure modified.	Interim revisions to *AASHTO LRFD Bridge Design Specifications*, 7th ed.

specifications for changes in shear design of concrete bridges.

As a companion to its bridge design specifications, AASHTO has developed a bridge load rating manual, the *Manual for Bridge Evaluation* (MBE), which is now in its third edition. The current MBE includes the following provisions for shear load rating of concrete bridges:

Part 6A.5 addresses load and resistance factor rating for concrete structures:

- Article 6A.5.8—Evaluation for shear
- Article 6A.5.11—Rating of segmental concrete bridges
- Article 6A.5.11.4—Design-load rating
- Articles 6A.5.11.5.1—Service limit state legal load rating
- Article 6A.5.11.5.2—Service limit state permit load rating
- Article 6A.5.11.7—Evaluation for shear and torsion

Part 6B.5 addresses nominal capacity for allowable stress rating and load factor rating:

- Article 6B.5.2.4.3—Allowable stress method for concrete shear (diagonal tension)
- Article 6B.5.3.3—Load factor method for prestressed concrete

Rating a concrete bridge for shear is challenging and complicated because of the many changes to standards that have been made over time, as well as the multiple procedures in the current specifications. When an existing concrete bridge is load rated using a method that is different from the method used to design the bridge, it is difficult to assess the actual margin of safety.

Synthesis Report

In July 2017, the Federal Highway Administration (FHWA) started work to develop a synthesis report that compiles the technical aspects of shear load rating for concrete bridges that are difficult for owners to assess. A critical review of relevant specifications, technical literature, and transportation agency and industry practices and experiences was performed for the synthesis report. The review investigated the specifications and provisions related to shear design and shear load rating for concrete bridges specified in the current and previous editions of AASHTO design specifications and the MBE.

A survey of selected state departments of transportation was conducted to help define challenges that need to be addressed in the review. In the survey, states reported that the current general shear procedure using modified compression field theory (MCFT) in the AASHTO LRFD specifications occasionally gives a lower shear capacity for existing post-tensioned box girders than the procedure described in Section 5, Appendix B5, of the AASHTO LRFD specifications. Some states also found that shear may govern the load rating for some existing concrete bridges with no visible sign of shear distress. Another finding from the survey was that shear load rating for the HL-93 notional live load does not correlate with the shear load ratings for legal and permit loads. States reported that for some existing concrete bridges, shear load rating controls when applying the LFRD MCFT for permit loads, which may require more stringent restrictions. They also found that for certain legal loads, the shear load rating controls and a bridge may require load posting.

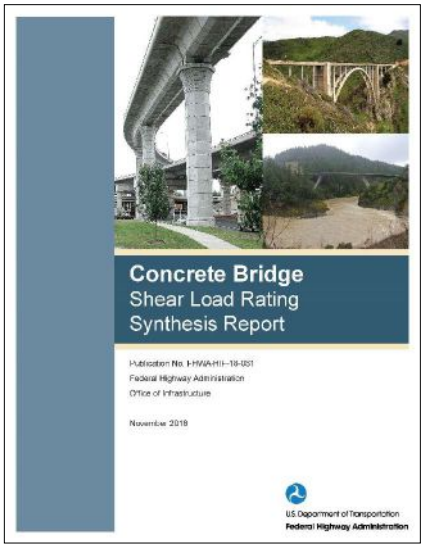
The final FHWA report,[1] which was published in November 2018, includes a history of changes in shear design provisions in AASHTO design specifications and the MBE. The report also summarizes states' practices and identifies the issues that states are facing in rating existing concrete bridges using the MBE shear provisions.

Further Work

Fifteen findings are identified and documented in the *Concrete Bridge Shear Load Rating Synthesis Report*.[1] Some could be addressed through modifications to the MBE or the AASHTO LRFD specifications. Other findings, such as the following, warrant further investigation or additional guidance:

- Finding 7: Correct estimation of the strain is critical for use in MCFT-based calculations.
- Finding 8: Using MCFT and the strain equations in LRFD when prestressing is on the compressive flexural side can provide incorrect and overly conservative shear strengths.
- Finding 9: MCFT-based shear strength calculations are load dependent.
- Finding 10: An existing girder not meeting minimum shear reinforcement requirements with MCFT may have its shear strength unduly penalized.
- Finding 12: Reinforcement detailing should be verified for adequacy when load rating concrete bridges in shear.

FHWA recently awarded a task order to develop improved concrete bridge shear load rating guidance and examples using the MCFT. The objectives of the work are to address the findings and pertinent recommendations in the report and provide training for bridge design and analysis engineers.



For example, based on the available shear test data set that is in the published literature,[2,3] the task-order contractor will develop methods for determining shear resistance for sections that do not meet minimum reinforcement requirements (either minimum longitudinal tension reinforcement or minimum shear reinforcement). The procedure will include how to accurately compute strain, resistance, concurrent force effects, and rating factors, including shear-moment interaction and consistency in shear resistance determination for strain and force effects. A minimum of three shear analysis examples will be developed to demonstrate the best approaches for shear assessment of concrete bridge members.

