ASPIRE™

THE CONCRETE BRIDGE MAGAZINE

FALL 2014

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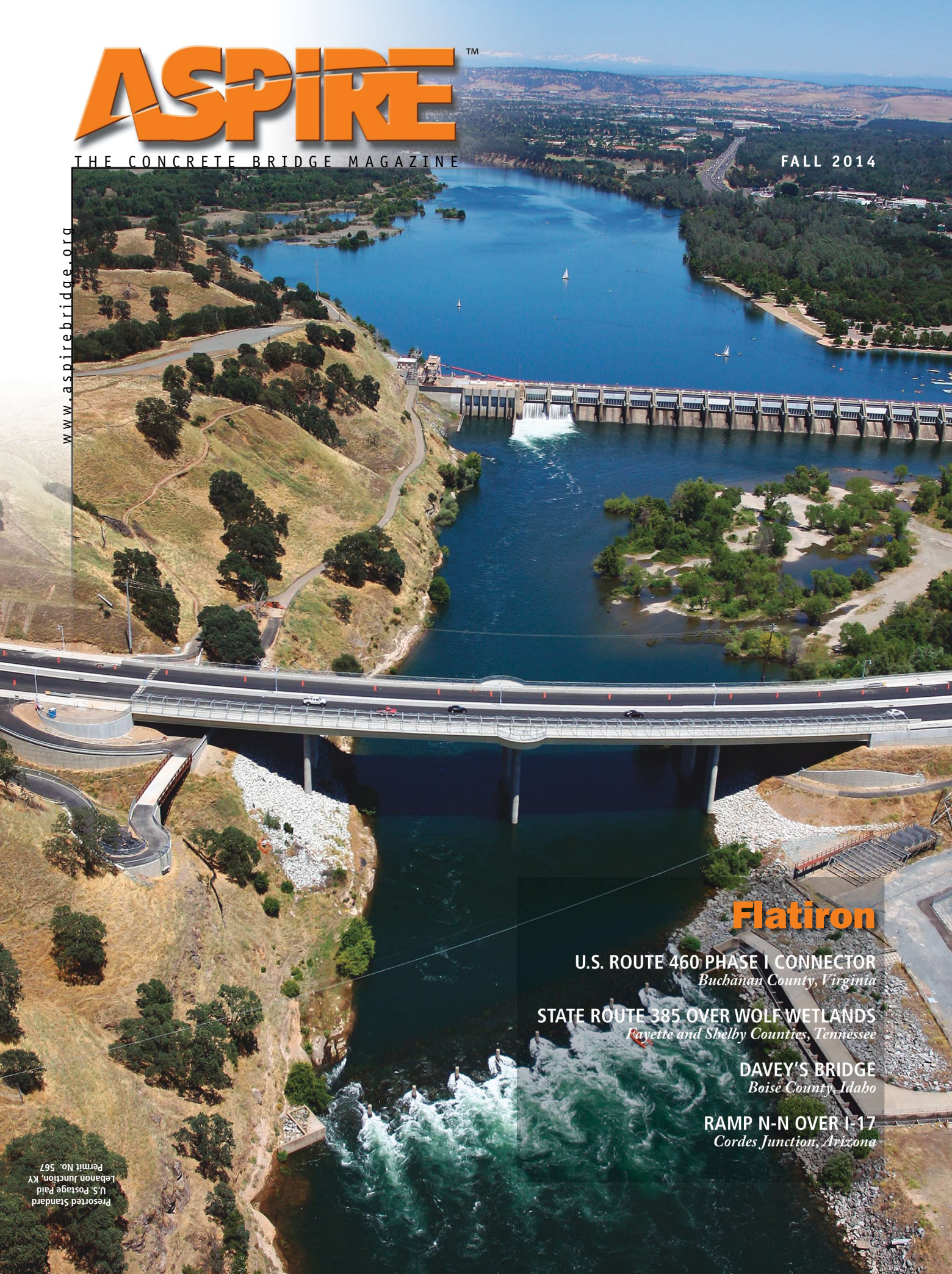

Flatiron

U.S. ROUTE 460 PHASE I CONNECTOR
Buchanan County, Virginia

STATE ROUTE 385 OVER WOLF WETLANDS
Fayette and Shelby Counties, Tennessee

DAVEY'S BRIDGE
Boise County, Idaho

RAMP N-N OVER I-17
Cordes Junction, Arizona

Presorted Standard
U.S. Postage Paid
Lebanon Junction, KY
Permit No. 567

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Photo: Flatiron.

Photo: Mark's Photo.

Photo: Tennessee Department of Transportation.

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Departments

Photo: Western Federal Lands.

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Photo: PCI.

Always Promote the Inevitable

William Nickas, *Editor-in-Chief*

In the Summer 2014 issue of *ASPIRE*,™ I discussed how human talent and creativity is as vital as technique. It is also completely necessary as our profession pushes ahead to the next generation of concrete bridges. In this issue, we feature our second contractor profile, which emphasizes how selecting and maximizing internal and external skills for a project creates the most powerful solutions for their team.

In 2004, I was asked to serve as a delegate on a team looking at prefabricated bridge elements and systems (PBES) in Japan and Europe. Little did I know that the team, which was co-lead by Mary Lou Ralls of the TxDOT (now retired and an independent consultant) and Ben Tang of the Federal Highway Administration (FHWA) (now retired and at Oregon DOT), would offer some of the framework used to motivate so many engineers in re-thinking our profession.

In a 2008 meeting, Mike Culmo of CME Associates shared with the participants of the Transportation Research Board Workshop 109 a thought-provoking question: "Is the old adage 'you can only have two of the three: High Quality, Rapid Construction, or Low Cost' still valid when it comes to accelerated bridge construction (ABC), or will we deliver ABC bridges with all three attributes?"

This momentum for change was truly accelerated with FHWA's programs like Highways for LIFE, Peer2Peer Summits, and Everyday Counts. Procurement and delivery techniques are always changing and departments of transportation (DOTs) guide those changes through their systems routinely. The applied physics, upon which structural engineering is based, will always remain the same; many DOT bridge design manuals take years to change. It is the "hows" and "whats" that we work to reinvent at times. These are what take the longest. Over the last 10 years, our profession has built up a good backlog of the newest ABC concrete bridge experiences. In this issue, we are including the final article in the ABC series.

In this ABC feature, the authors further demystify how designers in seismic regions do have the ability and desire to re-think the use of ABC structural components and systems. Some of the most complex bridge connection issues come about in seismic areas.

"Always Promote the Inevitable" was a saying used by Octave J. DuTemple, executive director of the American Nuclear Society, to remind his members to recognize that when practical consensus had been reached, stop arguing, and get behind the direction being taken to make it work.

When a change is a natural progression, everyone should get on board, but when it is a major overhaul, there will be those early adopters. . . And sometime these changes just become the way we do it. In an effort to continue to showcase the best practices and briefly offer nuggets of knowledge, the *ASPIRE* team is launching a new series titled, "Concrete Bridge Technology." These topics will cover some of the hows and whys of concrete bridge design, detailing, and construction. This is not intended to stop advancing innovations with the most versatile construction material; however, there are many topics that have a historical significance and may be worth re-sharing. As an inventory of ideas for ground breaking research is assembled and orchestrated (see Graybeal's Summer 2014 *ASPIRE* article), the *ASPIRE* team is picking selected topics to bring forward tidbits of history. If you have a specific request, please send us your topic. I hope you enjoy this new series and remember "always promote the inevitable."

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Cover
Precast, prestressed concrete girders with a length of 145 ft were used for the Hazel Avenue Bridge over the American River in Sacramento, Calif., reducing the number of piers in the sensitive river, which had a salmon-hatching area downstream and a dam upstream. Photo: Flatiron.

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

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

Photo: Ted Lacey Photography.

CONCRETE CALENDAR 2014/2015

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

October 26-30, 2014
ACI Fall Convention
Hilton, Washington
Washington, D.C.

October 27-28, 2014
ASBI 26th Annual Convention
Hartford Marriott Downtown
Hartford, Conn.

December 4-5, 2014
2014 National Accelerated Bridge Construction Conference
Hyatt Regency, Miami, Fla.

January 11-15, 2015
94th Annual Meeting Transportation Research Board
Walter E. Washington Convention Center
Washington, D.C.

February 2-6, 2015
World of Concrete 2015
Las Vegas Convention Center
Las Vegas, Nev.

March 9-11, 2015
DBIA Design-Build in Transportation
Henry B. Gonzalez Convention Center
San Antonio, Tex.

April 6-7, 2015
ASBI 2015 Grouting Certification Training
J. J. Pickle Research Campus
The Commons Center
Austin, Tex.

April 12-15, 2015
ACI Spring Convention
Marriott Hotel and Kansas City Convention Center
Kansas City, Mo.

April 19-24, 2015
2015 AASHTO Subcommittee on Bridges and Structures Annual Meeting
Saratoga Hilton
Saratoga Springs, N.Y.

April 26-28, 2015
2015 PTI Convention
Royal Sonesta Hotel
Houston, Tex.

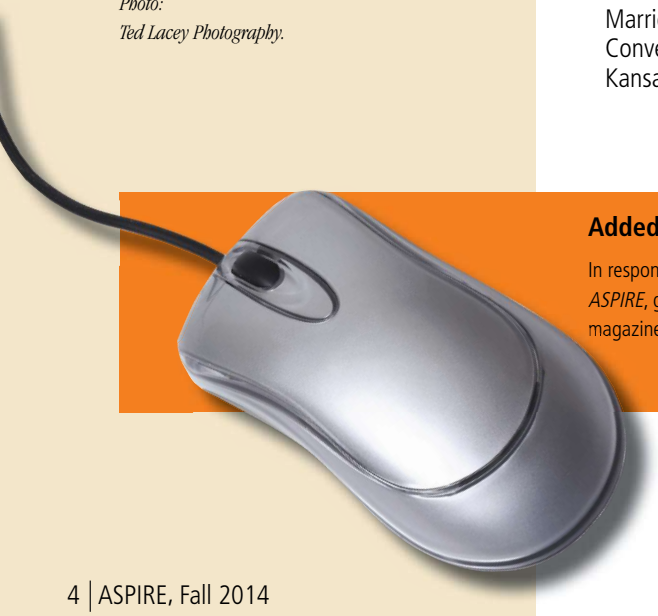
April 30 – May 3, 2015
PCI 2015 Committee Days and Membership Conference
Hyatt Magnificent Mile
Chicago, Ill

June 8-11, 2015
International Bridge Conference
David L. Lawrence Convention Center
Pittsburgh, Pa.

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

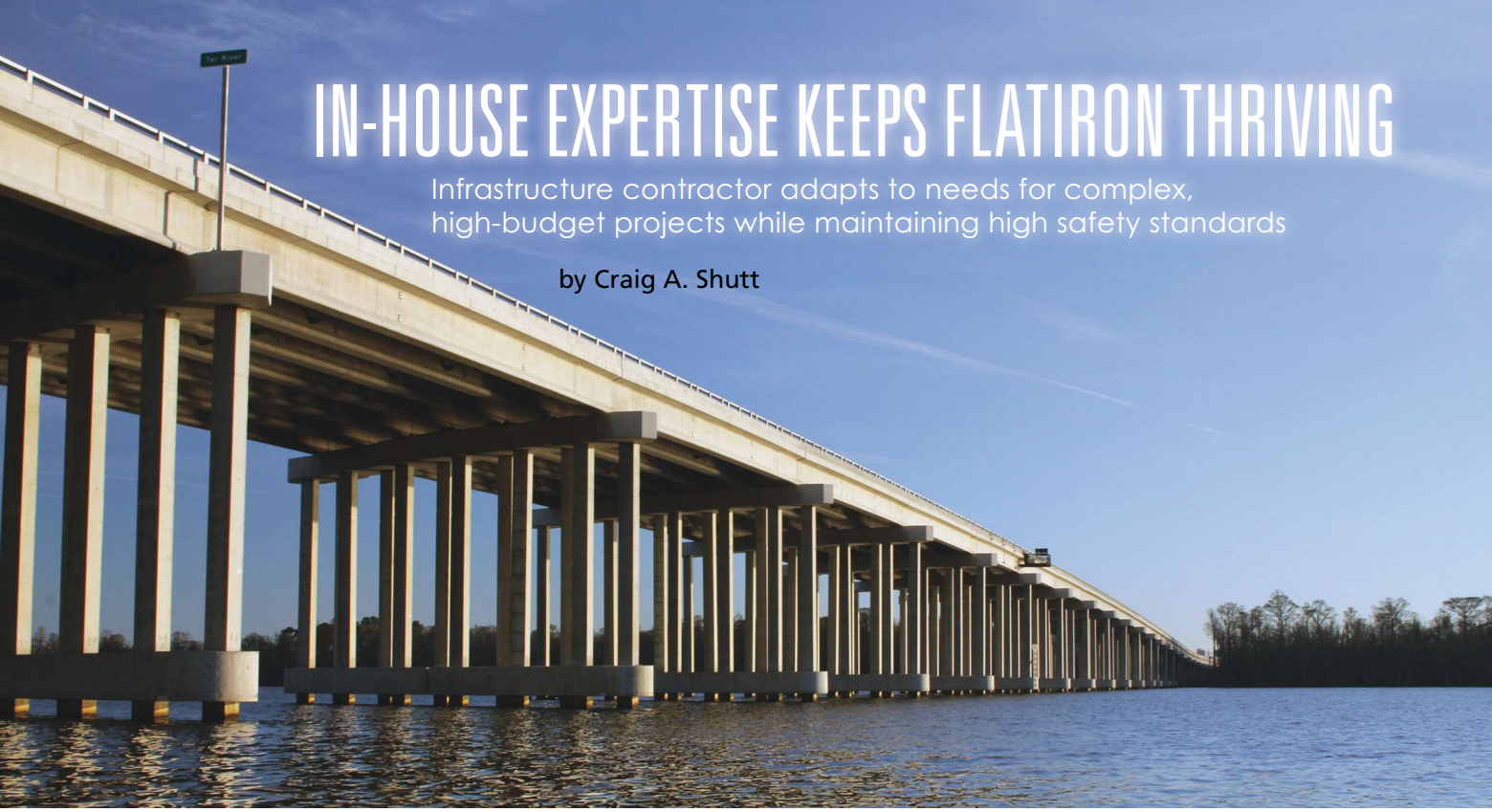
October 15-18, 2015
PCI 2015 Fall Committee Days and Membership Conference
Louisville, Ky.

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



Added Web Search Feature

In response to reader comments, the *ASPIRE* mobile site has added a search feature. To search past issues of *ASPIRE*, go to aspirebridge.org and click on "Check out the NEW phone/tablet friendly version of the *ASPIRE* magazine here." From the page that appears, click on the + sign to browse or search all archived issues.



IN-HOUSE EXPERTISE KEEPS FLATIRON THRIVING

Infrastructure contractor adapts to needs for complex, high-budget projects while maintaining high safety standards

by Craig A. Shutt

The $192-million U.S. Route 17 Washington Bypass project in Beaufort County, N.C. All Photos: Flatiron.

Bridge projects are evolving in many ways, including delivery methods, complexity, and increased environmental sensitivity. Project teams are required to adapt and use technical expertise to innovate. Infrastructure contractor Flatiron meets the needs of this evolution through a determined focus on constructability, which is enhanced by using in-house resources to design a best-value concept for clients. The company also prides itself on having award-winning safety procedures that minimize the risk of accidents.

"Our technical expertise and in-house engineering capabilities make us stand out," says Allan Brayley, chief engineer at the Firestone, Colo.-based firm. "We do a lot of construction engineering in-house and have access to a lot of specialized equipment for all types of bridges, including segmental and those requiring falsework and formwork. It gives us an edge because we work closely with the estimators in the office at bid time and with the field people at construction time to ensure the plans come together efficiently."

