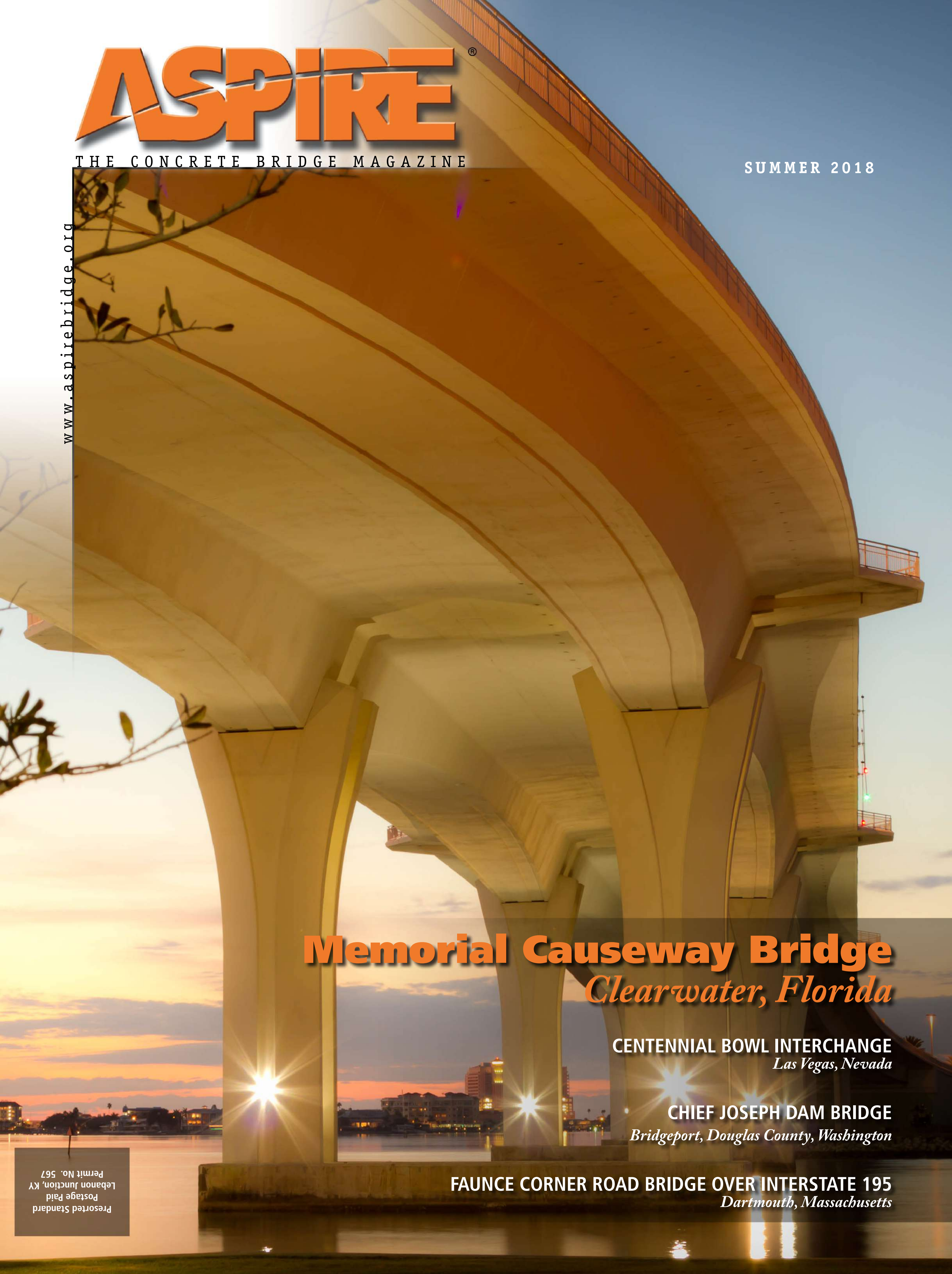

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THE CONCRETE BRIDGE MAGAZINE

SUMMER 2018

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Memorial Causeway Bridge
Clearwater, Florida

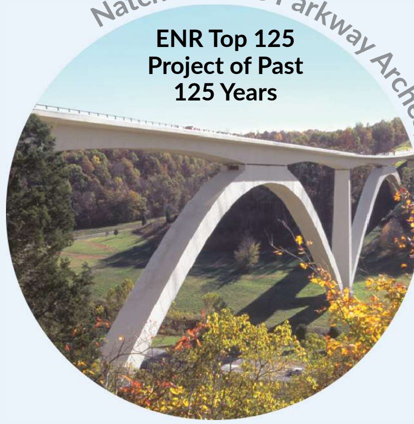
CENTENNIAL BOWL INTERCHANGE
Las Vegas, Nevada

CHIEF JOSEPH DAM BRIDGE
Bridgeport, Douglas County, Washington

FAUNCE CORNER ROAD BRIDGE OVER INTERSTATE 195
Dartmouth, Massachusetts

I-275 Bob Graham Sunshine Skyway Bridge, Florida

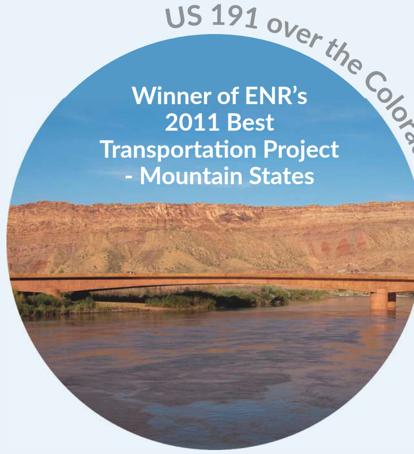
Natchez Trace Parkway Arches, Tennessee



Penobscot Narrows Bridge & Observatory, Maine

New I-35W Bridge, Minnesota

US 191 over the Colorado River, Utah



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EDITORIAL

Photo: PCI.