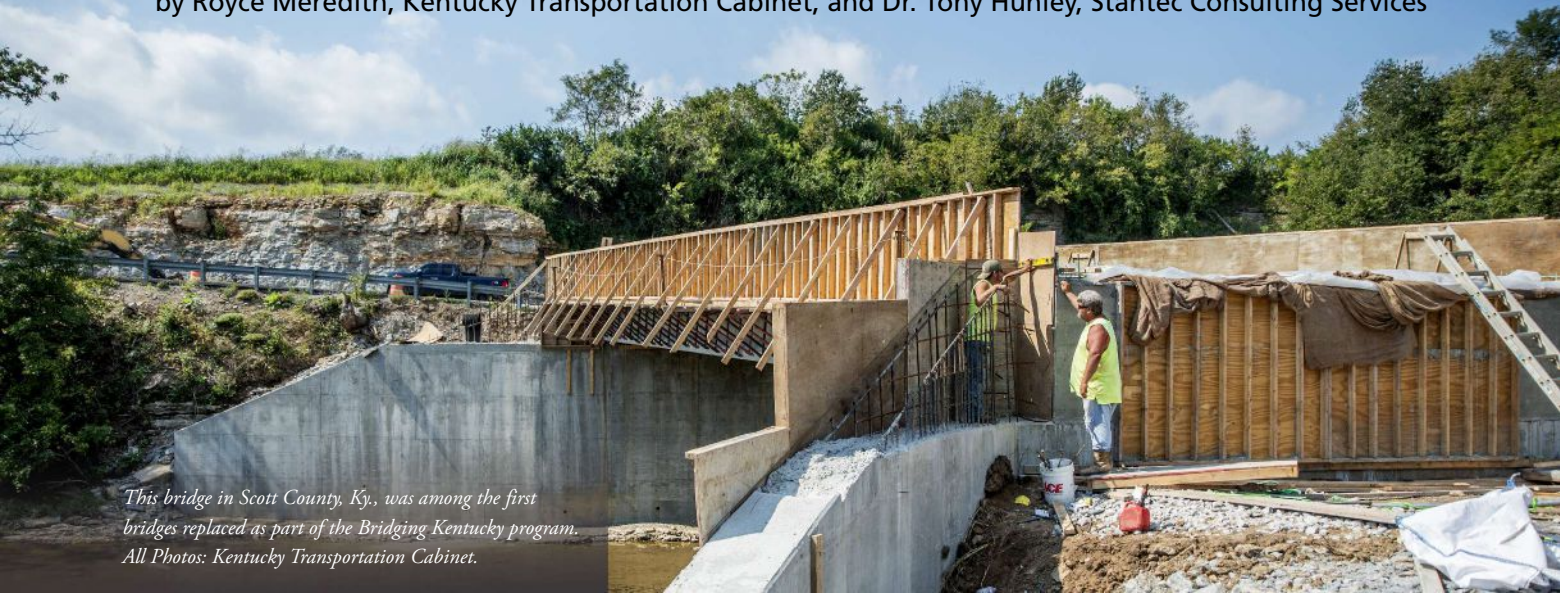
References

1. Holt, J., U. Garcia, S. Waters, C. Monopolis, A. Zhu, O. Bayrak, L. Powell, K. Halbe, P. Kumar, and B. Chavel. 2018. *Concrete Bridge Shear Load Rating Synthesis Report.* Publication No. FHWA-HIF-18-061. Federal Highway Administration, Washington, DC. https://www.fhwa.dot.gov/bridge/loadrating/pubs/hif18061.pdf.
2. Hawkins, N.M., D.A. Kuchma, R.F. Mast, M.L. Marsh, K-H. Reineck. 2005. *Simplified Shear Design of Structural Concrete Members*. National Cooperative Highway Research Program (NCHRP) Report 549. Washington, DC: Transportation Research Board. http://www.trb.org/Main/Public/Blurbs/156636.aspx
3. Hawkins, N.M., and D.A. Kuchma. 2007. *Application of LRFD Bridge Design Specifications to High-Strength Structural Concrete: Shear Provisions*. NCHRP Report 579. Washington, DC: Transportation Research Board. http://www.trb.org/Main/Public/Blurbs/158608.aspx.

STATE

Kentucky

by Royce Meredith, Kentucky Transportation Cabinet, and Dr. Tony Hunley, Stantec Consulting Services



This bridge in Scott County, Ky., was among the first bridges replaced as part of the Bridging Kentucky program. All Photos: Kentucky Transportation Cabinet.

Maintaining and improving Kentucky's inventory of more than 14,000 aging bridges is a big task, which grows larger each year. Like so many other states, Kentucky has had precious few dollars allocated in recent years for major transportation projects. Given the financial constraints, leaders have focused primarily on protecting current assets and investing strategically in key expansion projects. Meanwhile, the state's inventory of smaller bridges that are key connectors but often have limited traffic volumes has been a lower priority. The state is now making a concerted effort to rectify this situation.

State Program to Address Safety Concerns

As of mid-2018, Kentucky had more than 1000 structures with posted weight restrictions or National Bridge Inventory (NBI) ratings of 4 (poor) or below. About half of these structures have concrete superstructures. Bridges in the state were receiving substandard ratings faster than they could be repaired or replaced. Most of these substandard structures were smaller bridges owned by county governments and other local agencies that could not afford to fix them.

The Kentucky Transportation Cabinet (KYTC) knew that the condition of the bridges was becoming a safety issue that needed to be addressed for the long-term benefit of the state. Enter *Bridging Kentucky*, a six-year, $700 million program to restore approximately 1000 state and locally owned bridges. Its goal is to ensure bridges throughout the state are safe and sound for crossings by school buses, emergency vehicles, and commercial traffic.

The program, launched in 2018, is providing significant savings for Kentucky and allowing the state to restore, on average, three to four times as many bridges as it previously would have in a typical year. In the first year of the program, the rate of design and delivery has been about six times the normal pace. To date, Kentucky has awarded $79.8 million in construction contracts for 139 bridges (realizing a savings of about 23% compared to the budget for these projects), with a goal of delivering more than 400 program bridges to construction phase through the end of fiscal year 2020 (June 30).

A Programmatic, Rehab-First Approach

To maximize available funding and restore so many bridges in a relatively short period, KYTC knew it needed a new way to address the state's needs. Working with a 22-firm general engineering consultant (GEC) team led by Stantec, QK4 Inc., and AECOM, KYTC set up a bridge-restoration program focused on saving structures and maintaining current assets.

A key element of the Bridging Kentucky program is to reduce project development costs and accelerate delivery of bridges by using innovative approaches and limiting the scope of work for each project. Historically, bridges in Kentucky with significant deterioration and/or load restrictions were fully replaced and, in many cases, expanded to address nearby roadway deficiencies or accommodate potential future needs.

To stretch program dollars KYTC committed the program to perform only the work that was needed to restore the bridge to an NBI rating of 7 or above and an appropriate load rating (44 tons for state routes and 40 tons for county routes). The program also set a minimum design service life of 75 years for replacement projects and 30 years for rehabilitation projects and provided exceptions for historic bridges such as covered bridges and some steel truss bridges.

A Bridging Kentucky project in Kenton County, Ky., added at least 30 years of service life to the bridge by eliminating joints at piers, resurfacing and protecting the deck, rebuilding curbs, and patching abutments and piers. The historic railing was rebuilt and preserved.

Another view of the Bridging Kentucky repair project in Kenton County, Ky..

A construction crew in Franklin County, Ky., places concrete for the deck of a new concrete bridge that is part of the Bridging Kentucky program.

In cases where full replacements were necessary to accomplish these objectives, KYTC and the program team adopted the goal of generally replacing the exisiting bridge with a structure of the same size, in the same location, and with the same hydraulic opening. These basic criteria have helped rein in costs and speed up delivery of projects by limiting right-of-way acquisition, additional environmental permitting, and utility relocations.