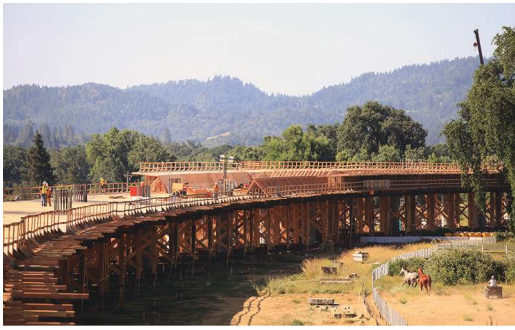
Adds Richard Grabinski, vice president of Flatiron's western region, "We're respected as being good builders. We build the projects ourselves—we're out there with the tools doing it. We compete for bid-build projects, but we also win many based on our best-value proposals. We offer our services, skills, experience, ingenuity, and people, and that often wins the day. We prefer that approach—it's competitive in a way that plays to our strengths."

Flatiron has participated in traditional bid-build projects, as is shown by its current work on a bypass project in Willits, Calif., for the California Department of Transportation (Caltrans). A joint venture, the $107.9-million project provides a new segment for Route 101. Two interchanges and 15 bridges are included, most consisting of cast-in-place concrete box girder bridges with one having a length of 6000 ft.

Alternative Methods

The company also specializes in other delivery methods, which provide more opportunity to utilize their skills and to innovate, while increasing efficiency, lowering cost, and decreasing risk for clients. Owners are noticing. "More states are open to design-build projects today," says Brayley.

The firm worked on the Minnesota Department of Transportation's first design-build project, the I-35W St. Anthony Falls replacement bridge, which used construction speed as



Flatiron's work on the $107.9-million Willits, Calif., bypass project included work on two interchanges and 15 bridges, including a 6000-ft-long, cast-in-place concrete box girder bridge.

For the Willits, Calif., bypass project, the team created an Environmental Hazard Analysis for communicating environmental hazards to the pile-driving crew and integrating permit, wildlife, and pollution concerns into the operations.

a deciding factor. (See *ASPIRE*™ Fall 2008.) "We had the highest bid price but the shortest time frame, and it was accepted," he notes. "Design-build sets up scoring for different elements—technical expertise, design, schedule, and pricing. States always want low pricing, but they also factor in other elements."

A recently completed design-build bridge project on Interstate 85 over the Yadkin River near Salisbury, N.C., shows its creative approach. (See *ASPIRE* Winter 2014.) The $136-million project, consisting of precast concrete girders and a cast-in-place concrete deck, reconstructed a 55-year-old deteriorating bridge using a single temporary work bridge constructed between the two new parallel structures. "Creating this trestle is something we do on a fairly regular basis now, because it offers a lot of flexibility and minimizes disruption," Brayley says.

Flatiron has also found success with public-private-partnership (P3) projects. "We are well-versed in P3 projects from work in Canada, where the laws make it more common," says Grabinski. Those projects often require different construction criteria, he notes, as concessionaires, who will maintain the bridge for decades after construction, look more long-term—and often have more resources to invest upfront to avoid later costs. "They have a heightened awareness of life-cycle costs."

Both Brayley and Grabinski see this format growing in the United States. "The lack of steady government funding for transportation projects is a growing concern," says Grabinski. "States and cities are asking how they can build new infrastructure and maintain so many bridges with limited funds. P3 projects bring private equity into the equation and add funds with longer repayment terms."

Grabinski also notes growing interest in the construction manager/general contractor (CM/GC) format, in which the contractor consults on the early stages of design, which is produced by the owner's team. "While popular in construction of buildings, CM/GC is being used more regularly for infrastructure projects," he says. That's especially true in California. Caltrans is now running a six-project demonstration pilot program to study CM/GC more closely. Flatiron has worked with the Federal Highway Administration in California, recently completing one CM/GC bridge project and performing preconstruction services on a second.

"Owners understand that early input from the contractor offers benefits for constructability and maximizing schedule efficiency," says Grabinski. "It helps the owner identify and mitigate risks before construction begins. It also allows us to set a firm price earlier in the process, providing owners with greater price certainty."

'Owners understand that early input from the contractor offers benefits for constructability and maximizing schedule efficiency.'

The $136-million concrete bridge on Interstate 85 over the Yadkin River in Salisbury, N.C., features precast concrete girders and a cast-in-place concrete deck that were constructed using a single temporary work bridge between the new structures.

All of the pieces for the U.S. Route 17 Washington Bypass project were set in place from an overhead gantry system, which was used to drive piles, set precast concrete bents, and erect beams. Once the deck was cast, the gantry moved forward to the next span.

The U.S. Route 17 Washington Bypass project features 140 spans of precast, prestressed concrete girders as well as precast, prestressed concrete hollow piles, beams, post-tensioned pile caps, and a cast-in-place concrete deck.

Pile placement for the U.S. Route 17 Washington Bypass project took place from an overhead gantry system.

Shared Risk

Risk is being shared more often as projects grow in complexity and budget. Many of Flatiron's larger projects are undertaken as joint ventures, often with Flatiron as the lead. "As projects grow, we need to share resources, risk, and financial backing," Brayley explains.

One of its more prominent joint ventures was the $192-million U.S. Route 17 Washington Bypass project in Beaufort County, N.C., a design-build project that Flatiron led with joint-venture partner United Contractors of Chester, S.C. The project featured 140 spans of precast, prestressed concrete girders plus precast, prestressed concrete hollow piles, prestressed beams, post-tensioned pile caps, and a cast-in-place concrete deck. (See *ASPIRE* Fall 2008.)

The bridge was constructed with an overhead gantry system, which was used to drive piles, set precast concrete bents, and erect the girders. Once the deck was cast, the gantry pushed forward to the next span. "This approach was taken because of the sensitive environmental area, which is becoming a bigger concern overall," says Brayley.

Choosing joint-venture partners is an art and science, he notes. Key factors include finding someone with expertise in the type of bridge being built, familiarity with the owner, and past success. "We want to work with people who have been successful with this owner and type of project, and they look at us the same way," says Brayley. "A lot of this industry is about personal relationships and past experience."

The industry focuses on mitigating risk in many ways. One approach Flatiron has emphasized is safety training throughout its culture. "When people talk about risk, the first thing I think of is safety," says Brayley. "That's the major way we can mitigate risk for our company."

A few years ago, the company updated and expanded its safety program. That led, in 2012, to Flatiron working more than 4 million worker-hours without a lost-time safety incident. The firm received a First Place Safety Excellence Award from the Associated General Contractors of America to note that accomplishment. Flatiron has been recognized by the Alberta Roadbuilders

Commitment to Safety

Flatiron's enhanced safety plan permeates the company's entire culture:

Programs. It operates a variety of high-visibility campaigns, including "Don't Walk By—Take Action," which encourages workers to take corrective action at the site. It also emphasizes a "Stretch & Flex" exercise program, mandatory protective gloves, data-capture analysis, jobsite walk-throughs, Supervisor Training in Accident Reduction (START), field-safety coaching, and a two-day Leadership for Safety Excellence.

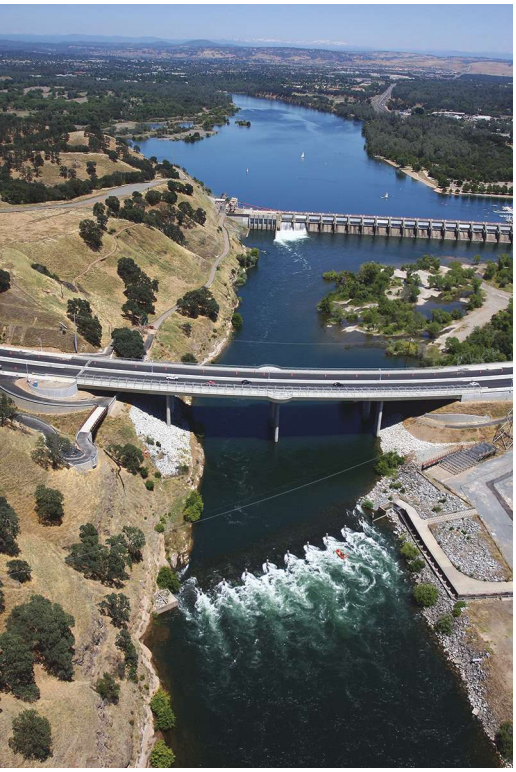
Incentives. The company launched an employee safety-suggestion program in 2010 to recognize safe behavior and implement suggestions with points redeemed for prizes and gift cards.

Leadership. Every operational meeting begins with a review of safety requirements and each manager receives annual safety training. The managers also participate in several site-safety self-assessment audits each year.

Training. Employees receive 20 hours of annual safety training through classes, online courses, and external training. Biannual "Safety Stand Down" events are held in each region, at which managers present material and bring in guest speakers.

Communication. The firm runs a Zero/Zero campaign designed to create a full year of no recordable or lost-time safety incidents. It also publishes a quarterly employee magazine with a "Spotlight on Safety" section.

Outside Influence. Flatiron encourages safety programs for subcontractors and sponsors a scholarship through the American Society of Safety Engineers. Flatiron was also one of 31 contractors to participate in the first annual Safety Week in 2014, designed to raise safety awareness industry-wide.



Precast, prestressed concrete girders with a length of 145 ft were used for the Hazel Avenue Bridge over the American River in Sacramento, Calif., reducing the number of piers in the sensitive river, which had a salmon-hatching area downstream and a dam upstream.

and Heavy Construction Association, the American Road & Transportation Builders Association, and the Government of Alberta for safety excellence.

More Concrete Designs

The firm's focus on controlling the process and using in-house expertise has led to more concrete bridges, especially precast concrete. "Our preference is to build concrete bridges, because there's more for us to self-perform in those structures," says Brayley. "With concrete segmental bridges, we can do our own precasting. That means we control the manufacturing process, scheduling, cost, and quality in-house." Concrete also has benefits on larger projects, he notes. "There are fixed costs for yard production and forms, so the more spans we create using those assets, the cheaper each one becomes. We can spread the costs further."

Concrete pricing also has been more consistent, adds Grabinski. "We are building more with concrete because there's a bigger push by owners in that direction. Pricing is less volatile, making it easier to estimate, and long-term maintenance costs are lower. That's making owners ask for more concrete designs."

Concrete also has benefits for high-seismic regions, due in part to changes in connection detailing. "California works mostly in concrete due to seismic concerns," Brayley says. Adds Grabinski, "In many ways, bridges are designed as they were 20 years ago, but with nuances for increased seismic capacity. Many connections on concrete bridges, including those between columns and footings, girders and pier caps, and girders and abutments, are more forgiving today. The bearing plates are different to allow more movement with seismic forces."

Transportation innovations also have opened more opportunities for concrete girders, Grabinski notes. "Transportation issues are a key issue and are evolving quickly. There's more ability to transport precast concrete units efficiently and not worry about cracking. Precasters are now casting and transporting longer girders than ever before. Pieces are getting bigger and longer, which helps reduce material and erection costs."

Evolving erection technology also is making components easier to handle. "Cranes have improved and can handle much larger units," Grabinski says. "Self-erecting cranes and large-capacity ones can set larger units and maneuver on difficult sites." Brayley agrees, "The size of the equipment has grown to meet the new capabilities. We've handled concrete segments up to 140 tons, although our target weight is 80 to 90 tons. That's a practical size and economical to use."

One recent project to use large girders was the Hazel Avenue Bridge over the American River in Sacramento, Calif., which had a salmon hatching area downstream and a dam upstream. "It was an especially sensitive area that required minimizing impacts to the river," says Grabinski. The company used sixteen 145-ft-long precast concrete girders, erected with 200-ton cranes, to reduce the number of piers. The format also sped construction, requiring roadway closures for only one weekend.

"Quite often, we're facing the challenge of building projects in ever-more environmentally sensitive areas," Grabinski says. "These projects will require creating solutions we haven't used before or refining past solutions. The nature of bridge construction is evolving. It's a lot different than it was 25 years ago when I started, and those changes will continue."

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

67 Years of Growth

Named after the unique rock formations found near Boulder, Colo., Flatiron has grown from a small materials company at its founding in 1947 to one of the largest transportation and infrastructure contractors in North America.

Its expansion didn't begin until 1991, when it opened an office in southern California. In 1997, it secured its first design-build project in Maine and began work on Boston's "Big Dig" project. In 1998, it expanded into northern California, and the next year it won projects in the Carolinas, Florida, Texas, Louisiana, and Utah.

In 2005, it expanded into Canada, from where it secured its largest project ever, the Port Mann Bridge in Vancouver, B.C., in 2008. In 2007, the firm was acquired by Hochtief Aktiengesellschaft (HOCHTIEF), a German construction company. In 2010, it acquired E.E. Crux of Holmdel, N.J., to expand into the Northeast, and in 2011 it formed a joint venture with Leighton to pursue work in Hong Kong. It currently operates 10 offices in North America.

Engineering News Record lists the firm as the sixth largest bridge contractor, the seventh largest highway contractor, and the eleventh largest in transportation overall.

PROJECT

U.S. ROUTE 460 PHASE I CONNECTOR OVER GRASSY CREEK AND ROUTE 610

Durable and cost-effective cast-in-place segmental design-build project in Virginia

by Leo Spaans, Janssen & Spaans Engineering Inc.

Located in western Virginia near the state line with Kentucky, the U.S. Route 460 Phase I Connector over Grassy Creek and Route 610 in Buchanan County spans a deep valley. Due to the need for such tall, flexible columns, geometry control for the superstructure became quite challenging. This article describes the project and further explains how critical geometry control for both the substructure and superstructure was maintained.

Description of Project

The project consists of twin, cast-in-place, segmental concrete superstructures with one prestressed concrete I-beam approach span at each end. The total length of the structure is 1725 ft. Span lengths for spans 1 through 6 are 110, 268.5, 489, 489, 268.5, and 103 ft. The pier heights measured from the bottom of the footing to the top of the superstructure for piers 1 through 5 are 109.5, 227, 261.5, 158.75, and 72.5 ft.

Piers 1 and 5, which support the prestressed concrete I-beam spans and the end spans of the cast-in-place segmental structure, are a more common pier type. This type consists of a concrete footing supported by either micro-pile or directly founded on rocks (spread footing). The column consists of two circular columns that are 6 ft in diameter with a 42-ft-wide cap.

Piers 2 through 4 have a spread footing founded on rock, with an H-shaped column integrally connected to the cast-in-place concrete superstructure.

Cost-Effective Solution

Since the project used a design-build contract, it provided the designer with unique opportunities; it allowed the designer to evaluate the different options, such as the best solution for the project based on the capabilities of the specific contractor with which the designer is working. With contractor's team input, the design options/alternatives were directly evaluated for cost effectiveness as well as utilizing the contractor's experience and capabilities.

The pier table was 50 ft long to accommodate two form travelers. All Photos: Mark's Photo.

profile

U.S. ROUTE 460 PHASE I CONNECTOR OVER GRASSY CREEK AND ROUTE 610 / BUCHANAN COUNTY, VIRGINIA

BRIDGE DESIGN AND CONSTRUCTION ENGINEER: Janssen & Spaans Engineering Inc., Indianapolis, Ind.

INDEPENDENT QUALITY CONTROL: Entran PLC, Lexington, Ky.

PRIME CONTRACTOR: Bizzack, Lexington, Ky./CJ Mahan, Columbus, Ohio

PRECASTER: Prestress Services Industries Inc., Melbourne, Ky.—a PCI-certified producer

POST-TENSIONING CONTRACTOR & FORM TRAVELER: VSL, Dallas, Tex.

EXPANSION JOINT & BEARING SUPPLIER: D.S. Brown, North Baltimore, Ohio

OTHER SUPPLIERS: MMFX, Irvine, Calif.

The tallest pier is 261.5 ft tall from bottom of the footing to top of the superstructure.

During the pre-bid phase, several options were studied including both steel and concrete superstructures and different span lengths in the range of 250 to 500 ft.