Book of Promises

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

Have you ever had a catchy song or clever slogan become instantaneously lodged in your memory? I hope that the phrase "Responsibility, Authority, and Accountability" from the editorial for the Winter 2017 issue of *ASPIRE*® has had that sort of effect on our concrete bridge readership. Sometimes, these types of taglines go viral immediately; other times, they take longer to spread. Keep sharing the message—it's not too late for this motto to take hold.

A couple months ago, an engineer let me know that certain points I had made about design-build projects more than 15 years ago are still being invoked today. This engineer was referring to a 2002 presentation I had given—while still employed by the Florida Department of Transportation (FDOT) as the state structures design engineer—and the subsequent panel discussion about new adjustments to design-build policies. Although the design-build construction procurement process was not new at that time, the industry was seeing an uptick in requested zero-dollar change orders that altered the scope of work. Some of these policy changes were a concern to the unsuccessful design-build teams as well as others involved in the process.

During my presentation and the panel discussion, we addressed what commitments are established by a contractor's proposal (what I then referenced as the "Book of Promises") and what contract terms can be changed after the project is awarded as the design is refined. When design-build scopes are assembled to respond to an emergency, they tend to be based on limited information due to the urgency. In this context, one would expect to have a few changes in conditions that both parties will need to address with supplemental agreements as reconstruction commences. In nonemergency scenarios, proposing teams may spend hundreds of days scoping and proposing the design-build project. Because the owner and proposing teams have taken the time to more fully understand the challenge ahead, one can expect fewer changes to the contract. Note that when this panel convened, the alternative technical concept (ATC) system had not yet been deployed. Now, the ATC process works very well to vet a change in scope or policies before the final project is even awarded.

Another topic of considerable debate at that time was whether warranties reduce the number of witnessed operations for the owner's inspection forces. Texas employees had adopted the slogan "You get what you inspect, not what you expect." My all-time "favorite" feedback on this topic was, "The marketer in the home office promised that I can walk on water, but now you and I have to be reasonable and build this job."

Some engineers, contractors, and suppliers seemed to believe that, if you look long enough, you will find a state highway agency employee who will authorize after-award changes in scope. However, that type of approach may put all involved in a position where he or she is unaware of the Book of Promises set forth in the proposal; some of these after-contract changes may even go against the advertised policies and standards for the project and blur authority and accountability.

Clay McGonagill, special counsel for FDOT, reminded everyone on the FDOT Design-Build Task Team that the author of a document is legally responsible for the words on the page. Today, that point may be self-evident for owners who write scopes and contracts every day or contractors who sign contract certifications for submittals. However, in 2002, the proposal was generally regarded as a routine attachment to the contract, and its legal weight as the Book of Promises was not fully recognized. McGonagill's assertion was reinforced with training about how owners can hold design-build teams to commitments in the proposal as the project progresses.

Many people who now use the saying "Book of Promises" in reference to a project proposal do not know its history, but the message is still there: A commitment made will need to be delivered. In a similar way, let's all work harder to make "Responsibility, Authority, and Accountability" a motto for every concrete and steel bridge project. This slogan reminds us to prevent any well-intentioned but ill-informed person (contractor, inspector, owner, or vendor) from changing something on a project in a way that will have detrimental long-term consequences that blindside future generations.

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Fred Gottemoeller served as aesthetic and urban-design consultant for the Memorial Causeway Bridge in Clearwater, Fla. Photo: Saegrande.

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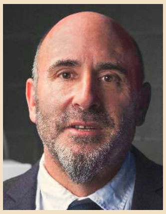

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Dr. Henry Russell is an engineering consultant who has been involved with the applications of concrete in bridges for over 35 years and has published many papers on the applications of high-performance concrete.

CONCRETE CALENDAR 2018–2019

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

July 23-27, 2018
PCA Professors Workshop
Skokie, Ill.

August 5-9, 2018
AASHTO Committee on Materials and Pavements Meeting
The Westin
Cincinnati, Ohio

August 14-15, 2018
ASBI 2018 Flexible Filler Certification Training
FAMU-FSU College of Engineering
Tallahassee, Fla.

August 22-24, 2018
PTI Level 1 & 2 Bonded PT Field Specialist Training and Certification
Chicago, Ill.

August 27–29, 2018
2018 NDE/NDT for Structural Materials Technology for Highway and Bridges Joint Conference with the International Symposium on NDT in Civil Engineering
Hyatt Regency New Brunswick
New Brunswick, N.J.

September 4, 2018
Vermont Agency of Transportation's Programmatic Implementation of ABC Webinar

September 16–19, 2018
AREMA 2018 Annual Conference & Exposition
Hilton Chicago
Chicago, Ill.

October 6–12, 2018
fib Congress 2018
Melbourne, Australia

October 10–13, 2018
PCI Committee Days and Membership Conference
Loews Chicago O'Hare Hotel
Rosemont, Ill.

October 14–18, 2018
ACI Fall 2018 Convention and Exposition
Rio All-Suites Hotel and Casino
Las Vegas, Nev.

October 22–25, 2018
PCA Design and Control of Concrete Mixtures: The Course
PCA Campus
Skokie, Ill.

November 6–7, 2018
ASBI 29th Annual Convention
Loews Chicago O'Hare Hotel
Rosemont, Ill.

January 13-17 2019
Transportation Research Board 98th Annual Meeting
Walter E. Washington Convention Center
Washington, D.C.

January 21–25, 2019
World of Concrete 2019
Las Vegas Convention Center
Las Vegas, Nev.