Life-Cycle Cost Model

To help determine the right design solution for each bridge, the Bridging Kentucky program team screened each bridge using data already collected by KYTC bridge inspectors, including inspection reports, photos, and notes; NBI ratings; load ratings; element-level condition states; and posting results and notes. This process allowed the team to distinguish potential rehabilitations from replacements. In some cases, the need for major or minor changes yielded an obvious decision to replace or rehabilitate the bridge, and those bridges moved directly to design to expedite projects.

A construction worker assists in placement of a new concrete deck on a bridge in Nelson County, Ky.

The remaining bridges underwent a life-cycle cost evaluation to determine the most cost-effective solution. The program team worked with Paul D. Thompson, an internationally recognized expert in transportation asset management, to compare life-cycle costs of rehabilitating versus replacing each of these bridges. The analysis included a deterioration model customized for Kentucky's environment, user costs developed by the program team, and estimated construction costs to restore elements in poor condition. This cost estimate was baselined against similar recent work on Kentucky projects. Through this life-cycle cost analysis for the greater cost advantage, the team found that about 40% of the bridges could be rehabilitated rather than replaced.

Project Bundling

Another cost-saving and time-saving approach used by the Bridging Kentucky program team has been project bundling, which KYTC and the program team have pursued with input from the Federal Highway Administration (FHWA) Every Day Counts (EDC-5) initiative team. Following an FHWA-supported peer exchange with states such as Ohio, Pennsylvania, and Georgia, KYTC finalized its plans for project bundling. This is the first time that Kentucky had used widespread bundling for bridge construction projects.

KYTC began bundling construction projects in March 2019 and has continued to consolidate projects based on location, scope of work, and project priority. Full bridge replacements and rehabilitation projects with superstructure replacements are grouped together when possible, as are other rehabilitation projects of similar-type structures.

To date, KYTC has awarded a dozen project bundles ranging in size from 2 to 13 bridges per bundle, for a total of 53 bridges. The average cost savings on these bundled projects has been 12%, with more significant savings seen with the larger bundles.

Kentucky also is using design-build as a delivery method. KYTC recently announced a design-build project to replace 102 bridges in eastern Kentucky. Five teams responded to the request for qualifications, and a design-build team is expected to be selected later in 2019.

Kentucky's New Standards for Concrete Bridges

Beyond expediting critical bridge projects and providing millions in costs savings, the Bridging Kentucky program has brought an additional benefit to Kentucky as the program team has worked with KYTC bridge engineers to develop new design and repair standards, particularly for concrete bridges. Several of these standards are highlighted here.

Prestressed Concrete Box-Beam Bridges

Many of the program bridges to be replaced have short spans, which are well suited for adjacent prestressed concrete box-beam bridges. Designers are specifying concrete decks on all replacement projects to extend the life of the structures and achieve the program's 75-year service-life goal. Adjacent box beams allow for an expedited construction schedule because deck forming is not required. One challenge is that many of the projects cannot be completed using phased construction due to the narrow bridge widths, and often road closure is the only economical option.

Specifications for 3-ft-Wide Prestressed Concrete Box Beams

KYTC currently has standard designs and drawings using 4-ft-wide box beams, and these standard drawings allow for an "off-the-shelf" design that can be used by the state and counties for future projects. The program team is expanding the state's standard designs to add

This concrete arch bridge in Louisville, Ky., is among the historic bridges being rehabilitated and restored through the Bridging Kentucky program.

3-ft-wide box beams. These narrower beams provide two main benefits:

- They allow for custom fits of new superstructures on existing substructures that previously supported concrete, steel, or slab superstructures.
- They can be used on replacement projects to match existing bridge widths, minimizing impacts on right-of-way and utilities and reducing costs.

Innovative Concrete Repair Methods

In many rehabilitation projects, the team has used innovative processes that reduce overall project costs while meeting the program's 30-year service-life goal. These innovations include:

- Developing an aggressive procedure for concrete patching by surveying industry specialists. This procedure includes removing concrete beyond reinforcing steel, saw cutting repaired edges, using abrasive blast cleaning to create an appropriate surface profile, utilizing self-consolidating concrete and a form-and-pour technique on repairs greater than 1 ft^2, requiring contractors to perform quality-control testing, and adding a galvanic protection system with embedded galvanic anodes to extend the life of concrete and protect reinforcing steel from corrosion.
- Resolving the source of water and chloride infiltration by eliminating transverse deck joints on superstructure rehabilitation and replacement projects. Additional measures include retrofitting deck drains to deflect water away from girders and piers and adding drip strips along fascia girders on bridges that have over-the-side drainage.
- Proactively preserving new and existing concrete surfaces from water and chloride infiltration by sealing repaired substructure concrete on rehabilitation projects with a high-quality epoxy-coating system and treating concrete decks with silane sealers.

Superstructure Strengthening

When possible, the team is strengthening concrete bridges to meet program goals. This has primarily been achieved by adding structural concrete overlays to achieve composite action of conventionally reinforced concrete or prestressed concrete girder bridges and by using carbon-fiber wrap systems to strengthen conventionally reinforced concrete girders.

Looking to the Future

By the end of fiscal year 2020, KYTC plans to have designed projects and awarded construction contracts for more than 450 of the 1000 bridges being restored through the Bridging Kentucky initiative. However, much work remains to keep the program on track.

Since the program's inception, about 160 more bridges in Kentucky have been load posted or closed by the state's bridge engineers due to unsafe conditions. KYTC is preparing its transportation budget and road plan for fiscal year 2021–2022, which starts in July 2020. To support Kentucky's data-driven approach to planning, the program team currently is using the life-cycle cost model it developed to screen the additional bridges and reprioritizing its list of remaining bridges to help KYTC and state leaders determine which bridges should be restored next and included in the road plan for the next two years.

The program team will repeat this process in two years, bringing additional value to a program that already is saving Kentucky millions of dollars and fixing bridges that the state needs to maintain a strong transportation system.