In developing cost curves, where both substructure and superstructure were figured in cost per square foot of deck area, it became apparent that longer spans were more cost effective. For example, the tallest pier already required 1700 yd^3 of concrete. The estimates clearly indicated that fewer piers with longer spans were more cost effective.

Structure height and the difficult site conditions made crane erection not cost effective for superstructure erection. The preliminary designs considered steel plate girders and steel box girders using a launching system, and cast-in-place segmental box girders constructed with form travelers.

Again, erection with overhead launching trusses for such a short structure (1725 ft) was not cost effective.

In the final analysis, the cast-in-place concrete segmental option turned out to be the most cost-effective solution for this contractor team.

The cast-in-place concrete segmental option turned out to be the most cost-effective solution for this contractor team.

Superstructure Design

The cast-in-place concrete segmental superstructure consists of a variable depth, single-cell box girder which is 12.5 ft deep at midspan and 30.25 ft deep at the pier. The overall width of the box girder is 43 ft 4 in. The typical segment length is 16 ft 6 in. and the pier table is 50 ft long. This allowed two form travelers to be placed on the pier table at the same time without too much interference with each other.

For durability, the superstructure used an 8.0 ksi compressive strength concrete, as well as non-prestressed reinforcement meeting ASTM A1035. The deck of the box girder is transversely post-tensioned with four 0.6-in.-diameter strand tendons at 2 ft centers. The design was based on zero allowable tension in the top of the deck at full-service-load conditions.

The longitudinal analysis used the same zero-allowable-tension criteria. The cantilever tendons consisted of nineteen, 0.6-in.-diameter strands. Twelve 0.6-in.-diameter strands were used for the continuity tendons. Both groups of tendons used the bonded system, with ducts placed in the concrete section. External unbonded tendons were provided for future use. The design provided anchor points and deviation blisters to allow for 10% of the permanent post-tensioning to be added at any time in the future.

Based on the as-built structure, the following quantities were used for the superstructure:

- 14,590 yd^3 of concrete, providing an equivalent thickness of 3 ft
- 2,590,550 lb of nonprestressed reinforcement, based on 178 lb/yd^3
- 2,239,000 lb of post-tensioning tendons, with 2.5 lb/ft^2 in the transverse direction of the deck and 17 lb/ft^2 in the longitudinal direction

Piers 1 and 5 used two columns with a cap beam.

COMMONWEALTH OF VIRGINIA DEPARTMENT OF TRANSPORTATION, OWNER

BRIDGE DESCRIPTION: Twin, 1725-ft-long, post-tensioned, cast-in-place, segmental box girder bridge

STRUCTURAL COMPONENTS: Each bridge consists of five cast-in-place concrete pier columns supporting four spans of cast-in-place single-cell box girder and two spans of precast, prestressed concrete I-beams

BRIDGE CONSTRUCTION COST: $47 million ($314/ft^2)

AWARDS: *Roads & Bridges* magazine Top Ten Bridges, Number 1

The walls of the H-shaped pier columns continue into the box girder to provide an integral connection.

The single-cell box has a depth of 30.25 ft at the pier.

Substructure Design

The tallest pier column from the bottom of the footing to the top of the superstructure is 261 ft tall, which created some interesting design challenges. To make connections integral with the superstructure and eliminate bearings, an open section was used. This, as opposed to a typical box section, avoids the steel staircase, eases future inspection, and simplifies forming.

Switching from a box section to an open H section has significant consequences for the stiffness of the column, especially during the construction of the free cantilever superstructure. Dead loads, construction loads, an unbalance moment at the top of the column, and additional wind loads on the cantilever are significant, requiring a delicate balance between the needed stiffness and the acceptable deflections.

The reduced moment of inertia compared to that of a closed box section created issues with the torsional stiffness. This also required the designer to address local buckling of the flanges, horizontal rotation, and torque in the pier columns.

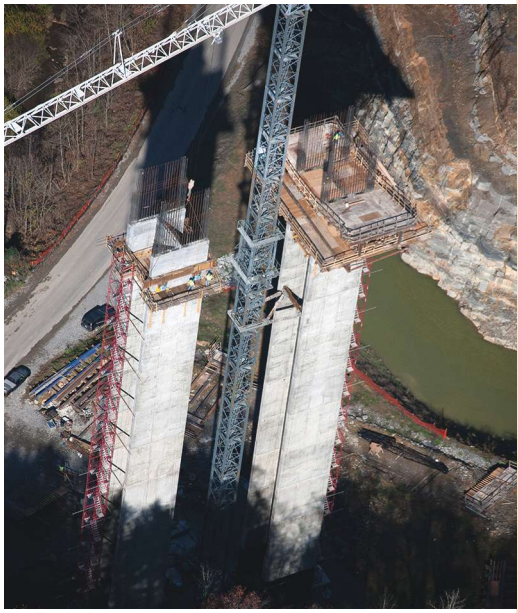
The H-shaped pier column is ideal for making an integral connection between the column and superstructure. The walls of the H column continue into the box girder and act as diaphragms in the box. Because the width of the wall actually widens the bottom slab, the shear forces in the webs of the box girder flow directly into the pier column. There is no need to have complicated reinforcement and/or transverse post-tensioning to push or pull the superstructure reaction forces onto a bearing support. These forces simply go into the column. This type of integral connection greatly simplifies the design and translates into simpler, easier, and more cost-effective construction.

Geometry Control

Geometry control of both the substructure and superstructure was critical. The columns were cast in segments about 20 ft tall with a climbing form system. The tallest column was 222 ft 2 in. tall and required 11 separate concrete placements.

The typical H-shaped pier has wall thickness of 2 ft except for the bottom 34 ft, where the minimum thickness is 3 ft. Upon completion of each concrete placement, the contractor performed a survey similar to that used for the segmental superstructure. These survey results were included in the information for setting the formwork for the next concrete placement. Using this method, the contractor was able to keep the geometric deviation within ½ in. for the entire column.

Upon completing the weekly cycle of casting superstructure segments, a survey was taken of the as-cast segments. These data were then incorporated into the setting of the form travelers for the next casting cycle. The contractor and the designer both performed the calculations, with the designer serving as an independent check.



The H-shaped pier column provided a flexible column, which made geometry control very important.

The geometry control for the superstructure became important with such tall, flexible columns. Not only did the loads cause rotation of the cantilever, they also caused horizontally movement up or down station of about 4 in. depending on which direction the unbalance moment acted. The end of the cantilever also moved about 12 in. up or down. These displacements were just due to dead load and did not include wind load, construction load, and/or movements caused by differential temperatures.

In theory, geometry control is quite simple if the engineer knows the exact properties of the column. However, the actual modulus of elasticity of the concrete and differential temperature of column play a big role in the behavior of the column. For geometry control during the erection, theoretical and actual behavior were fine-tuned so that the predictions for geometry control matched the actual behavior of the structure.

Leo Spaans is chairman of Janssen & Spaans Engineering Inc. in Indianapolis, Ind.

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

AESTHETICS COMMENTARY

by Frederick Gottemoeller

A major part of the visual impact of haunched box girder bridges is created by the interaction of two basic structural decisions. The first is setting the ratio of the girder depth at the haunch to the girder depth at midspan. The second is establishing the slope of the webs. The most noticeable visual aspect of the girders—the three-dimensional curves of their bottom edges—are a geometric result of those two decisions. The greater the depth ratio and the greater the slope of the webs, the more pronounced those curves become, and consequently the more memorable the structure.

Two of the completed cantilevers highlight the curves of the structure.

Obviously, considerations of structural efficiency, constructability, and the geometric interaction of the two factors themselves limit how far one can go. The U.S. Route 460 Phase I Connector over Grassy Creek and Route 610 has found a good balance. The curves are strong enough to catch the eye. In so doing, they make the bridge visually interesting while illustrating the flow of forces in the structure. The girder is thickest above the piers, where intuition says the forces will be greatest.

The split piers at pier 4 provide another element that engages the viewer. From most angles, they seem thick and robust. But from straight on, where the void between them is evident, they almost disappear. As a viewer moves around a structure the alternation from solid to slim is breathtaking. Concrete box girder bridges constructed in balanced cantilever are at their most dramatic at that moment when the cantilevers are done but not yet connected. The immense girders seem balanced on toothpicks. With split piers, that is indeed the case.

Engineers know the forces involved and the strength of the materials involved and so take all of this in stride. But to non-engineers, it is a kind of magic. In a sense, split piers draw back the magician's veil and show how the trick is done. Engineers will be forgiven if they do that more often.

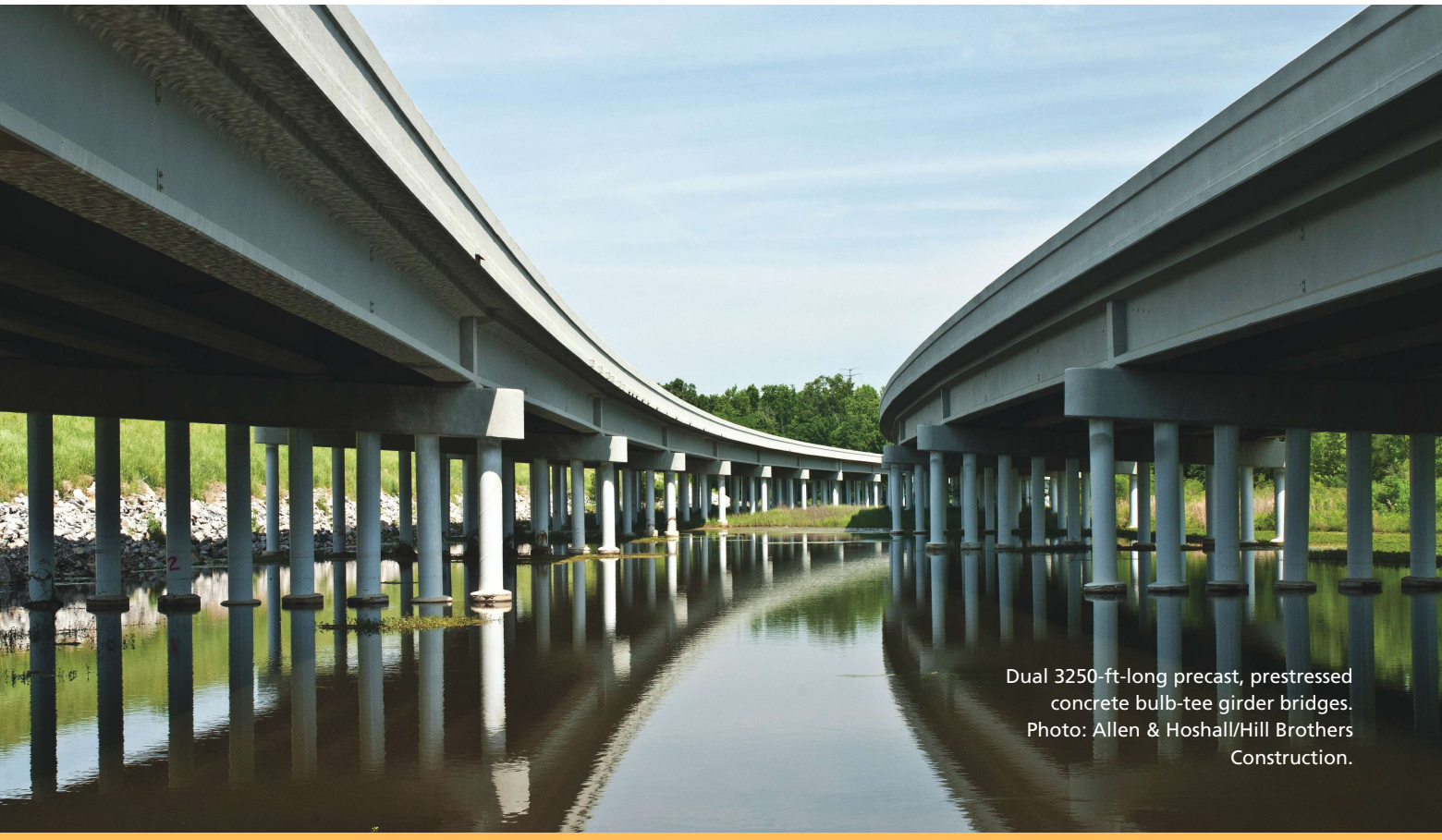
PROJECT

SPANNING
the Wolf River Wetlands in Tennessee

by Tim Huff, Tennessee Department of Transportation

As part of the proposed transportation improvements in southwest Tennessee, the Tennessee Department of Transportation completed plans for a new highway between existing State Routes 193 and 196 in Fayette and Shelby counties. The SR 385 (also known as I-269) project originally consisted of nine bridges. Bridge No. 3, with a length of 720 ft, crossed the Wolf River. Bridge No. 4, with a length of 177 ft, crossed a Wolf River tributary. The original concept for these two Wolf River bridges used precast, prestressed concrete girders on cast-in-place concrete columns with pile caps. Given the extreme depth of the soil profile in this area of the Mississippi embayment, concrete friction piles would typically be used for such a concept.

Environmental concerns, which identified the surrounding area as wetlands, necessitated a change in the plans for Bridge Nos. 3 and 4. It would not be possible to fill in the wetlands between the two bridges and the entire area would need to be bridged. The two structures were combined and lengthened to become dual, 3250-ft-long structures, each comprised of 33 spans in four continuous units with pile bent substructures. Double



Dual 3250-ft-long precast, prestressed concrete bulb-tee girder bridges. Photo: Allen & Hoshall/Hill Brothers Construction.

profile

STATE ROUTE 385 OVER THE WOLF RIVER / FAYETTE AND SHELBY COUNTIES, TENN.

BRIDGE DESIGN ENGINEER: Tennessee Department of Transportation, Nashville, Tenn.

CONSTRUCTION ENGINEERING INSPECTION: Allen and Hoshall, Memphis, Tenn.

PRIME CONTRACTOR: Hill Brothers Construction Company, Memphis, Tenn.

PRECASTERS: Prestress Services Industries of Tennessee, Memphis, Tenn.—a PCI-certified producer (girders) and Construction Products Inc. of Tennessee, Jackson, Tenn.—a PCI-certified producer (deck panels)

bents were provided at the adjacent ends of each unit. Approximately half of each structure lies in a horizontal curve having a 2865-ft radius with the other half in a tangent section.

Alternatives

Two alternative designs for the superstructure were included in the contract plans. The two options were a 54-in.-deep bulb-tee concrete girder design with a typical girder spacing of 9 ft and a rolled steel girder design.

The steel girder alternative required more transverse deck reinforcement, and less longitudinal deck reinforcement, compared to the bulb-tee girder alternative. The net effect was a deck reinforcement total of 2,450,898 lb for the concrete option and 2,117,119 lb for the steel option. Given that the bulb-tee girders each weighed 686 lb/ft, while the steel girders weighed 149 lb/ft, a significant substructure savings was possible for the steel option. Eighteen inch-diameter pipe piles with a wall thickness of $^3/_8$ in. were required for the steel girder alternate versus ½ in. for the bulb-tee alternate. This savings was not realized at the bents at the main channel of the Wolf River (bents 23 and 24). Due to a longer unsupported length, 24-in.-diameter by ½-in.-thick piles were required for both design options at these substructures. The total steel piling estimates for the alternates were 3,015,910 lb for the steel option and 4,721,580 lb for the concrete option.

Specified concrete compressive strengths for the bulb-tee girder alternative were 5.0 ksi at transfer and 6.0 ksi at 28 days.

Both prestressed concrete and rolled steel girder alternatives were designed and detailed to behave as simple spans for non-composite dead loads and continuous thereafter.

Double bents were used at the ends of continuous span units. Photo: Tennessee Department of Transportation.

Site Characterization

The project limits lie in the Mississippi embayment of the New Madrid seismic

Completed dual structures. Photo: Tennessee Department of Transportation.