February 26–March 2, 2019
PCI Convention
Kentucky International Convention Center
Louisville, Ky.

CALL FOR PAPERS

"Corrosion of Metallic Elements in Earth Retaining Structures and Marine Port Infrastructure"

Presiding Officers: Dave Meggers, Soundar Balakumaran, Stacey Kulesza, Brian Pailes
Sponsoring Committee: AHD45

The TRB Corrosion Committee (AHD45) is soliciting papers regarding the corrosion of metallic elements in earth retaining structures and marine terminals and port infrastructure. The mechanically stabilized earth structures have many metallic elements including soil nails, anchors/tiebacks and sheet piling, which are subject to corrosion. The second portion of this call for papers involves the influence of corrosive marine environment on critical elements of port facilities. The elements include steel bulkhead walls, reinforced concrete wharfs and both steel and concrete piles. Maintenance and upkeep are critical to the transportation system and the mitigation of corrosion damage and control is critical to the life cycle of the system. The goal of the session/s is to understand the corrosion process and learn new approaches to control and mitigate corrosion damage in these environments. Papers are due to the Transportation Research Board by August 1, 2018 to be included in the January 2019 Annual Meeting program.

FOCUS

Bridgescape: A Leader in Aesthetic Engineering

Bridgescape LLC owner Fred Gottemoeller stresses attractive designs without sacrificing function, integrity, or budget—and he encourages engineers to unleash their creative side.

by Craig A. Shutt

Gottemoeller worked to refine the aesthetics of the St. Croix River Crossing in Stillwater, Minn., which connects to Wisconsin. The bridge features an extradosed design with stayed, post-tensioned concrete box girders. Photo: Minnesota Department of Transportation.

Fred Gottemoeller's one-person aesthetic consulting firm, Bridgescape, stresses the impact that aesthetic design can have on a bridge's reception and success. The author of a 2004 book for engineers, *Bridgescape: The Art of Designing Bridges*,[1] he rejects the notion that an aesthetic design must cost more and encourages engineers to make aesthetics a priority in every project.

"Engineering education gives no guidance on aesthetic elements," Gottemoeller says. "It focuses on creating functional structures, with the appearance resulting strictly from engineering design parameters. We need to encourage engineers to express their aesthetic ideas during design. They often don't feel qualified to do so, but they are. Some of the best aesthetic ideas I see come from engineers."

'Some of the best aesthetic ideas I see come from engineers.'

Five Fundamental Ideas

Gottemoeller's book outlines five fundamental ideas that often are overlooked or disputed but stand as core truths. Fifteen years after the book was published, he believes those elements remain the same. "They're common concepts that trace back to the Greeks," he states. "That's one reason I haven't felt it necessary to update the book."

All bridges make an impact.

"The bridge will make an impression: of excitement, appreciation, repulsion, or perhaps boredom," Gottemoeller wrote. This holds true whether or not the engineer intentionally plans that impact.

People can agree on what is beautiful for bridges.

Beauty may be in the eye of the beholder, but, Gottemoeller argued, that does not mean people can't agree on what is attractive. Key elements for bridges include simplicity of elements, thinness, continuous lines, and shapes that reflect the magnitude of forces (that is, the thickest shapes indicate where forces are greatest).

Engineers must take responsibility for the aesthetic impact of their bridges.

"Engineers are used to dealing with issues of performance, efficiency, and cost, but they must also be prepared to deal with the issues of appearance," Gottemoeller wrote. They can't avoid these issues by focusing on structural elements and leaving aesthetics to others, he stressed. "The appearance is dominated by the shapes and sizes of the structural elements themselves, not by details, colors, or surfaces."

Engineers should consider good appearance to be co-equal with strength, safety, and cost.

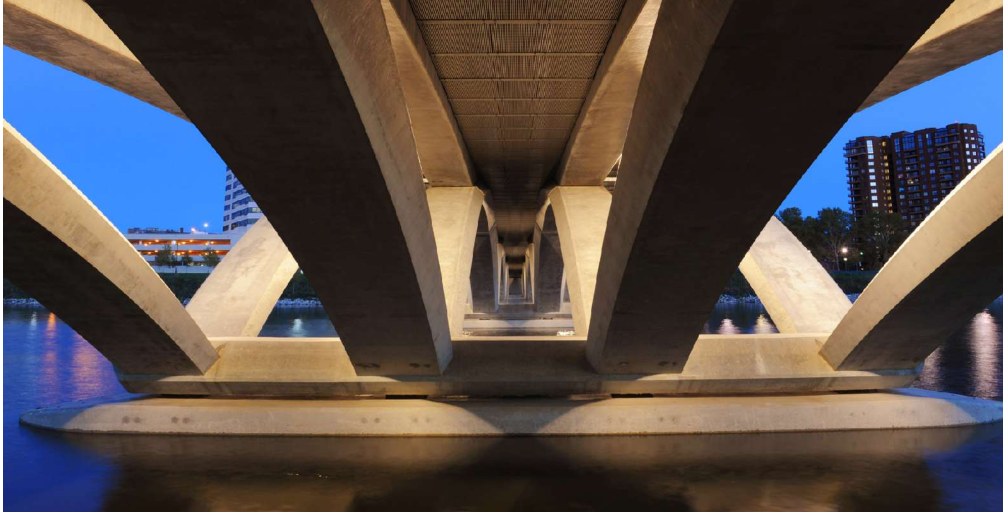
Some engineers believe achieving compelling aesthetics automatically compromises other core requirements or adds cost because aesthetic designs add features such as color or special finish materials. "In fact," Gottemoeller wrote, "The greatest aesthetic impact is made by the structural members themselves. If they are attractive, then the bridge will be attractive." Details, colors, and surfaces add aesthetic interest, but they may not always add sufficient aesthetic impact to justify the additional cost.