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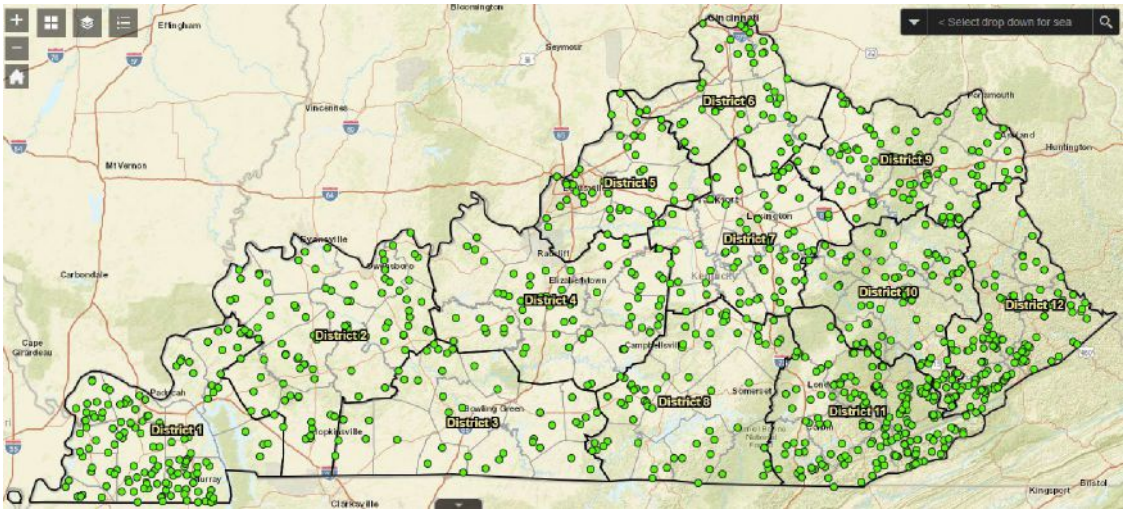
EDITOR'S NOTE

For more information on Bridging Kentucky, visit the website at bridgingkentucky.com



An overview of all of the bridges included in the Bridging Kentucky program from the program website.

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CONCRETE CONNECTIONS

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

IN THIS ISSUE

http://armeniconsulting.com
This is a link to the website of Armeni Consulting Services. The firm specializes in pre- and postconstruction services and is the featured company in the Focus article on page 6.

https://www.asbi-assoc.org/projectGallery/project.cfm?articleID=C8CA08A5-D74E-281F-21BAA2FFAC038717
This is a link to the American Segmental Bridge Institute's web page with photos and information on the recently completed "Missing Link" section of the Foothills Parkway in Great Smoky Mountains National Park. The nine Missing Link bridges are featured in a Project article on page 10.

https://www.hdrinc.com/portfolio/marc-basnight-bridge-bonner-bridge-replacement#
This is a link to the Marc Basnight Bridge designer's portfolio page for the project; the web page includes design and construction information, videos, and additional links. The bridge is featured in a Project article on page 18.

https://www.ncdot.gov/projects/bonner-bridge/Pages/default.aspx
This is a link to the North Carolina Department of Transportation's website on the Marc Basnight Bridge, which features project history and other information. The bridge is featured in a Project article on page 18.

http://www.dot.ga.gov/BuildSmart/Projects/Pages/CourtlandSt.aspx
This is a link to the Georgia Department of Transportation's web page with information and time-lapse video of the construction of the Courtland Street bridge in Atlanta, Ga. The bridge is featured in a Project article on page 24.

https://arcosalightweight.com/case-studies/structural-lightweight-concrete/shasta-bridge
This is a link to a website with information on the lightweight concrete used in the Shasta Viaduct Arch Bridge. The bridge is the subject of a Construction Bridge Technology article on page 32.

http://gsbridge.com/portfolio/current-projects
This is a link to a page on the website of Golden State Bridge, the contractor of the Shasta Viaduct Arch Bridge. The web page has construction photos of the arches. The bridge is featured in a Construction Bridge Technology article on page 32.

https://www.pci-foundation.org/programs
This is a link to the PCI Foundation's webpage providing information on architectural and engineering design studios at several universities. The program at California State University, Sacramento, is featured in a Perspective article on page 46.

https://www.fhwa.dot.gov/bridge/loadrating/pubs/hif18061.pdf
This is a direct link to a downloadable version of *Concrete Bridge Shear Load Rating Synthesis Report* (publication FHWA-HIF-18-061). The report is the focus of the Federal Highway Administration article on page 48.

https://store.transportation.org/Common/DownloadContentFiles?id=1712
Clicking this link downloads the table of contents for the third edition of the American Association of State and Highway Transportation Officials' *Manual for Bridge Evaluation*. The manual's shear load rating provisions are discussed in the Federal Highway Administration article on page 48.

https://bridgingkentucky.com
This is a link to the website for the Bridging Kentucky program, a six-year, $700 million program to restore approximately 1000 state and locally owned bridges. Kentucky is the featured state in an article on page 50.

OTHER INFORMATION

http://www.virginiadot.org/vtrc/main/online_reports/pdf/19-R29.pdf
This is a link to a downloadable version of *Concrete Beams Prestressed Using Carbon Fiber Reinforced Polymer* (Final Report VTRC 19-R29). The Virginia Transportation Research Council report is the result of a study of an in-service bridge that was constructed using prestressed concrete bulb tees with carbon-fiber reinforced polymer reinforcement.

http://mdot.ms.gov/documents/research/Reports/Interim%20and%20Final%20Reports/State%20Study%20288%20-%20Best%20Practices%20for%20Estimating%20Camber%20of%20Bulb%20T%20and%20Florida%20Girders.pdf
This is a link to a downloadable version of *Best Practices for Estimating Camber of Bulb T and Florida Girders*, a study sponsored by Mississippi Department of Transportation. The study's investigators conducted a literature search for estimating beam camber, surveyed other state departments of transportation regarding current practices, and analyzed beam camber data to gain a better understanding of factors that influence beam camber.

https://www.fhwa.dot.gov/publications/research/infrastructure/structures/bridge/uhpc/19011/19011.pdf
This is a link to a downloadable version of *Design and Construction of Field-Cast UHPC Connections*, published by the Federal Highway Administration. This publication updates an earlier document and provides guidance on the design and deployment of field-cast ultra-high-performance concrete connections.

https://abc-utc.fiu.edu/webinars/webinar-archives
This is a link to the website of the Accelerated Bridge Construction Center at Florida International University. The site offers access to current and archived webinars on case studies of accelerated bridge construction techniques.

http://elearning.pci.org
This is a link to the PCI eLearning Center website, with contains online courses on precast, prestressed concrete elements and materials for the transportation and building industries. These courses are free and satisfy continuing education requirements of engineers in all 50 states.