STATE OF TENNESSEE, OWNER

BRIDGE DESCRIPTION: Dual, 33-span, 3250-ft-long, precast, prestressed concrete bulb-tee girder bridges

STRUCTURAL COMPONENTS: Three hundred and thirty 54-in.-deep, bulb-tee girders (31,941 ft); 3384 3½-in.-thick, precast, prestressed concrete deck panels with 4.75-in.-thick cast-in-place concrete deck; and cast-in-place concrete pier caps, pipe-pile-bents, and integral abutments

BRIDGE CONSTRUCTION COST: $14,220,000 ($50/ft^2)

zone (NMSZ). The embayment depth was estimated to be approximately 2000 ft from maps in the literature.[1] As site amplification in AASHTO is based on properties in the upper 100 ft of the profile, it is clear that the effect of embayment depth in the NMSZ is one area in which research is needed. Eighteen, 80-ft-deep borings were made to investigate subsurface conditions.

Borings away from the main channel of the Wolf River indicated average blow counts computed in accordance with *AASHTO Guide Specifications for LRFD Seismic Bridge Design* of about 19, while those nearest the main channel were in the 12 to 15 blows/ft range. These blow counts correspond to Site Class D conditions away from the main channel and Site Class E conditions at the main channel of the Wolf River. Using the site latitude and longitude, software was used to establish an inferred shear wave velocity for comparison to the blow count correlations already reported. The software calculated an inferred shear wave velocity of 785 ft/sec. The range for which Site Class D conditions are applicable is 589 to 1178 ft/sec. Thus, blow count data combined with an inferred shear wave velocity of 780 ft/sec. indicate Site Class D conditions.

The project lies in Seismic Design Category "2" in accordance with the AASHTO *LRFD Bridge Design Specifications* at the time of design work. Thus, no pushover analysis was required for the structural design.

Unique Design Features

For the prestressed concrete bulb-tee alternative, end blocks were incorporated into the discontinuous ends of the girders to enhance shear capacity and to resist bursting forces at these critical points. Vertical post-tensioning was included to provide added shear and bursting capacity,

Abutments were designed and detailed to behave integrally. No expansion bearings were used at the double bents. Rather, movements from thermal expansion and contraction were accommodated via flexure of the pipe piles at each double bent.

Precast Concrete Superstructure Component Summary

Structure	Girder Type	Girder Qty, ft	Deck Panel Size	No. of Panels
SR-385 / Wolf River Lateral	Type II	2298	8 ft 7 in. by 8 ft	256
SR-385 / Fletcher Road	BT-72	2278	5 ft 8 in. by 8 ft	248
SR-385 / Wolf River	BT-54	31,941	5 ft 10 in. by 8 ft	3384
SR-385 / Wolf River O.F.	Type II	1710	8 ft 7 in. by 8 ft	194
SR-385 / Wolf River Tributary	BT-63	1170	5 ft 10 in. by 8 ft	256
Raleigh-Lagrange / SR-385	BT-63	1075	8 ft 1 in. by 8 ft	102
SR-385 / Johnson's Creek	BT-63	1166	5 ft 10 in. by 8 ft	128
SR-385 / Monterey Road	Type IV	1768	7 ft 8 in. by 8 ft	200

Note: BT = bulb-tee

To simultaneously permit longitudinal thermal displacements and prevent transverse (out-of-phase) deformation at the expansion double bents, a series of four pipe restrainers per bent was installed. To accommodate the transfer of seismic loads from the superstructure to the cap to the piles, a reinforced concrete 'plug' was designed at the interface between piles and bent caps. The design force for restrainers in such an application may be conservatively taken as twice the transverse seismic shear on one of the bents, equivalent to assuming that the bents would move in opposite directions simultaneously during strong ground shaking.

Construction

Five contractors submitted bids for the SR 385/I-269 project. All bidders selected the 54-in.-deep bulb-tee girder alternative over the rolled steel girder option. While this article focuses on the Wolf River Bridge, cost data are reported for the entire project.

The awarded bid for the project totaled $53,473,493, which consisted of $32,079,825 in roadway costs and $21,393,668 in bridge costs. The state estimate was $55,433,176. The dual, 44-ft-wide Wolf River bridges totaled $14,220,000 ($50/ft^2).

For the eight bridges on the project, prestressed component bid prices were

- $85/ft for Type II AASHTO I-beams,
- $150/ft for Type IV AASHTO I-beams,
- $129/ft for 54-in.-deep bulb-tee girders,
- $175/ft for 63-in.-deep bulb-tee girders,

- $155/ft for 72-in.-deep bulb-tee girders, and
- $36/ft for 14-in.-square precast concrete piles.

Precast, prestressed concrete deck panels that are 3.5 in. thick were selected by the contractor for each of the eight bridges on the project instead of an 8.25 in. full-depth, cast-in-place concrete deck. Epoxy-coated reinforcement was used in the precast, prestressed concrete deck panels and in the deck. Girder type, number of deck panels, and deck panel nominal sizes are given in the table. In addition to the precast concrete superstructure elements, 20,805 ft of 14-in.-square precast concrete piling was installed on the project.

Reference

1. Fernández, J. A. 2007. *Numerical Simulation of Earthquake Ground Motions in the Upper Mississippi Embayment*, Atlanta, Ga., Doctoral Dissertation, Georgia Institute of Technology.

Tim Huff is a civil engineering manager 1 with the Tennessee Department of Transportation in Nashville, Tenn.

The deck consisted of precast, prestressed concrete panels with a composite cast-in-place concrete topping. Photo: Allen & Hoshall/Hill Brothers Construction.

PROJECT

DAVEY'S BRIDGE PROJECT

A super simple span

by Samir Sidhom and Greg May, Central Federal Lands Highway Division of the Federal Highway Administration

The Banks-Lowman Highway, also known as Forest Highway 24, is 35 miles north of Boise, Idaho. It is a vital 25-mile-long residential and commercial artery, and a state-designated Scenic Byway. It meanders along the South Fork of the Payette River in the Boise National Forest. Between the towns of Banks and Garden Valley, and south of the town of Crouch, the highway crossed the South Fork on the 54-year-old Davey's Bridge.

Constructed in 1960, the old Davey's Bridge was a 150-ft-long, five-span bridge with two 12-ft-wide lanes and 2-ft-wide shoulders with a 30-ft 6-in.-wide concrete deck supported by cast-in-place concrete girders. Each of the bridge bents was supported on six 13-in.-diameter treated timber piles with unknown lengths. The bridge was considered functionally obsolete and had a sufficiency rating of 46.9.

In July of 2010, the Western Federal Lands Highway Division (WFLHD) Office, in coordination with a consulting firm, completed a study to replace Davey's Bridge.

A single-span bridge was used as the replacement for the original five-span bridge. All Photos: Western Federal Lands.

Replacement Study

The study recommended that the new bridge had to be widened to facilitate the complete reconfiguration of the Banks-Lowman and Middle Folk Road "T" intersection to improve traffic flow and safety. The "T" intersection is located about 100 ft west of the bridge. The reconfigured intersection has both designated right and left hand turn lanes.

The study also concluded that to improve channel hydraulics it was desirable to reduce or eliminate the number of support piers in the river and evaluated several span arrangement options with different superstructure types.

In March of 2011, the WFLHD assigned the design of the bridge project to the Central Federal Lands Highway Division (CFLHD) bridge design team located in Lakewood, Colo.

Single Span, Best Design

After careful review of the study report, the design team decided, early on, that a single-span bridge was the best alternative for the bridge. This eliminated the two-span option presented in the study. A single-span bridge avoided the need for, significant cost of, and environmental degradation inherent in pier construction in the stream bed. A single-span bridge also shortened the construction schedule, and eliminated problems associated with dewatering, cofferdam construction, and equipment access.

profile

DAVEY'S BRIDGE / GARDEN VALLEY, BOISE COUNTY, IDAHO

BRIDGE DESIGN ENGINEER: Central Federal Lands Bridge Office of Bridges and Structures, part of the Federal Highway Administration's Office of Federal Lands Highway, Lakewood, Colo.

PRIME CONTRACTOR: Legacy Contracting Inc., Stayton, Ore.

PRECASTER: Hansen Structural Precast, Caldwell, Idaho—a PCI-certified producer

Upstream view of the bridge.

The single-span option improved seismic resistance, and simplified the seismic design analysis, compared to that required for a multi-span structure. From a sustainability standpoint, a single-span bridge, with no piers, optimizes flow capacity and hydraulic efficiency of the bridge opening. It prevents the need for costly maintenance and repair due to the snagging and retention of flow debris, and the potential for scour at the piers.

Design

The design team had to overcome a number of challenges. These included

- dealing with the location of the bridge, which is in seismic zone 2; the closest active fault contributing to the seismic hazard at the project site is the Squaw Creek Fault located about 18 miles to the west,
- designing the bridge deck with variable fillet (haunch) thickness to accommodate a partial vertical curve on the bridge,
- using staged construction to maintain existing two lane traffic at all times as requested by the county,
- environmental recommendation to eliminate piers in the river,
- ensuring that the unusually long precast concrete girders could actually be delivered to the site,
- completing the two designs of the project in a timely manner, and
- yielding the most economical structure.

The design team looked at three design options. The first option was a 160 ft plus-long precast, prestressed concrete girder that would have to be transported up the narrow canyon of the Banks-Lowman Highway. The second option was a precast, pretensioned and post-tensioned concrete spliced girder that would be shipped in three pieces and spliced together in the field using one stage or two-stage post-tensioning. Finally, a steel plate girder that could be shipped in two or three pieces and field spliced using high strength bolts. The first and third options were selected for competitive bidding.

Technologically, the design of 161-ft-long prestressed girders spaced at 8 ft 10 in. could readily be accomplished. To transport the girders, which were over 7 ft deep, from the casting bed to abutment bearings, was another matter. Any sharp bend in an access highway could, for a very long transporter, nullify the single-girder option. However, input from a concrete girder precaster in the area, a full 50 miles away, indicated that highway geometry would allow the girder's delivery.

Accordingly, the second option was dropped by the design team and a type, size, and location plan for a single-span, 162-ft 4-in.-long and 69-ft 7-in.-wide bridge was developed by the design team for final approval by WFLHD, Forest Services, and the county.

The bridge width of 69 ft 7 in. with an 8½-in-thick concrete deck, comfortably accommodated two, 12–ft-wide through-lanes and one 12-ft-wide right hand turn lane onto Middle Fork Road. In addition, it included an 11-ft 7-in.-wide median, 12-ft 3-in.- and 8-ft 3-in.-wide shoulders, and two 9-in.-wide, crash-tested, guardrail parapets. An 18-ft-long approach slab was also provided at each end of the bridge to alleviate any possible settlement of fill materials at the bridge approaches.

Girder delivery.

BOISE COUNTY, IDAHO, OWNER

BRIDGE DESCRIPTION: A single-span, 162-ft 4-in.-long and 69-ft 7-in.-wide bridge with a deck area that provided 12-ft 3-in.- and 8-ft 3-in.-wide shoulders; three, 12-ft-wide travel lanes; and an 11-ft 7-in.-wide median

STRUCTURAL COMPONENTS: Eight 85-in.-deep Idaho bulb-tee girders, 161 ft 2 in. long, with an 8½-in.-thick cast-in-place concrete deck; two 5.5-ft-deep cast-in-place concrete abutment caps each supported on seven 18-in.-diameter concrete filled pipe piles; and 24-ft long and 14-ft deep wingwalls also supported on concrete filled pipe piles

BRIDGE CONSTRUCTION COST: $1.67 million ($148/ft^2)

Setting girders for stage I (left) and stage II (right).

The girders in the final layout were longer (161 ft 2 in.) than half the length of a football field and stood 7 ft 1 in. tall. These prestressed concrete "bulb-tee" girders were the longest ever designed by any of the Federal Lands Highway (FLH) Bridge Offices. The girders were prestressed with fifty 0.6-in.-diameter, grade 270, seven-wire prestressing strands. Eighteen of those were harped, in 12- and 6-strand bundles, through the 7-in.-thick girder web. The stirrup spacing of 3 and 6 in. extended nearly 11 ft from girder ends to resist shear forces. The required final concrete compressive design strength was 9.0 ksi while the required strength at transfer was 7.6 ksi.

Substructure elements at each abutment included 72-ft 2-in.-long by 5-ft 6-in.-deep concrete caps supported by 14 closed-end, 18-in.-diameter, concrete-filled steel pipe piles at 5 ft 2 in. spacing. The four, 24-ft-long, 14-ft-deep, and 14-in.-thick wingwalls including 6 ft cheek walls were also supported by concrete-filled pipe piles. A mechanically stabilized earth wall was constructed off the northwest wingwall to keep project construction limits within the existing right-of-way.

Traffic Flow

To maintain uninterrupted traffic flow during construction, a two-stage construction strategy was incorporated into the design plans. Stage I retained traffic on the existing bridge while seven pipe piles were driven, the Stage I cap section built, and four girders were set on their bearings. Then, concrete for the 30-ft 8-in.-wide deck, with epoxy-coated reinforcement, was placed. The exposed, transverse reinforcement doweling out from the Stage I deck placement came within a few inches of the existing bridge.

Traffic was then diverted to the completed Stage I deck and the original bridge was demolished and removed. The remaining seven pipe piles were driven, the cap concrete placed, the four girders set, and the 38-ft 11-in.-wide Stage II deck completed including a 4-ft 6-in.-wide closure placement linking the Stage I and II deck sections with their overlapping transverse deck reinforcement. Mechanical couplers were used to connect the transverse reinforcement between the two pile cap sections because the close proximity of the two stages did not allow for lap splices.

The bridge was completed and partially opened to traffic in December 2013 pending minor works to be completed by the summer of 2014.

Summary

The use of precast, prestressed concrete girders contributed to building a structure that is simple, aesthetically pleasing, and blends nicely into the natural environment of the rural Idaho countryside. Prestressed concrete girders also provided the lowest overall cost, considering production and lifetime maintenance and reduced life-cycle costs.

Because of the extensive coordination between the CFLHD and WFLHD Bridge Offices, an optimal bridge solution was implemented. It satisfied the hydraulic requirements with a single span. It accommodated the need, created by the nearby "T" intersection, for an unusually wide bridge deck. It allowed for uninterrupted traffic flow with an efficient and innovative staged construction strategy.

The use of precast, prestressed concrete girders contributed to building a structure that is simple, aesthetically pleasing, and blends nicely into the natural environment of the rural Idaho countryside.

With no doubt, the new Davey's Bridge is one of the signature bridges of the FLH Bridge Office and a tacit tribute to the capabilities and efficiencies of modern bridge engineering.

Samir Sidhom is the project team leader and functional team lead and Greg May is a member of the design team and a structural engineer with the Central Federal Lands Bridge Office of Bridges and Structures, which is part of the Federal Highway Administration's Office of Federal Lands Highway, based in Lakewood, Colo.

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

Types of Epoxy-Coated Reinforcing Steel

by Dr. David McDonald, Epoxy Interest Group

Epoxy-coated reinforcing steel is commonly coated to meet ASTM A775/A775M, "Standard Specification for Epoxy-Coated Steel Reinforcing Bars." Bars meeting this standard are commonly green in color. The second most common standard for epoxy-coated reinforcing steel is ASTM A934/A934M, "Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars." Bars meeting ASTM A934 are typically purple in color. So, why are there two types of products?

Epoxy-coated reinforcing steel was initially evaluated by the National Bureau of Standards (now the National Institute of Standards and Technology) and the first use of epoxy-coated reinforcing steel was in a bridge in Pennsylvania in 1973. In 1981, ASTM A775 was developed for these bars. This early specification only required bars to be bent to 120 degrees without the coating cracking as part of the quality-control procedures. The coatings would occasionally crack if bent more 120 degrees during fabrication, thus requiring the cracks to be repaired.