Aesthetic ability is a skill that can be acquired and developed by engineers, as well as anyone else.

"Engineers can learn what makes bridges attractive, and engineers can develop their abilities to make their own bridges attractive," Gottemoeller stated.

Aesthetics in Practice

Gottemoeller puts these five concepts into practice in his work, having served as an aesthetic design consultant for a variety of signature bridges. One of his most recent projects, on which he refined the aesthetic concept during the final design, is the St. Croix River Crossing in Stillwater, Minn., which opened in August 2017. It spans the St. Croix River, which is designated as a National Wild and Scenic River.



The five-span Rich Street Bridge in Columbus, Ohio, is supported on four lines of post-tensioned precast concrete arches. Photo: Randall Scheiber.

The extradosed bridge features stayed girders consisting of 18-ft-deep post-tensioned concrete segmental box girders with curved sides (see article on extradosed bridges in the Summer 2015 issue of *ASPIRE*®). The pier shafts have a split design tied together at the top and bottom. The split creates flexibility that allows the bridge to react to temperature variations and makes the piers nearly transparent. The structure was built with high-performance concrete and high-strength reinforcement. "It [the extradosed bridge concept] is a huge improvement on other concepts," Gottemoeller says. "I expect there will be more use of this option in the 450- to 600-ft range. It's a very attractive and exciting concept, and it's been very well received locally."

Concrete's Aesthetics

Gottemoeller remains "agnostic" about materials for bridges. He advocates for using whatever materials can accomplish the structure's goals with the best combination of features. But he sees advantages to working with concrete, especially from an aesthetic viewpoint. "Concrete bridges don't need to be painted, and they can retain their light, reflective coloring for long periods," he points out.

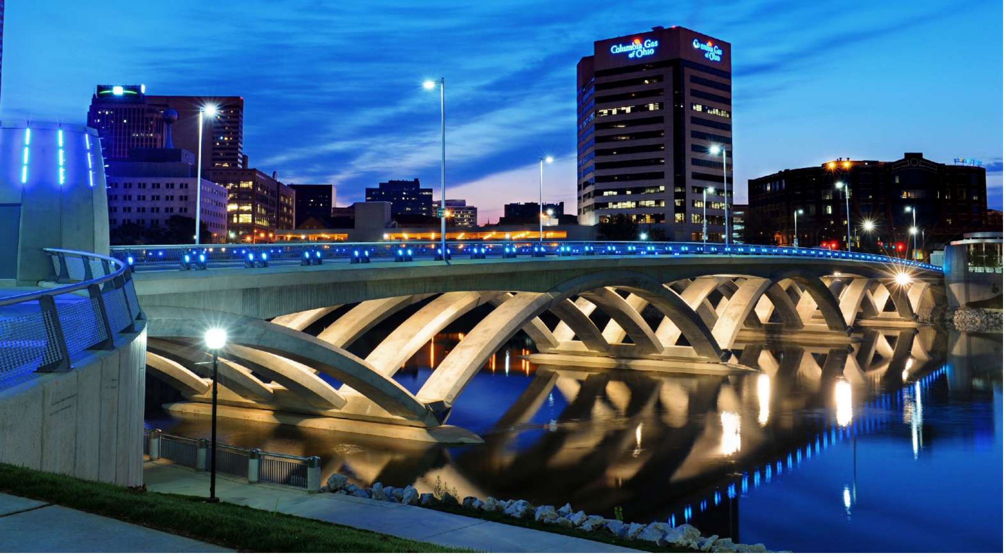
A key, but often overlooked, benefit of concrete derives from its inherent mass, which can dampen traffic sounds, especially beneath the bridge. "People who live near concrete bridges have a better opinion of them because their living experience is better," he says. "The material can affect sound levels, which is an underappreciated sensory element of aesthetics."

'The material can affect sound levels, which is an underappreciated sensory element of aesthetics.'

Gottemoeller always keeps an eye on concrete technologies to consider how changes can affect his designs. "Materials such as high-performance concrete and self-consolidating concrete are exciting to consider when the situation is right," he notes. Stayed girders are the most recent concrete concept that he has incorporated into his designs.

Specialized concrete mixtures were a key ingredient in the design of the Rich Street Bridge over the Scioto River in Columbus, Ohio, which was completed in 2013 and featured in a project article in the Fall 2012 issue of *ASPIRE*. The bridge's appearance was conceived as a visual unifier for two nearby bridges, combining the span arrangement of the Discovery Bridge with the open appearance of the Main Street Bridge,

Gottemoeller developed the overall concept and the architectural details for the Rich Street Bridge over the Scioto River in Columbus, Ohio. Photo: Randall Scheiber.



The lighting design of the Rich Street Bridge over the Scioto River in Columbus, Ohio, was developed by Gottemoeller. Photo: Randall Scheiber.

a tilted through-arch design completed in the style of Spanish architect-engineer Santiago Calatrava.

'Materials such as high-performance concrete and self-consolidating concrete are exciting to consider when the situation is right.'

The five-span, 568-ft-long Rich Street Bridge is supported on four lines of custom arches, with both arches and girders consisting of precast concrete segments using high-strength, lightweight concrete. The arches consist of three precast concrete segments: two identical arch legs and a keystone segment. A fourth standardized beam segment spans over the piers between the arch crowns. The segments were connected with short closure pours and then post-tensioned to function as a single five-span unit. The bridge came in below the city's budget for the project and at approximately one-third the cost of the Main Street Bridge.

Gottemoeller developed the overall design as well as the architectural details and the lighting concept for the Rich Street Bridge. Since opening, it has won multiple awards, including recognition from the American Council of Engineering Companies, the American Society of Highway Engineers, and an award from the Columbus Landmarks Foundation.