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Prestressed Concrete Girders Achieve Record Lengths

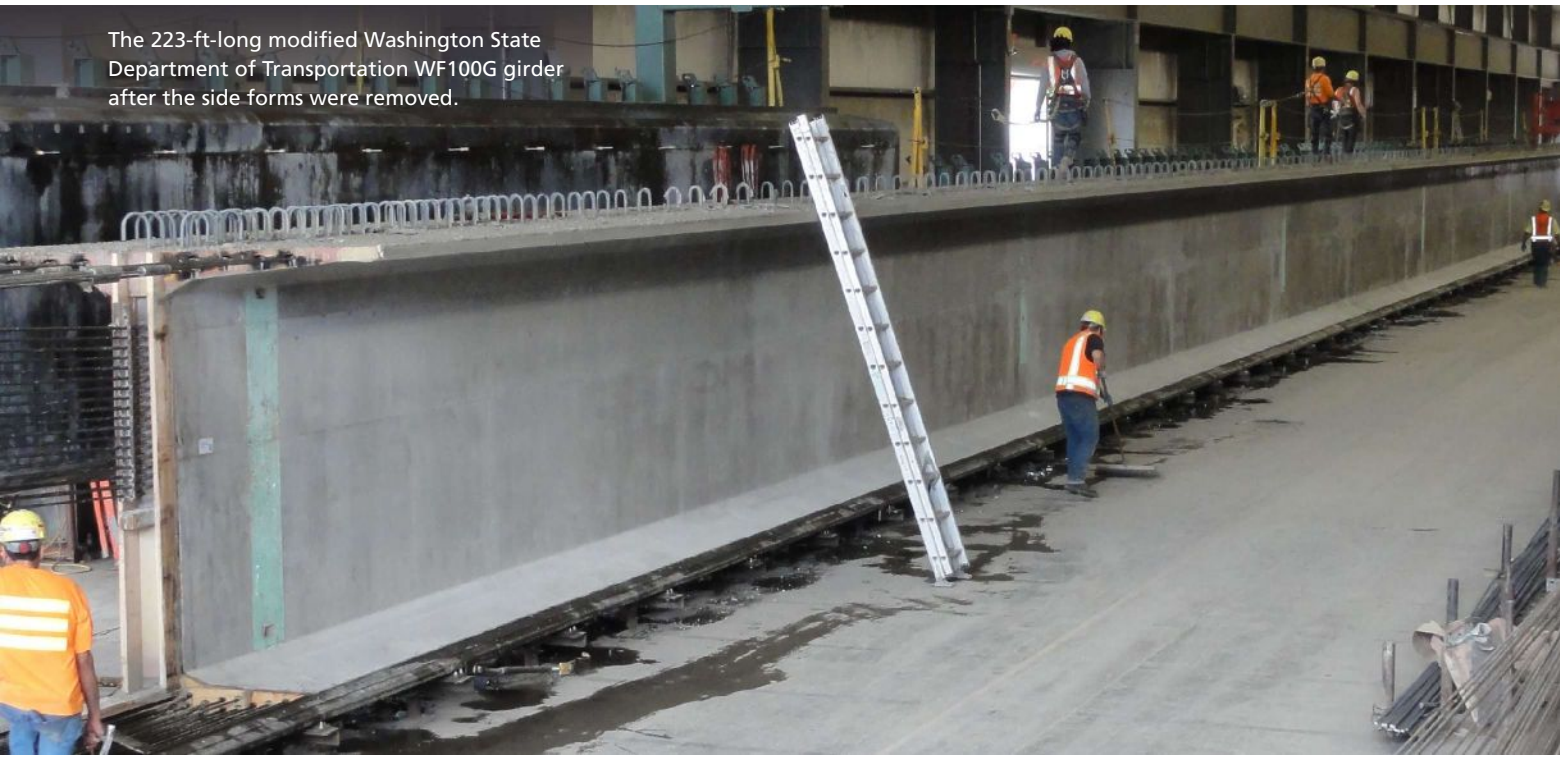
by Cameron West, Concrete Technology Corporation

The 223-ft-long girder before side forms were set. All Photos: Concrete Technology Corporation.

Concrete Technology Corporation (CTC) and the design-build team of Atkinson Construction and Jacobs have partnered in Tacoma, Wash., on the Interstate 5 (I-5) Portland Avenue to Port of Tacoma Road southbound high-occupancy vehicle project, which is pushing the length limits of what is currently achievable with prestressed concrete girders. This bridge over the Puyallup River and multiple rail lines requires long span lengths and extreme skews. The longest girder, spanning five existing and future rail lines, has a plan length of 223 ft and approximately 55-degree end skews. This girder now holds the record as the longest single-piece prestressed concrete girder made in the U.S. Other important features of the project include the use of lightweight concrete to meet handling and shipping requirements and the use of a modified Washington State Department of Transportation (WSDOT) WF100G girder section with a widened top flange to improve stability.

Limiting Girder Weight

An early challenge in the project was reducing girder weight to ensure that it could be hauled to the jobsite by truck. Because available hauling



The 223-ft-long modified Washington State Department of Transportation WF100G girder after the side forms were removed.

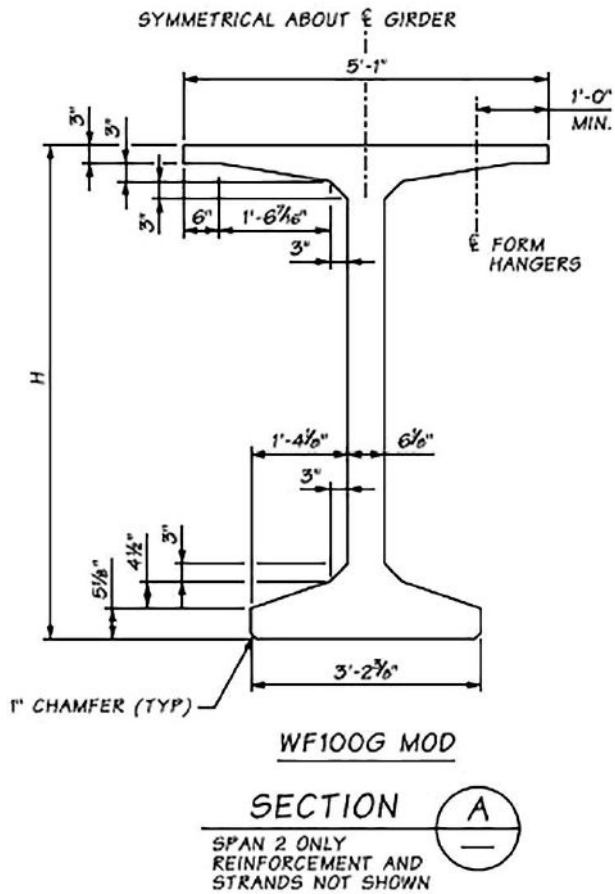
equipment in the region currently limits the maximum girder weight to 270 kip, lightweight concrete was chosen. On the I-5 Portland Avenue to Port of Tacoma Road project, the use of lightweight concrete with a fresh density of 0.125 kip/ft^3 and a reinforced density of 0.138 kip/ft^3 reduced the girder weight by 20%, to 247 kip. This concrete mixture met all project criteria and was able to reach a compressive strength of 8.4 ksi for transfer at 15 hours and 10.0 ksi at 28 days. CTC had previously used the same lightweight concrete mixture in several girder projects, including the I-5 Skagit River Bridge Replacement (see the Winter 2014 issue of *ASPIRE*® and the January–February 2015 issue of the *PCI Journal*) and State Route 162/6 Puyallup River Bridge (see the Summer 2016 issue of *ASPIRE*).