Research conducted in the 1990s found that the corrosion performance of epoxy-coated reinforcing steel was highly dependent on the manufacturing process, and during the 1990s, specifications were significantly improved to require coatings to pass 180-degree bending without cracking. Additional criteria were added to the ASTM specifications that increased coating thickness and required appropriate surface roughness and bar cleanliness prior to coating. In 1991, the Concrete Reinforcing Steel Institute (CRSI) implemented a voluntary Epoxy Plant Certification Program, which evaluated the ability of plant staff, equipment, and manufacturing processes. Prior to introduction of the CRSI program, backside contamination—a measure of steel cleanliness—would range from 10 to over 60%, but after certification the values were reduced to generally less than 20%. This improvement in cleanliness and assured surface roughness, along with modifications to coating chemistry, led to coatings that were more robust during fabrication.

ASTM A936 reinforcing steel used in the San Francisco - Oakland Bay Bridge. Photo: Caltrans.

ASTM A775 reinforcing steel being placed for the Willamette River Bridge in Portland, Ore. Photo: Harris Rebar, Auburn.

In parallel with the changes occurring in the 1990s for epoxy-coated bars meeting ASTM A775, the Federal Highway Administration and the U.S. Navy recognized the potential for problems with cracked coatings. It was suggested that the problem of coating cracking during bending be mitigated by fabrication of the bars prior to coating. In 1994, Douglas F. Burke wrote an article titled "Epoxy-Coated Rebar in Marine Concrete." This article[1] described construction of a new submarine pier at Pearl Harbor using prefabricated epoxy-coated steel reinforcing bars in accordance with a Naval Facilities Guide specification. ASTM A934 was issued in 1995.

There have been a few side-by-side research programs comparing the performance of bars meeting ASTM A775 and A934. These studies, typically conducted on deliberately damaged bars simulating jobsite damage, have not found substantial differences in corrosion performance for bars meeting either of the two specifications. As bars meeting ASTM A775 are frequently cut after coating, the ends of these bars are required to be patched. Suppliers of ASTM A934 coatings believe that this provides a weakness to the corrosion system. Requirements within ASTM A775 and ASTM A934 are similar except for bendability and for the impact resistance of the coatings, where the requirements for ASTM A934 are less severe than those of the ASTM A775 coating due to increased brittleness of the ASTM A934 coating. The abrasion resistance of the ASTM A934 coating material is better than that of the ASTM A775 material. The patching materials for both products are required to pass similar tests to those for the bar coating material.

Products meeting ASTM A775 are still being utilized for the majority of specifications throughout North America; however, products meeting ASTM A934 are routinely provided to the California Department of Transportation (Caltrans) for use in marine waters and to the U.S. Navy. Caltrans uses products meeting ASTM A775 away from the coast. In the future, increasingly robust, yet flexible coatings will be developed by product suppliers.

Reference

1. Burke, D. F. 1994. "Epoxy-Coated Rebar in Marine Concrete." *Military Engineer*, 10-10.

Dr. David McDonald is managing director of the Epoxy Interest Group and is based in Schaumburg, Ill.

PROJECT

RAMP N-N OVER I-17

Rebuilding the gateway to northern Arizona in Cordes Junction

by David B. Benton, Arizona Department of Transportation

As a regular destination for motorists to the Flagstaff or Prescott region, the junction of I-17 and SR 69 has been described as the gateway to northern Arizona. But in recent years, the outdated Cordes Junction traffic interchange had started to show its age.

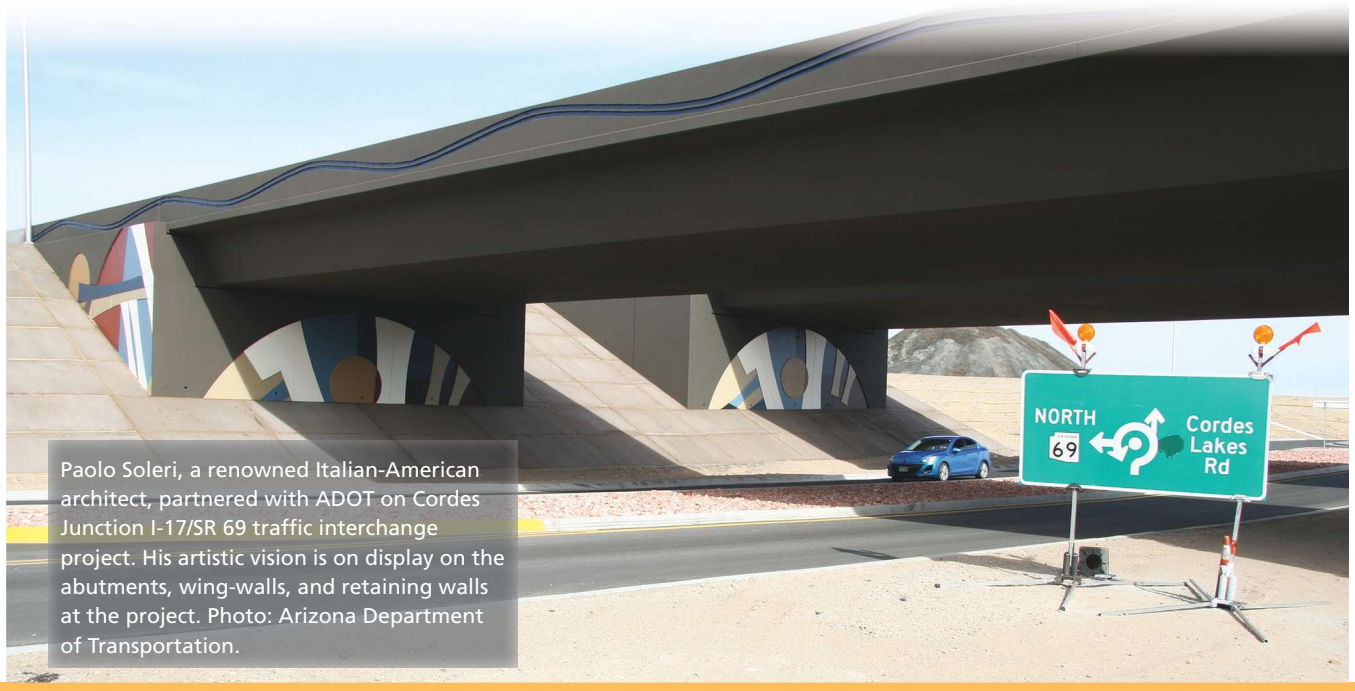
Built in the early 1960s, the Cordes Junction traffic interchange carried far more traffic than it was designed to accommodate. With traffic volumes expected to double in the coming decades, the Arizona Department of Transportation (ADOT) began a daunting task in the summer of 2011 to redesign and rebuild the busy interchange—which is located approximately 65 miles north of downtown Phoenix—with minimal disruption to traffic.

The solution was a reconstruction project that would transform the outdated intersection design and replace it with a modern interchange that could handle the increased traffic in the area. No longer would slower local traffic have to mix with high-speed highway traffic, causing congestion and safety concerns. Two years after construction started, ADOT completed the project on budget and on time in the summer of 2013.

Project Challenges

The junction of I-17 and SR 69 in central Arizona provides a vital link between the metropolitan Phoenix area to the south, the surrounding communities of Humboldt, Dewey, Prescott Valley, Chino Valley, and the city of Prescott to the north. Reconstruction of the Cordes Junction traffic interchange was a key project to ADOT to improve operational characteristics, geometrics, and safety at this interchange.

The existing underpass (UP) at this traffic interchange, known as Cordes Junction TI UP, consisted of a four-span continuous steel girder bridge consisting of five girder lines with maximum spans of 86 ft 3¾ in. and 79 ft 3¼ in. skewed at approximately 31 degrees over northbound and southbound lanes of I-17. The bridge needed to be replaced with the reconstruction of the traffic interchange due to the inability to accommodate the future widening of I-17 and substandard vertical clearance and shoulder widths. In addition, it conflicted with improvements needed at the traffic interchange to facilitate a connector road to SR 69 and the community of Cordes Lakes, east of this location.



Paolo Soleri, a renowned Italian-American architect, partnered with ADOT on Cordes Junction I-17/SR 69 traffic interchange project. His artistic vision is on display on the abutments, wing-walls, and retaining walls at the project. Photo: Arizona Department of Transportation.

profile

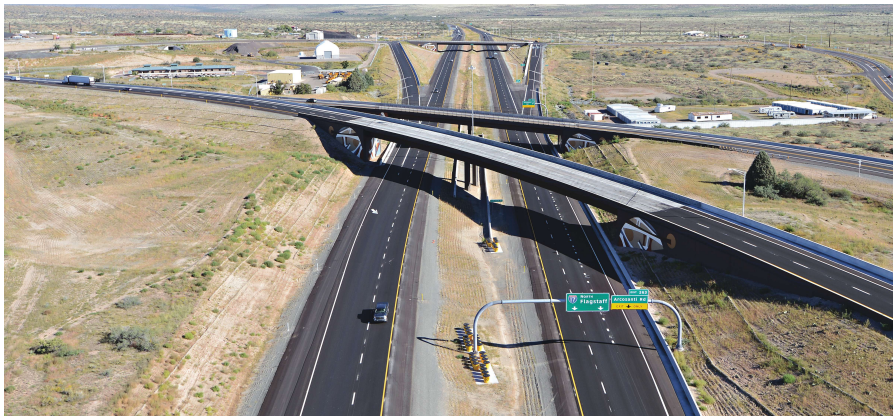
RAMP N-N OVER I-17 / CORDES JUNCTION, ARIZONA

BRIDGE DESIGN ENGINEER: Arizona Department of Transportation Bridge Group, Phoenix, Ariz.

PRIME CONSULTANT: HDR Engineering Inc., Phoenix, Ariz.

PRIME CONTRACTOR: Vastco-Sundt Joint Venture: Vastco Inc., Chino Valley, Ariz., and Sundt Construction, Tempe, Ariz.

POST-TENSIONING CONTRACTOR AND SUPPLIER: DWYIDAG-Systems International, Long Beach, Calif.



Originally constructed in the early 1960s, the Cordes Junction traffic interchange at the junction of Interstate 17/SR 69 needed replacement due to heavy traffic in the area. The new Cordes Junction I-17/SR 69 traffic interchange was designed to improve traffic flow and safety, while separating local and highway traffic. Photos: Vastco-Sundt.

The scope of the project included

- replacing the existing Cordes Junction TI UP,
- replacing adjacent bridges over Big Bug Creek to the south,
- providing a new diamond interchange to the north with access to the unique community of Arcosanti,
- building a new connector road bridge linking the community of Cordes Lakes to SR 69,
- constructing two new overpass bridges on SR 69 linking Cordes Lakes Road and Arcosanti Roads to businesses along Copper Star Road to the west, and
- adding a directional ramp linking I-17 northbound travel to northbound SR 69 known as Ramp N-N.

Delivery Method

During the initial stages of development, ADOT approached the project with a traditional design-bid-build delivery method. After reaching the 30% design milestone and the selected interchange configuration was agreed upon by the project team, it was apparent the project offered many challenges, mainly due to complexity of reconstructing the traffic interchange, maintaining two lanes of traffic open in each direction on a busy interstate, and keeping the connection of I-17 and SR 69.

The total reconstruction of the Cordes Junction traffic interchange was estimated at $50.9 million. With a challenging task of reconstructing a traffic interchange with estimated daily traffic volumes of over 27,000 vehicles on I-17 and 13,000 vehicles utilizing the existing interchange, ADOT was seeking a new and innovative way to approach construction.

The department decided to embark on a fairly new concept to ADOT on construction delivery and construct the traffic interchange utilizing a construction-manager-at-risk approach. This delivery method was the first of its kind at ADOT that involved heavy bridge construction on the interstate, through live traffic, and use of federal funds. The key factors to this decision were maintenance of traffic and having a qualified contractor on-board to perform the construction and provide preconstruction services. This assisted the design team in construction sequencing and traffic phasing to meet the requirements of the project for I-17 travel and keeping the existing interchange accessible.

Bridge Type

One of the key components of the reconstruction of the Cordes Junction traffic interchange is the new directional ramp connecting northbound I-17 travelers to northbound SR 69, known as Ramp N-N. Due to the outdated design of the previous interchange, slower traffic destined for the businesses and residences in the Cordes Lakes area were forced to mix with Prescott-bound traffic at this busy traffic interchange until the flyover bridge opened to traffic in December 2012.

The new flyover ramp consisted of a two-span, 397-ft 9⅞-in.-long structure with unequal spans of 215 and 175 ft to accommodate the ultimate width of I-17 at this location. The bridge is skewed at 40 degrees left, radial to the horizontal curved alignment with a length of 1833.83 ft, a radius of 1660.75 ft, and superelevation of 6%. The Ramp N-N bridge sits within a 1600 ft crest vertical curve with approach and departure grades of 2.04% and -0.62%, respectively.

ARIZONA DEPARTMENT OF TRANSPORTATION, OWNER

BRIDGE DESCRIPTION: Two-span, 397-ft-long, cast-in-place, post-tensioned concrete box girder bridge

STRUCTURAL COMPONENTS: Three-cell, cast-in-place concrete box girder with 14-in.-thick webs spaced at 11 ft on center, with 4-ft 6½-in.-long cantilever deck edges

BRIDGE CONSTRUCTION COST: $182/ft^2

AWARDS: Engineering Excellence Grand Award – American Council of Engineering Companies (ACEC) of Arizona 2013

ADOT selected a cast-in-place (CIP), post-tensioned box girder bridge because it best meets the geometric requirements of this directional ramp, provides the lowest initial construction cost, and has minimal maintenance during the life of the structure. Concrete superstructures are the predominate type utilized in the state of Arizona, due to low cost and readily available concrete materials. They provide a durable and resilient end product, built by a proven construction industry skilled in both CIP and prestressed concrete construction. Full-height abutment walls founded on spread footings were selected to minimize span lengths and limitations of superstructure depth with the use of falsework over I-17.

The pier substructure type was originally designed with a two-column integral pier founded on a single shallow spread foundation. However, through the process of review by the contractor during preconstruction services, it was recommended that the footprint needed to construct a spread foundation would be eliminated by using shallow rock sockets. This freed up valuable space in the median of I-17 to be utilized in temporary traffic shifts towards the median area. Having the contractor on-board working side by side with the design team in a preconstruction role, proved to be an invaluable asset to the project team and helped minimize risk and conflicts in construction that may have possibly delayed construction operations and most importantly, the traveling public.

The design of the CIP, post-tensioned concrete box girder bridge consisted of a 9-ft 3-in.-deep, three-cell box with webs spaced at 11 ft on center, augmented with sloped exterior webs at a 4 to 1 ratio and 4-ft 6½-in.-long deck cantilevers. The bridge deck thickness was set at 9 in., based on clear span requirements in ADOT *Bridge Design Guidelines*. A unique update to the ADOT *Bridge Design Guidelines* that was incorporated into this project was a requirement to use a minimum web thickness of 14 in., based on the depth of the superstructure meeting or exceeding 9 ft. Past experience with consolidation of concrete in webs with standard thicknesses of 12 in. around post-tensioning ducts with deep superstructures exceeding 8 ft had been an issue on some past ADOT projects and the update in the requirements was in line with commentary stated in the AASHTO *LRFD Bridge Design Specifications*, to help rectify some of these concerns. The bottom slab thickness was 6 in. The total post-tensioning force specified for this design was 15,620 kips utilizing four ducts per web with twenty-two 0.6-in.-diameter strands per duct.

Heavy cranes work on the Ramp N-N constructed abutment wingwall with box girder falsework over Interstate 17. Photo: Arizona Department of Transportation.

This photo shows the Ramp N-N median where the pier columns and abutment are being constructed. Photo: HDR.

AESTHETICS

Aesthetic treatments for the bridge were provided utilizing one of the last works by famed Italian-American architect Paolo Soleri, a one-time apprentice under Frank Lloyd Wright.