Cost Effectiveness Is Key

Gottemoeller disdains high-profile architects whose designs demonstrate little regard for budget. "They get photos of their 'works of art' in magazines, but the structures are not cost-effective or engineered well," he states. "The only thing they accomplish is to persuade agencies and engineers that aesthetics must be expensive." To the contrary, Gottemoeller emphasizes that "good aesthetics don't have to break the bank." Some departments of transportation, notably Minnesota and Colorado, understand this point and make budget and aesthetics equal priorities.

"A tight budget can and should act as a spur to creativity, encouraging a search for new approaches with both cost and aesthetic advantages," Gottemoeller wrote in his book. "Improvements in appearance should be sought just like improvements in any other area of concern: safety, durability, or maintainability."

'Good aesthetics don't have to break the bank.'

Leading by Example

Gottemoeller's role as consulting aesthetic engineer on projects gives him an opportunity to disseminate his concepts. For example, in 2004, he served as aesthetic and urban-design consultant for the Memorial Causeway Bridge over Clearwater Bay in Clearwater, Fla. The bridge, which replaced a design with a bascule span, features cast-in-place concrete segmental, haunched, trapezoidal box girders. The girders have tapered sides that minimize the width of the tapered piers, allowing the girders and piers to flow together in a single sweep. The design provides 330-ft spans that avoid obstructing scenic views. White concrete was used to reflect the water color and evening sunsets. The vertical spaces within the four split-center piers are illuminated to create a memorable nighttime appearance.

A more recent project reflecting Gottemoeller's commitment to aesthetics is the Virginia Street Bridge over the Truckee River in Reno, Nev., which was built to allow more clearance for flood waters. Completed in 2016 near historic buildings, the single-span bridge features a pair of cast-in-place, post-tensioned concrete through arches, paying homage to the two-span arch design of the original structure. The tie beams of the thin floor system are connected to the low arches with steel cables to create a low profile, which is enhanced by aesthetic lighting at night.

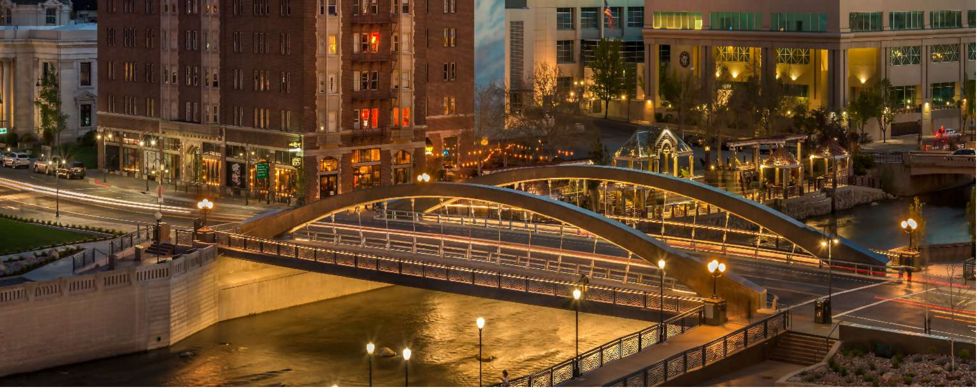
Gottemoeller worked with the engineering team and the historical review committee to create the architectural and structural concept for the Virginia Street Bridge and developed the architectural details. "Structurally, the bridge is a hybrid of a tied through-arch and a rigid frame," he explains.

Design-Build Aids Aesthetics

Creating aesthetically pleasing designs becomes easier as more bridges are designed aesthetically in cost-effective ways, Gottemoeller says. The openness of owners to considering new delivery formats advances that process. "The expanded role of design-build formats is changing how bridges are designed," he says. "The relationships between owner, engineer, and contractor change and can make it easier to suggest and incorporate aesthetic concepts. It creates more conversation among the parties and brings more ideas to the table."

Construction manager-at-risk formats especially help drive new ideas, he adds. "That delivery method produces an ideal setup, since ideas suggested early in the design phase can be quickly costed out by the contractor. Aesthetic concepts shouldn't be added after the fact to dress up a design."

Owners' attitudes are critical to



The single-span bridge on Virginia Street in Reno, Nev., consists of a pair of cast-in-place, post-tensioned concrete through-arches that support a thin floor system, allowing high floodwaters to pass while minimizing the vertical profile and impact on adjoining historic buildings. Photo: Vance Fox.

achieving high-quality aesthetics, Gottemoeller notes. If owners set aesthetics as a goal, the team will get behind it. The emphasis on short timetables does not necessarily hinder the process. "Doing everything quickly limits the ability to consider all of the options." But that does not give quickly constructed projects a free pass, he stresses. "Speed and cost can be used as excuses, but they aren't good reasons to not consider appearance. If aesthetics is a priority, an attractive bridge can be created with any budget or timetable."

Stakeholder Input Grows

Increased sensitivity to stakeholder input, especially from the community, can add challenges to bridge design and construction, but owners today are more willing to take this on. "They're not as scared to solicit opinions, and that can help create acceptance for a design," Gottemoeller notes. Conflicts among various ideas often can be overcome with clear communication, he says.

"Many of those involved [in discussions of a bridge project] aren't engineers, so it's important to communicate and educate, and it's also important to listen to community concerns and adapt where possible," he says. "Everyone pays taxes, so they realize some requests are going to be too expensive to accomplish."

Aesthetics will continue to be at the forefront of Gottemoeller's designs, and he encourages owners to solicit ideas and prioritize this element. "Owners should encourage engineers to express their opinions and offer suggestions." The split-pier concept for the St. Croix bridge was suggested by the engineers, he notes. "I just got on their bandwagon."