Girder Stability and Prestressing

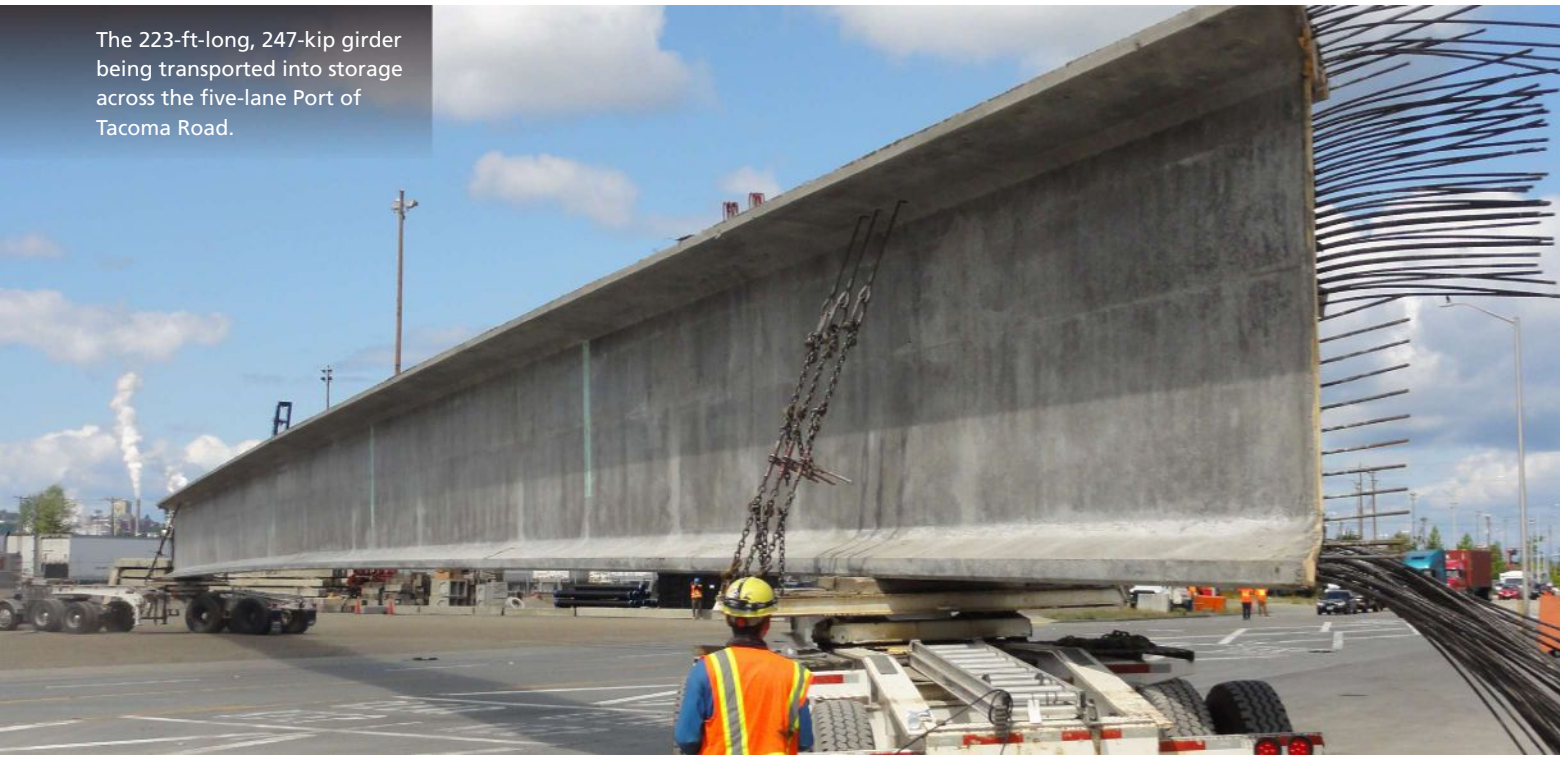
Special consideration was given to girder stability and stresses during plant handling, hauling, and erection. This analysis resulted in the selection of a modified WSDOT WF100G girder cross section, where the top flange was widened from 4 ft 1 in. to 5 ft 1 in. for this project to increase the weak-axis stiffness. The modified girder cross section was only used for one span with the longest girders; other spans used standard girder cross sections. To meet handling and hauling criteria, the design also required the use of ten 0.6-in.-diameter pretensioned temporary top strands, pick points at 26 ft from each end, and hauling bunk points at 25 ft from each end.

In addition to the record-setting length, this girder has a very high amount of prestress. The design specified 46 straight, 35 harped, and 10 temporary top 0.6-in.-diameter strands, which results in an initial tensioning force of nearly 4 million lb. This force, when combined with the girder length and lifting locations, required a concrete compressive strength of 8.4 ksi at transfer. The prestressing force in combination with the severe skew required strand debonding and supplemental mild reinforcement in the acute corner of the bottom flange to minimize cracking.

Cameron West is the chief engineer with Concrete Technology Corporation in Tacoma, Wash.



Modified girder cross section. Figure: Washington State Department of Transportation.



The 223-ft-long, 247-kip girder being transported into storage across the five-lane Port of Tacoma Road.

Fiber-Reinforced Polymer Reinforcement

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

Fiber-reinforced polymer (FRP) reinforcement consists of a continuous fiber, such as a glass, carbon, or aramid, embedded in a resin matrix, such as epoxy, polyester, vinyl ester, or phenolics. FRP reinforcement may be used as reinforcing bars, prestressing strand, or post-tensioning tendons. These types of reinforcement do not corrode, can be nonconductive, and are lighter to ship and install than steel reinforcement. However, tests have shown that some FRP bars can lose tensile strength in a highly alkaline solution.

Nonprestressed Reinforcement

In a 2016 survey, 17 of 42 transportation agencies in the United States and Canada reported that they had used FRP reinforcement in cast-in-place concrete bridge decks. However, its use is not mainstream; epoxy-coated steel reinforcement continues to be the dominant material for deck reinforcement.