Soleri, who established Arcosanti, an artists' community and a popular tourist attraction located just two miles northeast of the traffic interchange, served as a project consultant for ADOT prior to his death on April 9, 2013, at the age of 93. Soleri's artistic vision is now on display on many of the retaining walls and abutments on the bridges at the interchange. While most of his artwork was completed prior to his death, his visionary designs will be on display for drivers who pass through the area for decades.

David B. Benton is the bridge design manager for the Arizona Department of Transportation in Phoenix, Ariz.

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Photo: E.L. Robinson Engineering of Ohio Co.

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PRECAST CONCRETE SUBSTRUCTURES

A COMPARISON OF NON-SEISMIC AND SEISMIC CONNECTION DETAILS

by Dr. M. Lee Marsh, BergerABAM; Michael P. Culmo, CME Associates; and Dr. John F. Stanton, University of Washington

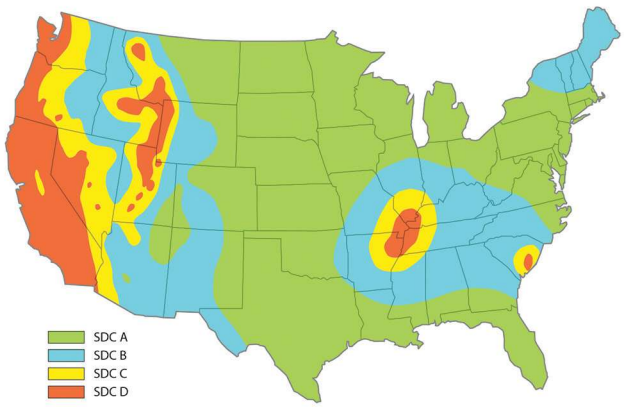

Figure 1: AASHTO seismic design category boundaries for Site Class E soils.[3] Figure: BergerABAM.

The development of accelerated bridge construction (ABC) techniques has been a focus of the American bridge design and construction communities for some time. Recently, the use of precast concrete substructure elements in bridge construction has vastly expanded. Their use, however, poses some particular challenges in seismically active regions, including development of seismic design methods, details, and specifications to address seismic performance issues.

The purpose of this article is to address seismic performance for such systems at a high level to assist the bridge engineering community in translating knowledge of the seismic performance of cast-in-place concrete bridges to precast concrete construction. This article focuses on desired performance and design philosophy for seismic loading. It is useful to understand the objectives of seismic design in the higher seismic regions in order to apply appropriate design principles in the lower and the so-called "non-seismic" regions of the country.

Design Philosophy and Procedure

For purposes of specifying design procedures whose rigor is directly related to the seismic hazard, the AASHTO design specifications[1,2] partition the country into four zones or design categories. For simplicity, this article focuses on the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* (SGS), which is a displacement-based design specification. The seismic design categories (SDCs) range from SDC A (lowest) to SDC D (highest). The boundaries between SDCs for the lower 48 states are shown in Figure 1 for Site Class E.[3]

The seismic design philosophy used for bridges is known as capacity design. The basic tenet is for the provision of an internal fuse or plastic mechanism that limits the forces that can be transmitted to adjacent elements, such that

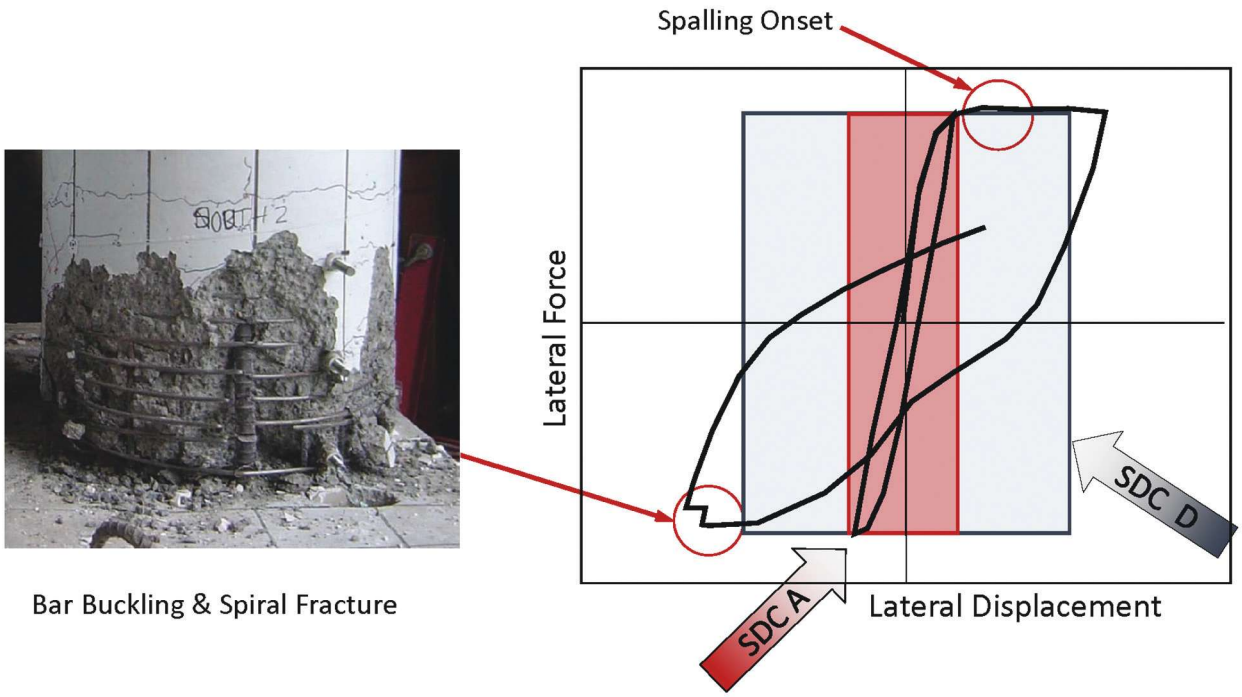

Figure 2: Damage expected from cyclic loading relative to seismic design category A and seismic design category D demand levels. Figure: BergerABAM.

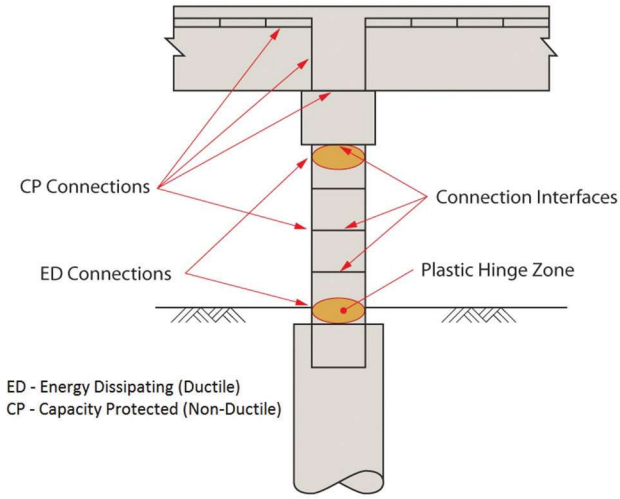

Figure 3: Example connection locations in a precast concrete bridge.[4] Figure: BergerABAM.

precise prediction of elastic seismic forces is unnecessary. The concept is illustrated by a chain, in which one link is designed to be ductile and to constitute the fuse. If all the other links are designed to be stronger than the ductile link, the performance of the whole chain will then be ductile.

For bridges, the plastic mechanism should occur in the substructure, because the girders are typically much larger and stronger than the substructure members. Capacity design protects all the elements other than the fuse against both damage and girder unseating. It accomplishes this by concentrating the inelastic behavior in the fuse element itself, which is designed to sustain the inelastic demands.

The substructure elements that form the fusing mechanism are expected to deform inelastically (plastically). Depending on the magnitude of such deformation, these elements will require adequate ductility capacity to achieve the necessary displacement capacity. This ductility capacity depends on the structural detailing, which must, therefore, be different in the low and high SDCs. The difference in potential seismic demand levels between SDC A and SDC D is shown in Figure 2.

Understanding the capacity design concept is essential, even when designing bridges in the lowest SDC, because the designer must control and suppress unintentional modes of seismic failure. In the higher SDCs, the substructure elements must be provided with relatively large ductility capacities, which is typically achieved by

- providing adequate confining transverse reinforcement,
- restricting the location of reinforcement splicing, and
- providing sufficient member shear capacity.

In the intermediate SDCs, less ductility capacity is necessary because seismic deformations are smaller, especially when the AASHTO SGS is used. Following the same logic, structures in the lowest SDC do not require significant ductility capacity, but they do require that the lateral strength of the substructure can be developed. This requirement is not directly evident to the designer in cast-in-place concrete construction, but it

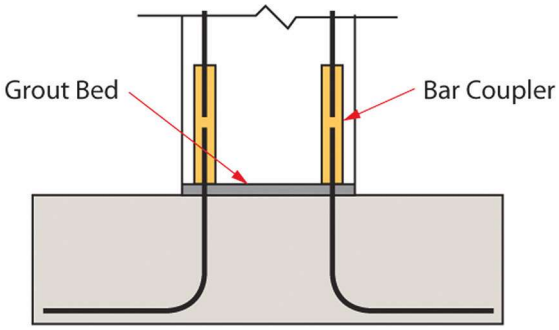

Figure 4: Bar coupler connection schematic.[4] Figure: BergerABAM.

Figure 5: Bar coupler application in seismic design category A. Photo: CME Associates.

must be recognized in precast concrete construction because connections between precast elements may introduce unintentional points of weakness if not properly detailed. Most precast concrete detailing is based on emulation design principles, where the precast concrete joint is designed and detailed to emulate a cast-in-place concrete construction joint.

Connections

Figure 3 illustrates example locations of connections between precast concrete elements for a bridge with precast concrete columns, cap beams, and superstructure. The design of connections in such plastic hinging zones must recognize the energy dissipating (ED) or inelastic nature of these locations. These ED connections often are located where the maximum internal forces and ductility demands occur.

In contrast, connections located away from such ED zones only need to be able to sustain the forces corresponding to the strength of the fusing mechanism. Such connections are known as capacity protected (CP) connections. In the three

Figure 6: Erection of bent cap with a grouted duct connection. Photo: BergerABAM.

highest SDCs, the ED connections must supply ample ductility capacity. In the lowest SDC, expected ductility demands are lower as seen in Figure 2. Thus what in the highest SDC might need to be an ED connection would in the lowest SDC need only develop the strength of the adjacent members.

The AASHTO provisions for the lowest SDC focus on prescriptive connection forces and support lengths to prevent unseating of spans under earthquake loading. The provisions presume that adequate lateral strength will be present. Until 2014, the AASHTO provisions required that the prescriptive connection forces be carried into the foundation. Changes were recently approved to clarify that the prescribed forces are only intended to provide sufficient connection capacity to prevent unseating. Additional commentary will be provided to guide the designer through a rational design process, which satisfies the performance objectives outlined in this article. The improved specification language should help designers of precast concrete substructures produce designs that satisfy capacity design principles.

Connection types for precast concrete substructure construction were categorized and discussed in detail in the recent NCHRP Report 698.[4] Several of those connection types are discussed herein: bar coupler, grouted duct, pocket, and socket connections. All four of these connection types are designed to be used as ED connections, although specific assembly details can affect the connection's seismic efficacy. Such details are beyond the scope of this article, but are given in the referenced documents.

Bar Coupler Connection

Bar coupler connections use mechanical couplers in one precast concrete element and mating protruding reinforcement in the adjacent precast concrete element. Once assembled the couplers are grouted. This arrangement is illustrated in Figures 4 and 5. Coupler connections have been widely used in lower SDCs and are currently being evaluated for use in the highest SDCs. Precast concrete elements with this type connection are easy to handle, although tolerances must be carefully selected for good fit up.

Figure 7: Grouted duct arrangement with integral cap beam and column.[5] Photo: BergerABAM.

The strength of these couplers, when placed in the plastic-hinge zone, has been proven in several laboratory tests, which means that they potentially can be used without restriction in SDC A bridges. For higher SDCs, the column must also

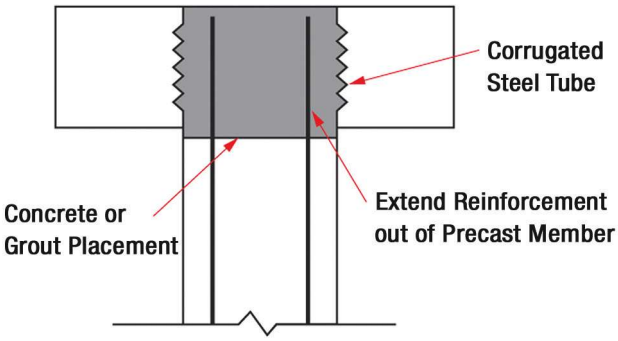

Figure 8: Pocket connection schematic.[4] Figure: BergerABAM.

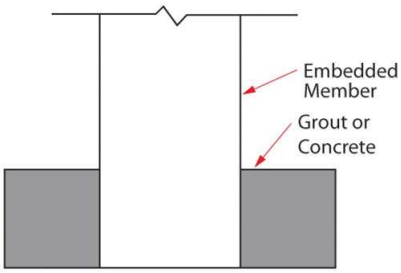

Figure 9: Socket connection schematic.[4] Figure: BergerABAM.

Figure 10: Socket connection application in seismic design category D. Photo: BergerABAM.

possess ductility if it is to act as a fuse element, and the effect of the couplers on ductility capacity is currently under investigation. One potential approach to this issue may be to place the couplers outside the hinge zone, for example within the footing and pier cap. Details for grouted couplers have been developed by and are available through the PCI Northeast Bridge Technical Committee (see page 40 for website address).

Grouted Duct Connection

Grouted duct connections consist of metal ducts in one element to anchor reinforcing bars from the other element and to splice to other reinforcement. The ducts typically have a larger diameter than the corresponding bar coupler arrangement, thereby easing tolerances. Experimental work has shown that these connections have the capacity to develop large diameter reinforcing bars in a relatively short length due to the high confinement offered by the duct. These connections were used in the recently completed FHWA Highways for LIFE project in Washington State.[5] Figures 6 and 7 illustrate the arrangement of such connections for integral column-to-cap beam connections.

Pocket Connection

Pocket connections use a single, large-diameter corrugated pipe to form an opening in the cap beam. Longitudinal bars projecting from the top of the column are fitted into the opening, which is filled with concrete, as shown in Figure 8. Compared with the use of individual ducts for the columns bars, the use of a single opening simplifies the fit-up, but it weakens the cap beam and may require external temporary supports during erection. The connection produces a nicely confined joint region, and thus can reduce reinforcement congestion.

Socket Connection

The fourth connection type is the socket connection, which works well for column bases, as seen in Figures 9 and 10. An extended column with roughened sides is positioned in the foundation and concrete is placed around the column. Reinforcing steel that would normally pass beneath the column is placed to the sides of the column, and shear friction provides the gravity force transfer mechanism. Socket columns may be used with either shallow or drilled shaft foundations, and both configurations have been tested to verify both the gravity load-carrying capacity and their seismic performance.[4]

Summary

All four of these connections are capable of developing ductile plastic hinges in the column, and the behavior is essentially emulative of cast-in-place construction. The research performed on these connections has provided recommended design provisions that can be used with the AASHTO SGS. Currently, NCHRP project 12-102 is assembling this information to produce a single guidance document for precast substructure seismic design. The project is scheduled for completion in 2016.

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Dr. M. Lee Marsh is president/CEO of BergerABAM in Federal Way, Wash.; Michael P. Culmo is vice president of CME Associates in East Hartford, Conn.; and Dr. John F. Stanton is a professor at the University of Washington in Seattle, Wash.