Encouraging engineers to give input will help them realize their ideas have merit. "Every time we ask, we get good ideas from them," he says. "I can never predict what will come up, but I know something good will arise if we encourage them and listen to their ideas."

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1. Gottemoeller, Fred. 2004. *Bridgescape: The Art of Designing Bridges.* Hoboken, NJ: Wiley.
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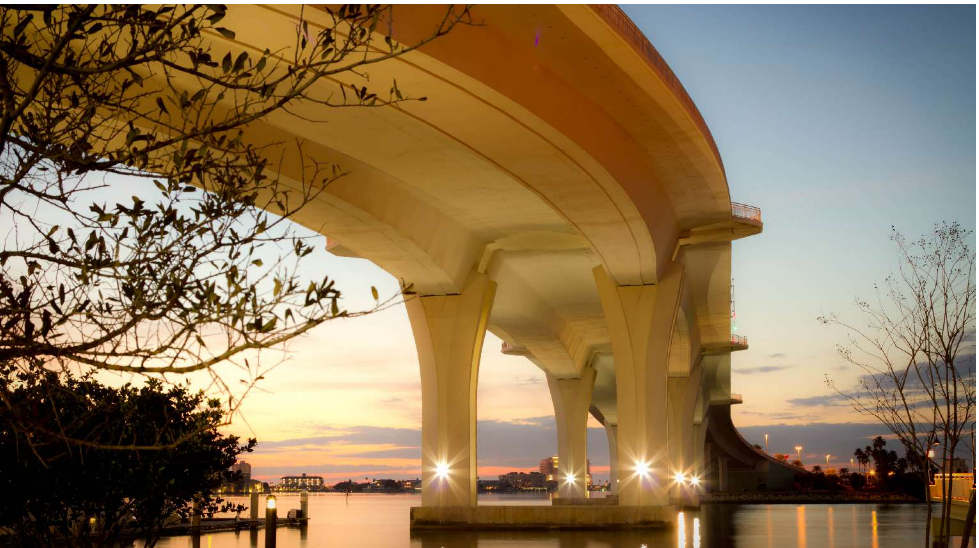
Bridgescape's Driving Force

Fred Gottemoeller is both an engineer and architect. He began his career with Skidmore, Owings & Merrill in 1967 and later worked in management positions with the Maryland Department of Transportation and Maryland Highway Administration as well as A. J. Properties, where he served as director of design and construction. In 1988, he created Frederick Gottemoeller & Associates, which became Rosales Gottemoeller & Associates in 1998. He formed Bridgescape LLC, his one-person firm, in 2005.

"My goal in starting Bridgescape was to focus on providing advice on aesthetics and community participation to engineers," he says. "The engineering side of my background helped me understand what engineers were trying to accomplish and the language to communicate with them in their 'native tongue,' so to speak. Much of what I do is translating engineering words into architectural and community terms and vice versa."

Gottemoeller has helped develop bridge aesthetic guidelines in Maryland and Ohio. He also has contributed to the *Bridge Aesthetics Sourcebook,*[2] the aesthetics chapter of the *Bridge Engineering Handbook,*[3] and a bridge aesthetics handbook for the Society of Engineers of India.

He served as chair of the Transportation Research Board Bridge Aesthetics Subcommittee for many years and has presented seminars on bridge aesthetics under its auspices. He regularly provides aesthetic commentary on bridge designs for projects featured in *ASPIRE*.



The Memorial Causeway Bridge in Clearwater, Fla., features two cast-in-place concrete segmental, haunched, trapezoidal box girders with sloped sides. Gottemoeller was aesthetic and urban-design consultant and developed the architectural details. Photo: Saegrande.

Overview of Delayed Ettringite Formation and Alkali-Silica Reaction

by Dr. David Rothstein, DRP, a Twining Company

Delayed ettringite formation (DEF) and alkali-silica reaction (ASR) are complex mechanisms that can significantly diminish the durability and service life of concrete elements. While reduced durability and service life are legitimate concerns, it is also important to recognize that DEF is both relatively rare and commonly misdiagnosed. Likewise, the mere presence of ASR in concrete does not necessarily indicate the end of the useful service life of an element. The purpose of this perspective is to provide an overview on how ASR and DEF are recognized in concrete and show that repairs may be available for arresting or slowing their progression. Articles in future issues of *ASPIRE*® will discuss how these mechanisms operate and steps that can be taken beforehand to mitigate ASR and DEF. There is extensive research on ASR and DEF that can provide additional information.[1-4]

What Are DEF and ASR?

DEF and ASR are chemical reactions that produce secondary deposits within concrete after it hardens and is put into service. In the case of DEF, components in the cement paste react with water to form secondary deposits that consist of the mineral ettringite, which has the chemical formula $Ca_6Al_2(SO_4)_3(OH)_{12}\bullet 26H_2O$. In the case of ASR, reactions occur between aggregate particles and the paste to produce secondary deposits that consist of a gel of indefinite composition that may be expressed as (Na, K, Ca) $SiO_3 \bullet x\, H_2O$.

Note that both ettringite and ASR gel contain water (H_2O). As such, the infiltration of water into concrete lies at the root of both of these deterioration mechanisms (Fig. 1). The deposits formed by DEF and ASR have a greater volume than the solid phases in the concrete, which results in an internal expansion that causes cracking once the tensile strength of the concrete is exceeded. Minimizing permeability and cracking and keeping the internal relative humidty of the concrete below 70% while it is in service are therefore keys to minimizing deterioration from ASR and DEF.