Prestressing Strand

Several demonstration projects have shown the potential of FRP prestressing applications in bridges, but its use has not become generally accepted. One hurdle has been the lack of a design document by the American Association of State Highway and Transportation Officials. This challenge should be remedied now that National Cooperative Highway Research Program Project 12-97, *Guide Specifications for the Design of Concrete Bridge Beams Prestressed with Carbon Fiber-Reinforced Polymer (CFRP) Systems*, has been completed and approved.

Key Resources

1. American Association of State Highway and Transportation Officials (AASHTO). 2009. *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings*. Washington, DC: AASHTO.
2. American Concrete Institute (ACI). 2007. *Report on Fiber-Reinforced Polymer (FRP) Reinforcement for Concrete Structures*. ACI 440R-07. Farmington Hills, MI: ACI.
3. ACI. 2008. *Specification for Construction with Fiber-Reinforced Polymer Reinforcing Bars*. ACI 440.5-08. Farmington Hills, MI: ACI.
4. ACI. 2011. *Prestressing Concrete Structures with FRP Tendons*. ACI 440.4R-04. Farmington Hills, MI: ACI.
5. ACI. 2015. *Guide for the Design and Construction of Structural Concrete Reinforced with Fiber-Reinforced Polymer Bars*. ACI 440.1R-15. Farmington Hills, MI: ACI.
6. Russell, H.G. 2017. *NCHRP Synthesis of Highway Practice 500: Control of Concrete Cracking in Bridges*. Washington, DC: National Research Council, Transportation Research Board.

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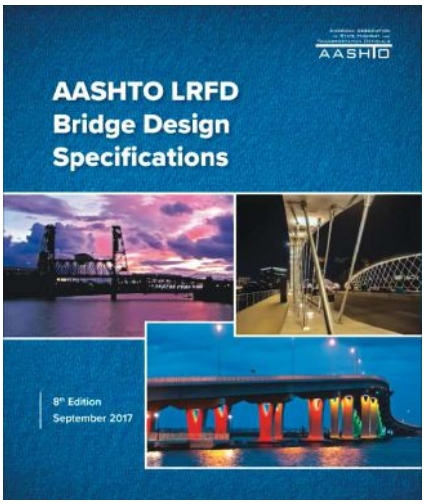
Upcoming Changes to the AASHTO LRFD Bridge Design Specifications

by Dr. Oguzhan Bayrak, University of Texas at Austin

The 2019 meeting of the American Association of State Highway and Transportation Officials (AASHTO) Committee on Bridges and Structures took place in Montgomery, Ala., during the last week in June. In that meeting, three working agenda items prepared by AASHTO Technical Committee T-10 Concrete Design were approved. This article covers the three changes, which will appear in the 9th edition of the *AASHTO LRFD Bridge Design Specifications*, which is scheduled to be published in 2020.



Strand Debonding Rules

The first change relates to strand debonding rules. Strand debonding is a technique used for reducing stresses in the end regions of pretensioned concrete beams. The *AASHTO LRFD Bridge Design Specifications* 8th edition[1] limits the amount of debonding to 25% of the total number of strands within a pretensioned girder. This limit was imposed in recognition of the potential detrimental effects that excessive debonding could have on shear performance.

The Transportation Research Board's National Cooperative Highway Research Program (NCHRP) Project 12-91, *Strand Debonding for Pretensioned Girders*,[2] was initiated to develop recommended revisions to the current debonding provisions. The study considered both service and strength limit states and various beam cross sections. The proposed revisions to the AASHTO LRFD design specifications include outcomes of the NCHRP research, additional research by others,[3–5] and past practice.

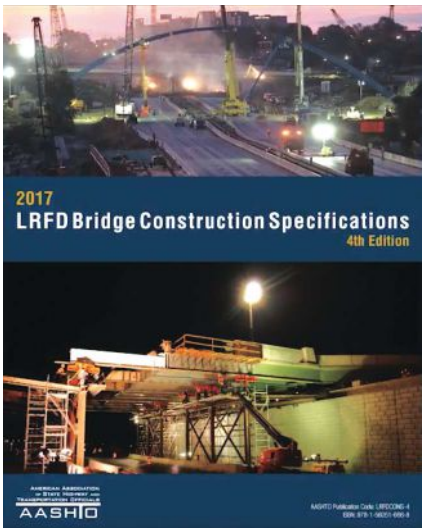
According to the adopted revisions to Article 5.9.4.3.3, straight pretensioned strands may be debonded at the ends of beams with the following restrictions:

- The number of strands debonded per row shall not exceed 45% of the strands in that row, unless otherwise approved by the owner.
- Debonding shall not be terminated for more than six strands at any given section. When a total of 10 or fewer strands are debonded, debonding shall not be terminated for more than four strands at any given section.
- Longitudinal spacing of debonding termination locations shall be at least $60d_b$, where d_b is the diameter of the strand.
- Debonded strands shall be symmetrically distributed about the vertical centerline of the cross section of the member. Debonding shall be terminated symmetrically at the same longitudinal location.
- Alternate bonded and debonded strand locations both horizontally and vertically.
- Where pretensioning strands are debonded and where service tension exists in the precompressed tensile zone, the development lengths, measured from the end of the debonded zone, shall be determined using Eq. 5.9.4.3.2-1 with a value of $\kappa = 2.0$.
- For simple-span precast, pretensioned concrete girders, debonding length from the beam end shall be limited to 20% of the span length or one-half the span length minus the development length, whichever is less.
- For simple-span precast concrete girders made continuous using positive-moment connections, the interaction between debonding and restraint moments from time-dependent effects (such as creep, shrinkage, and temperature variations) shall be considered.
- For single-web flanged sections (I-beams, bulb tees, and inverted tees):
  - Bond all strands within the horizontal limits of the web when the total number of debonded strands exceeds 25%.
  - Bond all strands within the horizontal limits of the web when the bottom-flange-to-web width ratio exceeds 4.
  - Bond the outermost strands in all rows located within the full-width section of the flange.
  - Position debonded strands farthest from the vertical centerline.
- For multiweb sections having bottom flanges (voided slabs, box beams, and U-beams):
  - Uniformly distribute debonded strands between webs.
  - Strands shall be bonded within 1.0 times the web width projection.
  - Bond the outermost strands within the section.
- For all other sections:
  - Debond shall be distributed uniformly across the width of the section.

- Bond the outermost strands located within the section, stem, or web.