EDITOR'S NOTE

This article provides synopses of two webinars given through the Accelerated Bridge Construction University Transportation Center (ABC-UTC) in May and June 2014. Recordings of these webinars may be accessed through the link below. http://abc-utc.fiu.edu/index.php/technology/monthly_webinar_archive/

Post-Tensioning Grouts: Challenges and a Path Forward

by Reggie Holt, Federal Highway Administration

Two recent issues with post-tensioning (PT) grout have generated concern over the long-term performance of PT tendons. These two independent issues occurred close to the same time, which has led to confusion by many in the bridge community. This article will provide background information on these two issues, as well as measures developed to address each issue.

Elevated Chlorides

In 2010, the discovery of PT grout with elevated chloride levels in a straddle bent cap in Texas triggered a follow-up investigation by the grout manufacturer, Sika. Their investigation revealed that its SikaGrout 300PT product was sometimes produced at its Marion, Ohio, plant with levels of chloride compounds that exceeded the American Association of State Highway and Transportation (AASHTO) and the Post-Tensioning Institute (PTI) specified limit of 0.08% by weight of cement. Approximately 24 million pounds of SikaGrout 300PT were produced at this plant from 2001 to the time production ceased in 2010. At that time, approximately 16 million pounds were used in bridge applications in the United States.

The investigation revealed that cement provided by a third-party supplier, and used in the SikaGrout 300PT production, was the source of the chlorides. Chloride levels ranged from well above to well below the specified limit. The governing ASTM specification for portland cement, ASTM C150, does not include limits on chloride. In addition, the AASHTO and PTI specifications only specified chloride thresholds for PT grouts, not for constituent materials of those grouts. Consequently, this issue went undetected for many years with very limited chloride level data being captured for PT grouts. The PTI M55.1-12 *Specification for Grouting of Post-Tensioned Structures* has since added requirements for frequent chloride testing during production.

Subsequent to the identification of this issue, Sika and Federal Highway Administration (FHWA) each started independent efforts to collect chloride data, identify end users, and perform research to determine the long-term performance of PT strand exposed to high levels of chloride. Results from these efforts were used to develop an FHWA technical advisory (TA) on assessing and managing the long-term performance of post-tensioned bridges having tendons installed with grout containing elevated levels of chloride (www.fhwa.dot.gov/bridge/t514033.pdf). This TA, released in November 2013, provides guidance on identifying SikaGrout 300PT end-users, assessing PT tendon service life risks, as well as recommended management practices.

Post-tensioning tendon corrosion testing on stressed tendons. Photo: FHWA Turner Fairbank Research Center.

Segregated PT Grout

A second issue deals with segregation of prepackaged, thixotropic PT grouts during or immediately after the grouting of PT ducts. PT grouts commonly used in the United States have sporadically been observed to exhibit this condition. When it forms, the grout matrix is typically separated into three common layers, with a top layer of soft (putty-like) grout, a middle layer of friable grout, and a lower layer of hard grout. The upper soft layer in segregated grout is highly corrosive and is therefore of great concern. Segregated grout formations are typically isolated to tendon high point and anchorage locations with volumes generally varying from minimal to filling the upper third of the of the PT duct along a limited length

Segregated grout at post-tensioning tendon end anchorage. Soft grout is located in upper layer. Photo: Texas Department of Transportation.

Inspectors visually examining a post-tensioning tendon using a boroscope. Photo: Structural/VSL.

within the tendon or anchorage.

There are multiple research projects underway investigating segregated grouts, as well as research developing new qualification testing procedures that can identify PT grouts susceptible to segregation. Preliminary research results and observations from in-place tendons have revealed a strong correlation between the creation of segregated grout and excess water in the grout matrix. Observations have also shown that the use of inert material in the grout mix, the use of expired pre-bagged grout, the inadequate mixing of the grout, and tendons with highly draped tendon paths can further increase the susceptibility to the occurrence of segregated grout. Equally noticeable is the non-occurrence of segregated grout in PT tendons with moderate to flat profiles. One can see that each of the previously listed features could generate excess/free water by either reducing the ability of the mixed-in water to react with materials in the grout or alter the dispersion of water within the grouted tendon.

The PTI M55.1-12 *Specification for Grouting of Post-Tensioned Structures* was recently updated to address concerns with grout segregation and strengthen its bleed water provisions.[1] In addition, extra care should be taken before tendon grouting to ensure that excess water is not inadvertently introduced into the PT grout from residual water in the grout mixer, conveyance, or duct. Because segregated grout formation is typically isolated to tendon high points and anchorages, post-grouting inspections at these locations are highly recommended. More information is needed to fully understand segregated grout; however, following the above requirements and recommendations should both greatly reduce the occurrence of segregated grout and provide a means to identify segregated grout during bridge construction when it occurs.

Going Forward

These rare but worrisome issues are unfortunate; however, they should not contaminate the excellent track record of our robust and well-performing post-tensioned bridges and structures. These issues have highlighted an overreliance on proper manufacturing and field installation of this highly engineered product, leading the bridge community to focus additional efforts toward replacement, inspection, and monitoring of PT tendons. Fortunately, many technologies that can advance the post-tensioning state-of-practice are available and the FHWA, industry, and state departments of transportation are working together on researching and developing PT tendons with these desired attributes.

Reference

1. Hunsicker, G., T. L. Neff, and M. F. Vejvoda. 2014. "Further Improvements in Post-Tensioning Grout Materials" *ASPIRE*, Winter, Precast/Prestressed Concrete Institute: Chicago, IL. pp 40-41.

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

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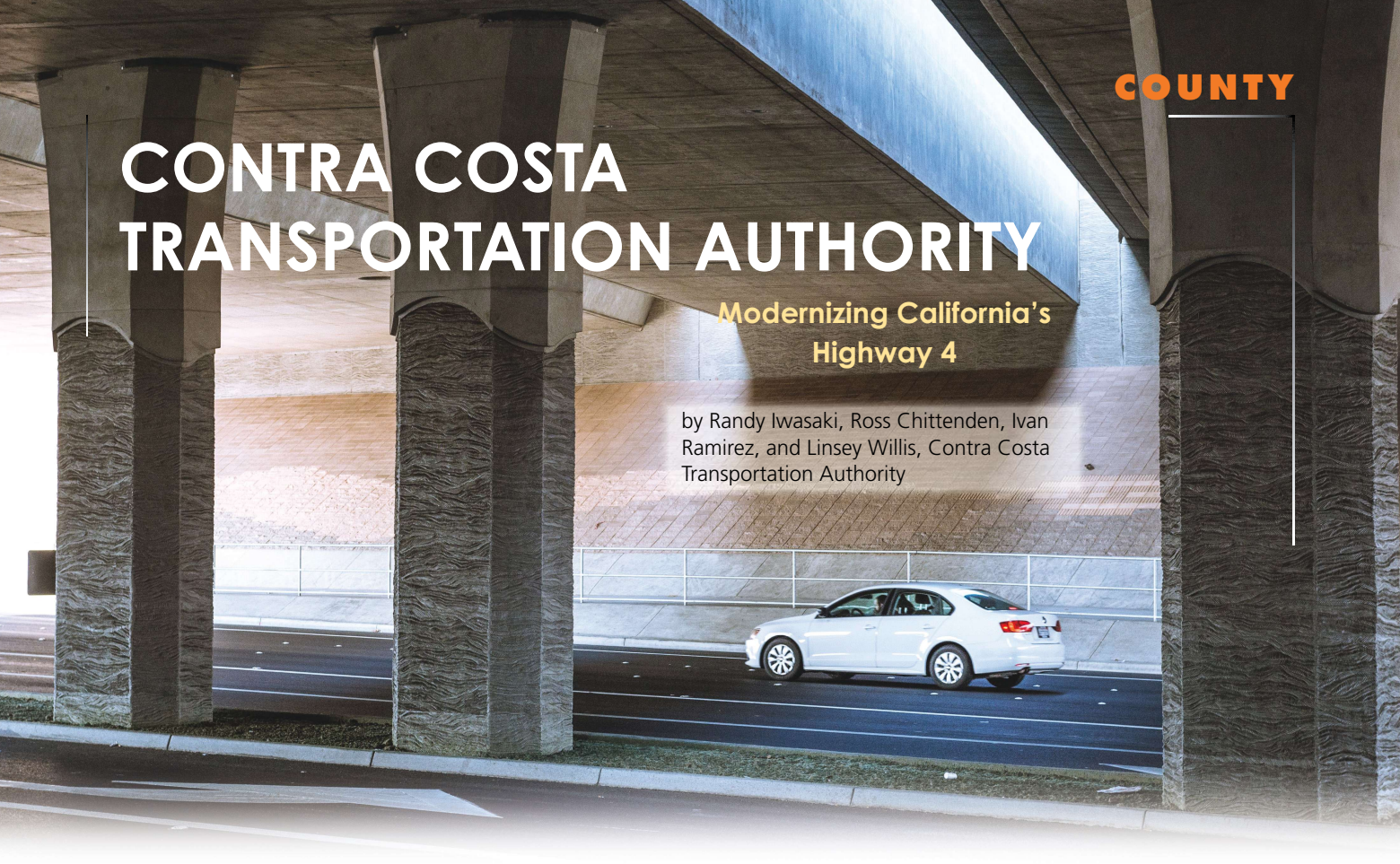
CONTRA COSTA TRANSPORTATION AUTHORITY

Modernizing California's Highway 4

by Randy Iwasaki, Ross Chittenden, Ivan Ramirez, and Linsey Willis, Contra Costa Transportation Authority



Detail of the Highway 4 overcrossing at the Somersville Road intersection in Antioch, Calif. All Photos: Contra Costa Transportation Authority.

The Highway 4 Corridor Project, led by California's Contra Costa Transportation Authority, will expand and modernize Highway 4 to accommodate increased traffic volumes and future construction of commuter rail in the fast-growing eastern portions of Contra Costa County, near San Francisco. The project expands Highway 4 from four to eight lanes between Loveridge Road in Pittsburg to just west of State Route 160 in Antioch. The project also

- expands the highway from two to four lanes from Lone Tree Way to Balfour Road in Brentwood,
- adds missing connector ramps at the State Route 160/Highway 4 interchange, and
- adds a commuter rail extension from Pittsburg to Antioch (called eBART).

The Loveridge Road overcrossing at Highway 4 in Pittsburg, Calif. The center median area provides space for commuter rail.

The upgrades to Highway 4 include replacing structures at five interchanges and two other local road crossings. These new bridges have wider lanes, new sidewalks, and smoother exit and entrance ramps to keep traffic flowing. Most bridges in the corridor are cast-in-place, prestressed, reinforced concrete box girder bridges supported on cast-in-drilled-hole concrete piles. All of the bridges are roughly 16 ft above the existing grade-level freeway. To achieve high-quality construction, concrete for the decks and girders for the SR 160/Highway 4 connector ramps is required to have a design compressive strength of 5.0 ksi, while the columns, abutments, and piles have a design compressive strength of 4.0 ksi.

A key challenge of modernizing Highway 4 in eastern Contra Costa County is keeping 250,000 daily trips moving while the work is underway. To address this challenge, a variety of techniques was employed, including careful construction staging, using precast and preassembled pieces, and performing significant and disruptive work at night time as much as possible. Examples of this include the recent construction of a foundation for the SR 160/Highway 4 connector ramps project. At one point, a 13-ft-diameter, 97-ft-long reinforcing bar cage was lifted into place overnight.

The Contra Costa Transportation Authority is a public agency formed by Contra Costa voters in 1988 to manage the county's transportation sales tax program and provide countywide transportation planning. The Highway 4 Corridor Project is expected to be completed in late 2015, with the eBART line on track to be operational by 2017. The project is funded by Contra Costa County Measure J sales tax, regional bridge toll funds, local government contributions, and state and federal grants. More information can be found at 4eastcounty.org.

Randy Iwasaki is the executive director; Ross Chittenden is the deputy executive director, projects; Ivan Ramirez is the engineering manager; and Linsey Willis is the director, external affairs with the Contra Costa Transportation Authority in Walnut Creek, Calif.

Accelerated Service Testing of Prestressed Concrete Railroad Spans

by Duane Otter, Transportation Technology Center Inc.

Precast, prestressed concrete bridge spans are highly popular for railroad bridge construction. They are an economical choice for short-span bridges and generally require little structural maintenance.

The Transportation Technology Center Inc. (TTCI) is currently testing two ballasted-deck concrete bridges at the Facility for Accelerated Service Testing (FAST) in Pueblo, Colo. FAST is a railroad test track with train cars that have a gross rail load of 315,000 lb per car, about 10% heavier than the revenue service standard. The test train at FAST applies traffic at about three times the rate of a typical main line, providing accelerated testing of railroad track and structural components.

The two prestressed concrete bridges installed at FAST in late 2003 have accumulated more than 1400 million gross tons of traffic (about 9 million load cycles). This is comparable to nearly 30 years of traffic on many revenue service lines.

Figure 1 shows the two-span conventional concrete bridge, with 24- and 32-ft-long double-cell, box girder spans. Double-cell box girder spans are commonly used in North America. The spans were provided from railroad inventory stock. Figure 2 shows the cross section of one-half of a typical double-cell box girder span. The double-cell design features an integral deck and ballast curb. Two of these units are set side by side to form a span with four cells and ballast curbs to the outside.

These precast concrete units are well-suited for replacing existing timber bridges with work windows of only a few hours between trains. Once new foundations are in place (typically driven piles with concrete caps), old spans can be cut out and new spans can be placed quickly.

Figure 2: End view of a half-section of a precast, prestressed concrete, double-cell box girder span.

Figure 1: Two-span, conventional concrete bridge. All photos: Transportation Technology Center Inc.

Figure 3: Three-span, precast concrete bridge built using high-performance concrete.

Track panels and ballast can be set in place, rails connected, and track surfaced, all within a matter of hours. Work can progress a few spans per day as traffic allows, with minimal interruption to service schedules. In case of an unplanned service outage, a new concrete trestle can be constructed in a matter of days.

Figure 3 shows the three-span, state-of-the-art concrete bridge. It features a 42-ft-long center span made with high-performance concrete (HPC). This is one of the first documented uses of HPC for a railroad span. The railroad that provided this span chose HPC for its improved freezing and thawing resistance on lines in northern climates. The 42-ft-long double-cell, box girder HPC span is flanked by a 15-ft-long slab span and a 30-ft-long double-cell, box girder span, both designed to a common standard used by two large North American railroads. These are some of the first spans built to that standard.

Use of standard designs for spans provides a host of benefits; the engineering for each bridge generally requires little more than a foundation location plan. Production costs are minimized with large quantities. It is practical to maintain an inventory of spans that can be used in emergencies. If budget priorities change, spans cast for one job can be used on another job.

While the testing at TTCI has included structural performance, the primary research effort has been to reduce track maintenance requirements on concrete bridges.

The structural performance of these prestressed concrete spans has been excellent to date. No maintenance has been required and no significant defects have been noted.

Duane Otter is a principal engineer with the Transportation Technology Center Inc. in Pueblo, Colo.

Francis Avenue Bridge U.S. 395 North Spokane Corridor

by Michael Rosa and Paul Kinderman, Washington State Department of Transportation

Aerial view of the Francis Avenue Bridge located in the north Spokane corridor in Washington state. All Photos: Washington State Department of Transportation Visual Engineering Resource Group (VERG).

When fully complete, the north Spokane corridor in Washington state will be a 10.5-mile-long, north/south, limited-access facility. The new alignment will connect I-90 at the south to U.S. 395 at the north. Planning for the mega project started prior to the World War II era and has resulted in 5.7 miles of new roadway to date. Construction has required dozens of new bridges, the majority of these using precast, prestressed concrete girders. All bridges include ambitious architectural designs.

Francis Avenue Bridge is 455 ft in length and 101 ft wide (curb to curb) with four spans of varying length. The bridge carries five lanes of traffic with bike lanes and sidewalks on each side. The main challenge during the initial design was taking an at-grade railroad crossing and elevating the road over the tracks and future U.S. 395 alignment. Due to existing intersections near each end of the bridge, significant grade changes (+7.38% to –2.98%) were required to maintain minimum vertical clearances over the existing railroad and the future relocated railroad.