Diagnosing DEF and ASR

A concrete petrographer can evaluate whether concrete is affected by DEF or ASR by using various microscopes to examine the internal microstructure of concrete and other cement-based construction materials. Petrographers can document whether there is evidence of internal expansion in the concrete and whether that expansion can be linked to the presence of secondary deposits. Because extensive microcracking is commonly observed in concrete affected by either ASR or DEF, the petrographer must then determine if the secondary deposits associated with the microcracks are ettringite or ASR gel.

Figure 2 shows an example of microcracking in concrete caused by DEF. In many cases, concrete affected by DEF has networks of fine microcracks filled with a material that—even under powerful optical microscopes—appears to be a gel but is actually ettringite. Petrographers may observe microcracks filled with such deposits and assume that ASR is present, unless they use a scanning electron microscope (SEM) to distinguish between ASR gel and ettringite. Most SEMs are equipped with an instrument known as an energy-dispersive x-ray spectrometer (EDS),

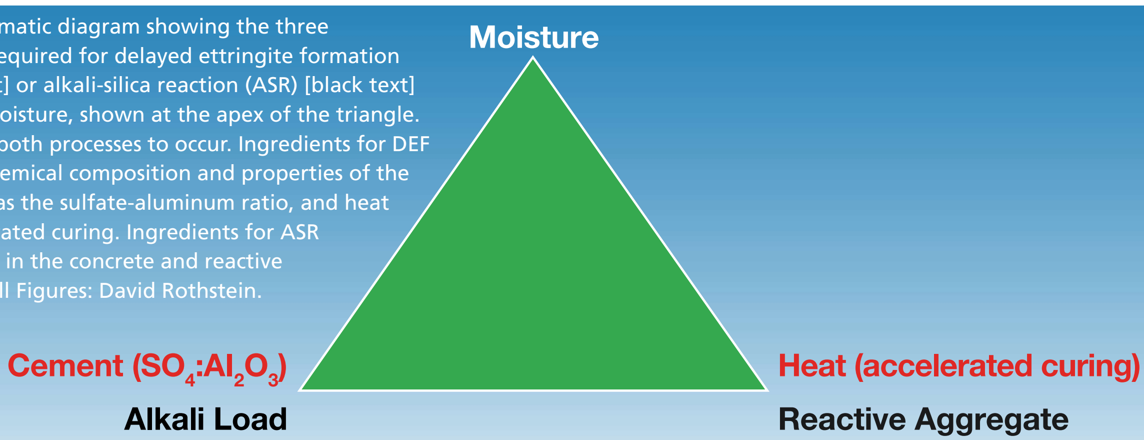


Figure 1. Schematic diagram showing the three components required for delayed ettringite formation (DEF) [red text] or alkali-silica reaction (ASR) [black text] to proceed. Moisture, shown at the apex of the triangle. is,needed for both processes to occur. Ingredients for DEF include the chemical composition and properties of the cement, such as the sulfate-aluminum ratio, and heat during accelerated curing. Ingredients for ASR include alkalis in the concrete and reactive aggregates. All Figures: David Rothstein.

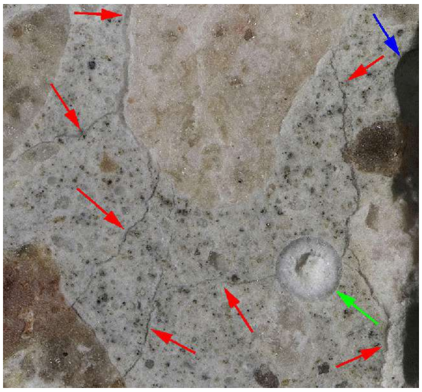


Figure 2. Reflected light photomicrograph of the polished surface of a core from a concrete structure with delayed ettringite formation. Arrows indicate microcracks that are filled with a clear, gel-like material (red), a void filled with ettringite (green), and the outer surface of the core (blue).

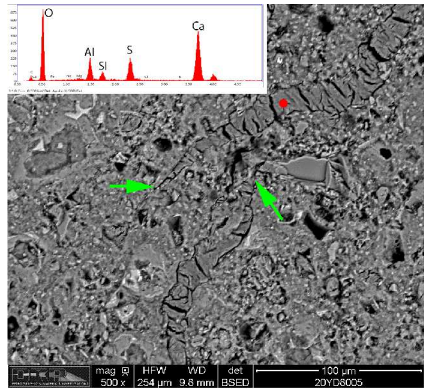


Figure 3. Backscatter electron micrograph at higher magnification showing microcracks (green arrows) from part of the area shown in Fig. 2. The red dot shows the area of the sample that produced the energy-dispersive x-ray spectrometer (EDS) spectrum shown in the upper left. The spectrum indicates the presence of calcium, aluminum, and sulfur, which is consistent with ettringite.

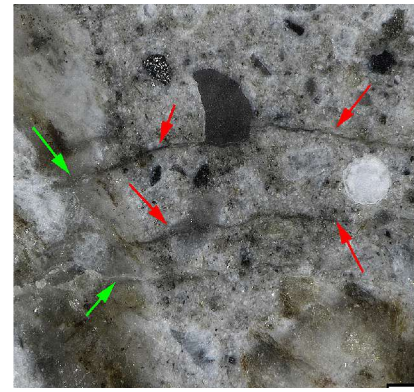


Figure 4. Reflected light photomicrograph of the polished surface of a concrete core with alkali-silica reaction (ASR). Arrows show microcracks with ASR gel within a granitic (and reactive) aggregate particle (green) and microcracks with gel that cut into the paste (red).

which allows determination of the chemical composition of materials that are only microns across. If elements such as calcium, aluminum and sulfur are detected by the EDS, a petrographer knows that the deposit is ettringite (Fig. 3). If the gel deposit contains elements such as sodium, potassium, and silicon, that indicates ASR gel.