Post-Tensioning Anchorage Hardware Requirements

The second change establishes consistency between the AASHTO LRFD design specifications and *AASHTO LRFD Bridge Construction Specifications*[6] with respect to the testing of and acceptance requirements for post-tensioning anchorage hardware. In the 8th edition of the AASHTO LRFD design specifications,[1] Article 5.4.5—Post-Tensioning Anchorages and Couplers requires anchorages to conform to the *AASHTO LRFD Bridge Construction Specifications*.[6] Prior to the 8th edition of the AASHTO LRFD design specifications, Commentary C5.4.5 required anchorages and couplers to develop 95% of the specified ultimate strength of the tendons; this requirement was contradictory to the *AASHTO LRFD Bridge Construction Specifications*, which led to confusion. The 8th edition of the AASHTO LRFD design specifications removed specific requirements and simply made reference to the *AASHTO LRFD Bridge Construction Specifications*[6] for anchorage requirements, which helped clear up confusion.



The newly approved change makes it clear that the requirements are based upon the *actual* ultimate tensile strength, not the specified tensile strength. The new provisions use the static strand-wedge testing requirements that are specified to be in accordance with PTI M50.1 *Acceptance Standards for Post-Tensioning Systems*[7] and will help standardize the testing requirements.

By specifying 95% of the actual ultimate tensile strength as the acceptance target, the updated requirements now align with the original research contained in NCHRP Report 356.[8] In addition, most of Article 10.3.2.1 of the *AASHTO LRFD Bridge Construction Specifications*[6] concerning the location of bonded tendon anchors is repeated in Article 5.9.5.6.1 of AASHTO LRFD design specifications,[1] which concerns the design of post-tensioning anchorage zones. This is because decisions on where to locate anchorages are usually made during the design phase of a bridge project. Finally, a sentence in Article 10.3.2.2 of the *AASHTO LRFD Bridge Construction Specifications*[6] was moved to Article 10.3.2.1 to improve clarity.

Detailing Ties in Reinforced Concrete Columns

The third change relates to the detailing of ties in reinforced concrete columns. According to the new rules, for columns that are not designed for plastic hinging, the spacing of laterally restrained longitudinal bars or bundles is not to exceed 24.0 in. measured along the perimeter tie. In this context, a restrained bar or bundle is one that has lateral support provided by the corner of a tie having an included angle equal to or less than 135 degrees. The change to reduce the spacing from 48.0 in. to 24.0 in. for laterally restrained longitudinal bars or bundles returns the language to the original intent of the 1980 Interim Revisions to the *AASHTO Standard Specifications for Highway Bridges*.[9] It is important to note that the 1980 interim revision references research by Pfister,[10] and the reduction in spacing is consistent with that research.

Conclusion

The three changes discussed in this article will improve the next edition of the AASHTO LRFD desgin specifications. In future articles, I will discuss in greater depth the technical background and implications of these changes based on feedback from the industry and our readers.

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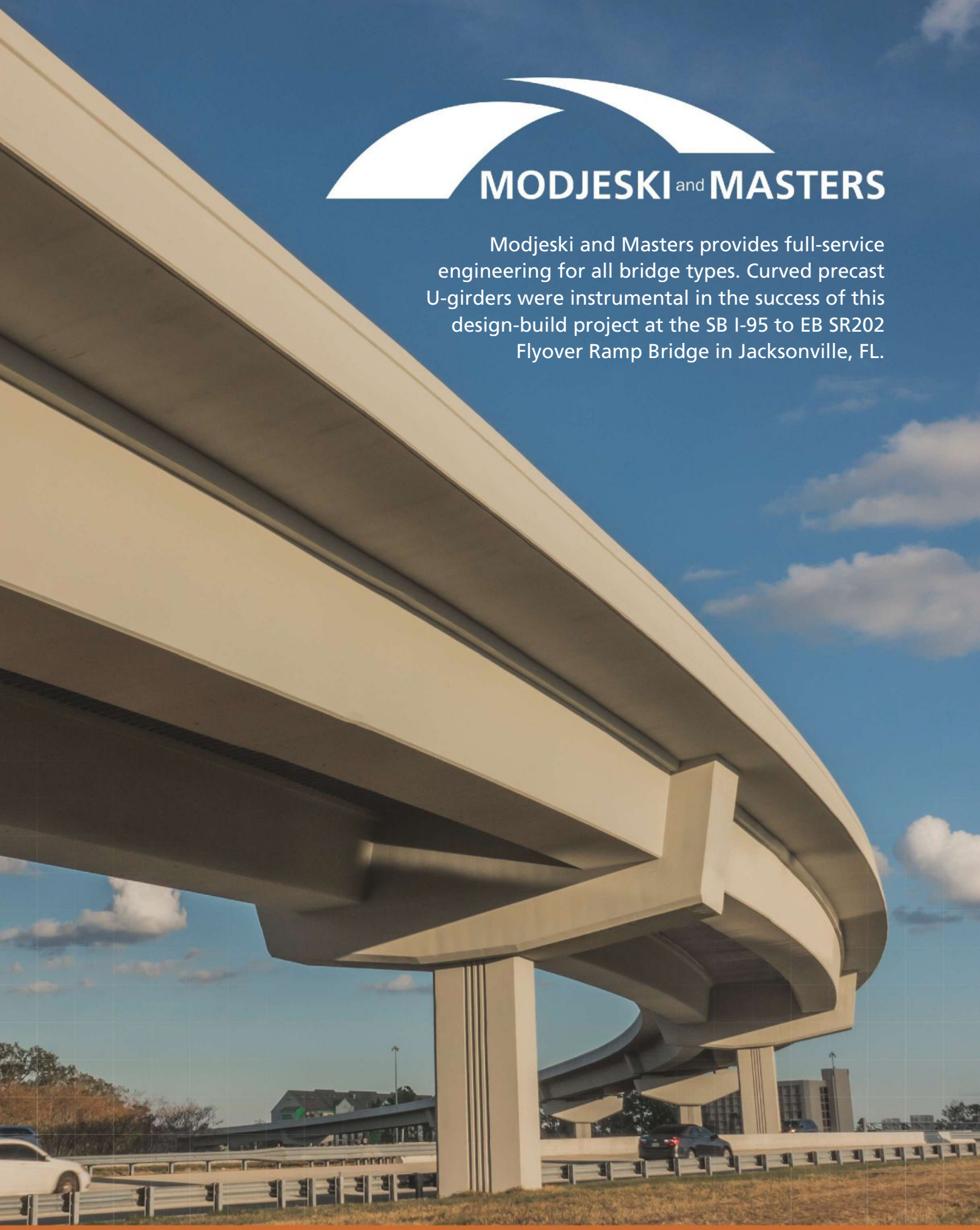
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