To help minimize the vertical profile, Washington State Department of Transportation (WSDOT) deck bulb-tee series girders (W41DG and W53DG) were utilized with a 5-in.-thick, cast-in-place concrete topping. Twenty-four girder lines placed flange to flange were used, requiring 96 total girders. This girder type also reduced the amount of temporary falsework over the active railroad tracks by eliminating the need for deck forms. Spread footings were used at piers 1 and 5, while piers 2, 3, and 4 were founded on drilled shafts primarily to reduce the impact on the adjacent railroad tracks. The girders were designed to envelope the worst-case scenario with and without sidewalks and changing the number of design lanes from six to eight.

Additionally, the geometry and architectural requirements of the bridge required special attention and detailing. Coordinating the locations of the gateway columns, fence posts, barrier joints, luminaire mounts, junction boxes, and future sidewalk lighting conduit stubs were critical. Notably, the outside girder top flange is purposefully exposed to view, creating strong horizontal lines visible as part of the traffic barrier elevation view.

The columns are designed to extend outside the bridge deck. They rise above the sidewalk to become "people places." They also obscure differences in superstructure depth while providing stylized abstractions of diesel engines.

This photo shows how the different girder depths are obscured by the vivid railroad motif columns. Inset: Column design.

Michael Rosa is a senior bridge engineer and Paul Kinderman is a state bridge and structures architect, both with the Washington State Department of Transportation in Tumwater, Wash.

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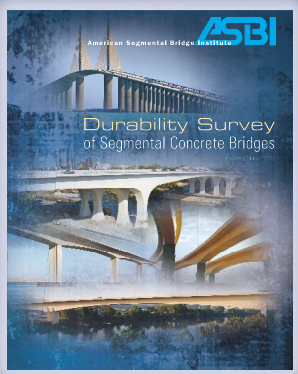

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CONCRETE CONNECTIONS

Concrete Connections is an annotated list of websites where information is available about concrete bridges. Fast links to the websites are provided at www.aspirebridge.org.

IN THIS ISSUE

www.virginiadot.org/projects/bristol/corridor_q_route_460_connector_-_phase_i.asp
Visit this Virginia Department of Transportation website for more information about the U.S. 460 Connector described on pages 10 to 13. A link to an on-site webcam is available.

www.wfl.fhwa.dot.gov/projects/id/daveys/
This website provides an overview of the Davey's Bridge replacement described on pages 18 to 20.

http://www.pcine.org/index.cfm/resources/bridge/Accelerated_Bridge_Construction
For details about grouted couplers mentioned on page 31, go to this Precast/Prestressed Concrete Institute Northeast website and click on "Guideline Details for Precast Concrete Substructures."

http://abc-utc.fiu.edu/index.php/technology/monthly_webinar_archive/
This website contains the two webinars mentioned in the ABC article about precast concrete substructures on pages 29 to 32.

www.trb.org/main/blurbs/168670.aspx
NCHRP Synthesis Report 440 titled *Performance-Based Seismic Bridge Design* and referenced on page 29 is available at this website.

www.trb.org/Main/Blurbs/165909.aspx
NCHRP Report 698 titled *Applications of Accelerated Bridge Construction Connections in Moderate-to-High Seismic Regions* and referenced on page 31 is available at this website.

www.fhwa.dot.gov/hfl/pubs/hif13037/hif13037.pdf
Go to this website for a copy of the FHWA Report titled *Precast Bent Systems for High Seismic Regions* referenced on page 31.

www.aar.com/tracks.php
This website contains information about the test tracks at the Transportation Technology Center mentioned on page 38.

www.fhwa.dot.gov/bridge/t514033.pdf
The FHWA technical advisory titled *Recommendations for Assessing and Managing Long-Term Performance of Post-Tensioned Bridges having Tendons Installed with Grout Containing Elevated Levels of Chloride* mentioned in the FHWA article on page 34 is available at this website.

www.4eastcounty.org
Visit this website for more information about the Contra Costa Transportation Authority Highway 4 Corridor Project described on page 36.

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

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

www.concretebridgeviews.com
This website contains 75 issues of *Concrete Bridge Views* (formerly *HPC Bridge Views*), an electronic newsletter published jointly by the FHWA and the NCBC to provide relevant, reliable information on all aspects of concrete in bridges.

NEW www.fhwa.dot.gov/bridge/lrfd/webinar.cfm
This FHWA website provides access to 12 webinars about implementation of the Load and Resistance Factor Rating Method.

NEW www.trb.org/main/blurbs/171058.aspx
The Virginia Department of Transportation has released a report that evaluates the extent that lightweight high-performance concrete with slag cement can be used in prestressed concrete bulb-tee beams and decks on bridges.

NEW http://tinyurl.com/UDOTLL
At this website, the Utah Department of Transportation provides a series of reports about lessons learned on their ABC projects. These include project specific reports and four overall performance reports issued between 2009 and 2013.

Bridge Research

NEW www.concreteresearchnetwork.org/
The American Concrete Institute Foundation has established a Concrete Research Network at this website to provide a forum for collaboration among funders, researchers, and users that will generate research proposals in response to industry needs, disseminate research findings, and champion the process for adoption of new or improved methods of concrete design and construction.

NEW www.trb.org/main/blurbs/171007.aspx
This website contains a Colorado Department of Transportation report about the evaluation of four bridge deck sealers commonly used on highway bridges.

NEW www.trb.org/main/blurbs/171013.aspx
The Virginia Department of Transportation has released a report that compares the durability of concrete mixtures containing supplementary cementitious materials by evaluating their permeability, absorption, and corrosion resistance.

NEW www.fhwa.dot.gov/research/tfhrc/projects/projectsdb/
This database contains a list of projects sponsored or conducted by the Federal Highway Administration's Turner-Fairbank Highway Research Center and the Exploratory Advanced Research Program. It is updated as new information is collected.

2016 Call for Papers–

Abstracts due February 6, 2015

Photo: Randall Lee Schieber Photography, Columbus, OH

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Abstracts should be submitted electronically. Visit www.pci.org and click on Call for Papers to access the submission site.

The 2016 Call for Papers submission site will open November 2014.

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wnickas@pci.org

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Pretensioning Strand Profiles: Harped or Straight

by Dr. Maher Tadros, eConstruct

Pretensioning is a common method of prestressing in the United States. It requires the use and set-up of a prestressing bed, normally in an off-site plant. The strands are tensioned in the bed between stressing heads. When concrete reaches adequate strength, the strands are detensioned, thus transferring the prestressing force from the prestressing bed to the concrete product.

A large amount of prestressing gives the midspan of a flexural member a large reserve capacity to resist gravity loads. In order to control flexural stresses at the member ends, where the stresses from the girder weight are less, some of the prestressing effects near the member ends must be reduced. Due to the nature of pretensioning, the tensioned strands must follow a series of straight lines. The two commonly used methods of stress control are harping and debonding.

Commercial harping device. Photo: Concrete Industries Inc.

End of debonding tube sealed with duct tape to the strand. Photo: Standard Concrete Products.

Harped Strands

The word "harped" is synonymous with the words "draped" and "deflected." Commercial hold-down devices may be used to drape the strands. The strands typically pass through rollers at a spacing of 2 by 2 in. In the Northwest, the strand group passes as a bundle through a tube and is then spread out into a 2- by 2-in. spacing pattern at the end of the member. In both types, the hold-down device is anchored to the bed pallet using threaded rods. When the prestress is ready to be transferred, the rods are cut before the rest of the detensioning proceeds. The designer should be aware of the limitations of the production facilities near the bridge location, in relation to the amount and location of the hold-down force.

A 205-ft-long girder used for the Alaskan Way Viaduct using bundled harped strands. Photo: Concrete Technology Corporation.

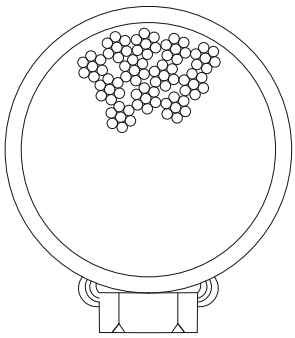
A 191-ft-long bridge girder used for the Cody Avenue Bridge in Okaloosa County, Fla., which used a straight strand profile during production. Photo: Dura-Stress Inc.

The pretensioning arrangement for the 205-ft-long, 100-in.-deep Alaskan Way Viaduct girders in Seattle, Wash., had 46 straight and 26 harped strands in the bottom of the girder. The harped strands were held down in five bundles consisting of four sets of six strands each and two strands, that were spread out into a 2- by 2-in. spacing at the girder ends. Each girder also had eight temporary top flange strands that helped control the top fiber stresses at time of prestress transfer and member camber making a total of eighty 0.6-in.-diameter strands. In Washington State, it is a common practice to drape about one third of the total number of bottom strands. The top strands also aided in the stability of the long, slender girders during handling and shipping. They were bonded only in the end 10 ft. They were cut after the girders were erected and braced and before the intermediate diaphragms were cast.

Advocates of harping indicate the desire to preserve the prestressing level throughout the member length.

Straight Strands

This process is a simpler one as it does not involve harping. For it to work, however, some of the strands at the members ends may be required to be "debonded," also known as "shielded" or "blanketed." Strand debonding is done by enclosing the strands in plastic sleeves. Slit sleeves have been used in the past to allow the sleeves to be placed on the strands after the strands have been tensioned and anchored.



Example of tube for bundling of strands at the harp point. Figure: *PCI Bridge Design Manual*.

A preferred solution is to use a full tube to ensure no concrete paste leaks into the space between the tube and the strands, causing unintentional bonding. In addition, some producers tape the end of the tube to the strand.

There are specific requirements for debonding given in the AASHTO *LRFD Bridge Design Specifications*. These call for a maximum debonding of 25% of the total number of strands, but no more than 40% debonded in any given row. These requirements are followed by most agencies. There are reports of 50% debonding in box beams and as much as 75% debonding in some U-shaped beams. In some European countries, for example Spain, there is no limit on the amount of strand debonding. In the United States, the interest in debonding has resulted in an ongoing National Cooperative Highway Research Program project to determine debonding limits and recommend auxiliary reinforcement details. For the time being, the AASHTO provisions have served designers well.

An example of an I-girder with all straight strands is one of the girders for the Cody Avenue Bridge in Okaloosa County, Fla. The details of this girder were provided by the precaster, Dura-Stress of Leesburg, Fla. The girder had a record length of 191 ft at the time it was produced. The 8-ft-deep Florida I-beam had sixty-nine 0.6-in.-diameter bottom strands, six 0.6-in.-diameter fully tensioned top strands, and four 3/8-in.-diameter partially tensioned top strands. The six large top strands were debonded in the middle 131 ft of the girder, and later cut to enhance the service load capacity. Of the 69 bottom strands, 18 were debonded at the ends to ensure that the concrete stresses at the time of detensioning were not exceeded.

Advocates of straight strand profiles emphasize the advantages of their production simplicity and reduced risk.

Advantages

Advocates of strand harping indicate the desire to preserve the prestressing level throughout the member length, and on having a strand profile that resembles that of the gravity load moment diagram. In addition, the amount of shear reinforcement is reduced by preserving the full prestressing force near member ends and by the vertical component of the prestressing force created through harping.

Advocates of straight strand profiles emphasize the advantages of their production simplicity and reduced risk. No harping hardware is required. Calculation of strand tensioning amount and sequence are relatively simple. Straight strand profiles allow for a smaller gap between girder ends during production. This may improve bed utilization and reduce strand waste in multi-girder beds.

Example Comparison

For a direct comparison, an example bridge girder design is illustrated assuming a 102-in. deep girder, a girder spacing of 9 ft, concrete strengths at prestress transfer and at service of 6.5 and 10.0 ksi, respectively, an 8-in.-thick, composite cast-in-place concrete deck with 4.5 ksi compressive strength, and the AASHTO *LRFD Specifications* apply without any modification. Given the results shown in the table, the differences in results between the two stress-control methods are insignificant.

Dr. Maher Tadros is principal with eConstruct, Omaha, Neb.

Comparison of harped versus straight strand design

Property		Harped Strand	Straight Strand
Straight full-length bonded bottom strands	No.	50	52
Draped strands	No.	18	0
Debonded bottom strands	No.	0	18
Total bottom strands at midspan	No.	68	70
Top strands	No.	6	6
Effective depth	d_v, in.	80	88
Required shear demand at 7 ft from support	V_u, kips	-556	556
Required nominal shear resistance	V_n, kips	618	618
Concrete shear capacity	V_c, kips	167	183
Vertical component of prestress	V_p, kips	49	0
Capacity to be resisted by stirrups	V_s, kips	402	435
Required stirrups	A_s	2#5@19.2 in.	2#5@19.2 in.

Table: eConstruct

Load Rating Concrete Bridges: Part 2

by Dr. Dennis R. Mertz

A single equation forms the basis of load rating using the load and resistance factor rating (LRFR) methodology of Section 6, Part A, of the American Association of State Highway and Transportation Officials' (AASHTO) *Manual for Bridge Evaluation* (MBE). All load rating factors (*RF*) for design, legal, or permit load levels and strength or service limit states are calculated using MBE Equation 6A.4.2.1-1:

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_{LL})(LL+IM)}$$

where

C = capacity

DC = dead load effect due to structural components and attachments

DW = dead load effect due to wearing surface and utilities

P = permanent loads other than dead loads

LL = live load effect

IM = dynamic load allowance

γ_{DC} = LRFD load factor for structural components and attachments

γ_{DW} = LRFD load factor for wearing surfaces and utilities

γ_P = LRFD load factor for permanent loads other than dead loads = 1.0

γ_{LL} = evaluation live load factor

All the loads except live load, *LL*, and dynamic load allowance, *IM*, are defined in Section 3 of the AASHTO *LRFD Bridge Design Specifications*. Live loads, *LL*, for design, legal, and permit load rating are defined in MBE Articles 6A.4.3.2.1, 6A.4.4.2.1, and 6A.4.5.4.1, respectively. Dynamic load allowance, *IM*, for design, legal, and permit load rating are defined in MBE Articles 6A.4.3.3, 6A.4.4.3, and 6A.4.5.5, respectively.

Locations of Load Factors in the MBE

Load Factor	Rating	MBE Table
γ_{DC}	All	6A.4.2.2-1*
γ_{DW}	All	2278
γ_{LL}	Design load	31,941
γ_{LL}	Legal load, commercial traffic	1710
γ_{LL}	Legal load, SHV**	1170
γ_{LL} Strength limit states	Permit load	1075
γ_{LL} Service limit states	Permit load	1166

*Also specified in the *LRFD Specifications*
**SHV = Specialized hauling vehicles

For the strength limit states, the capacity, *C*, is defined in MBE Equation 6A.4.2.1-2:

$$C = \varphi_c \varphi_s \varphi R_n$$

where

φ_c = condition factor

φ_s = system factor

φ = LRFD resistance factor

R_n = nominal member resistance (as inspected)

The optional condition factor, φ_c, and the optional system factor, φ_s, are specified in MBE Tables 6A.4.2.3-1 and 6A.4.2.4-1, respectively. The LRFD resistance factor, φ, and the nominal member resistance, R_n, for concrete members are as specified in Section 5 of the LRFD Specifications.

For the service limit states, the capacity, *C*, is defined in MBE Equation 6A.4.2.1-4:

$$C = f_R$$

where

f_R = stress limit specified in the LRFD Specifications

A future article will discuss the application of MBE Equation 6A.4.2.1-1 to the load rating of concrete bridges at the various load levels.

EDITOR'S NOTE

If you would like to have a specific provision of the AASHTO LRFD Bridge Design Specifications *explained in this series of articles, please contact us at www.aspirebridge.org.*

PRESTRESSED CONCRETE BRIDGES

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

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