Petrographers commonly see deposits of crystalline ettringite in voids, microcracks, and cracks in field concrete that is exposed to moisture. As such, the presence of ettringite alone is not an indicator of DEF or any other form of sulfate attack; it simply indicates that the concrete was exposed to moisture. An experienced petrographer, ideally working with an experienced engineer, is needed to determine whether the concrete is affected by DEF.

Extremely fine-grained, isolated deposits of ettringite in the paste are another feature of DEF. These "nests" of ettringite, which require magnification of about 2000 times to detect, can be identified with a SEM.

Concrete affected by ASR commonly has deposits of both ASR gel and ettringite. However, the patterns of microcracking caused by ASR are often much different from those caused by DEF. With ASR, microcracks can often be traced from the inner portions of reactive aggregate particles into the paste (Fig. 4). In DEF, the microcracks do not trace to the inner portions of aggregate particles and appear randomly throughout the paste.

Petrographers may use criteria such as whether cracks or microcracks can be linked to DEF or ASR to evaluate the severity of damage. Although not standardized, some methods are available that use petrography to quantify damage from ASR or DEF.[5] When deployed consistently, these methods can be used—along with field-based approaches such as monitoring the widths of cracks—to track the progression of damage over time.

Implications for In-Service Concrete

Although ASR occurs much more commonly than DEF, it is essential to understand that the presence of ASR gel in concrete does not mean that the concrete is no longer able to serve its intended purpose. ASR is commonly observed in concrete after several years of service, if the concrete is exposed to moisture, with no resulting cracking or even microcracking. A thorough investigation undertaken by experts in both engineering and petrography can determine whether ASR is causing cracking or microcracking that is structurally significant.

Because both DEF and ASR require moisture to progress, reducing the ingress of water is central to minimizing damage from these mechanisms. In many cases, experienced engineers working in tandem with specialty contractors can determine remediation measures that can greatly prolong the serviceability of an affected concrete element.

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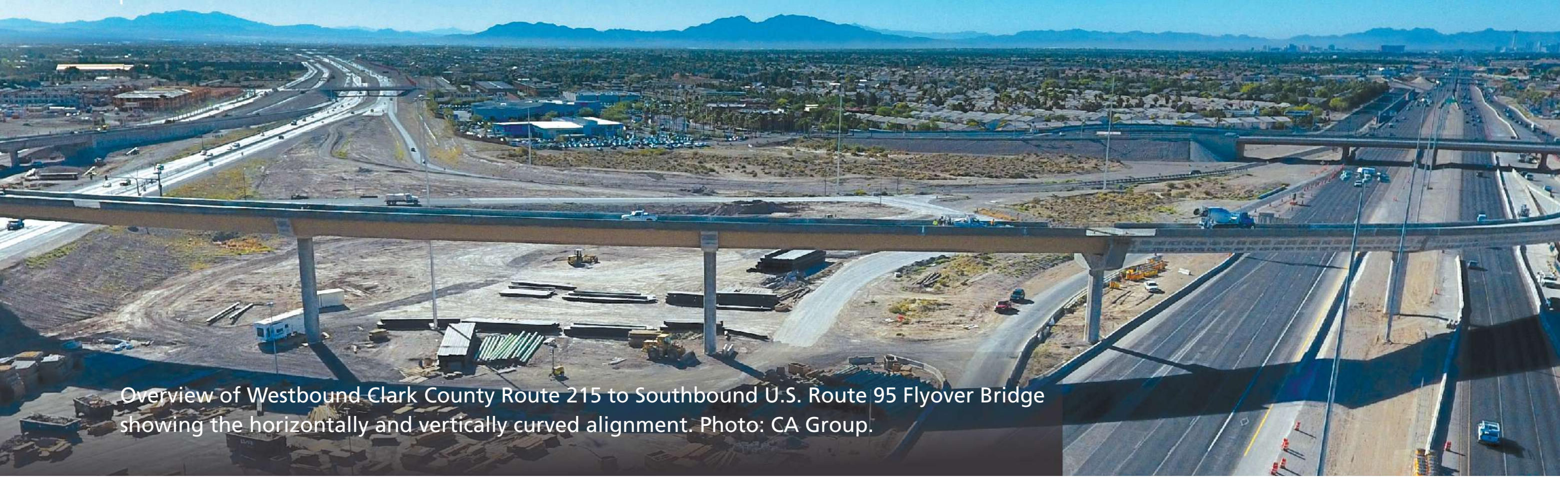
PROJECT

Centennial Bowl Interchange

Westbound Clark County Route 215 to Southbound U.S. Route 95 Flyover Bridge

by Michael Taylor, Nevada Department of Transportation



Overview of Westbound Clark County Route 215 to Southbound U.S. Route 95 Flyover Bridge showing the horizontally and vertically curved alignment. Photo: CA Group.

The U.S. Route 95 (U.S. 95) northwest corridor in Las Vegas is one of the fastest-growing areas in southern Nevada, and the U.S. 95/Clark County Route 215 (CC 215) Interchange serves as a critical link for regional commuters between the predominately residential areas in northwest Las Vegas and the large employment centers in the area. To meet the growth in the region, the Nevada Department of Transportation (NDOT) is currently upgrading the U.S. 95/CC 215 Centennial Bowl Interchange, the third busiest intersection in the state, from an at-grade intersection to a full system-to-system interchange.

In 2008, NDOT began design on the interchange, which was initially estimated to cost around $250 million. To work within funding constraints, the project was divided into three projects. In July 2017, the first phase of the interchange—a 2365-ft-long post-tensioned cast-in-place (CIP) concrete box girder carrying westbound CC 215 to southbound U.S. 95 (WS Flyover)—was completed.

Type Selection

Type selection and preliminary layout for the Centennial Bowl Interchange structures began in 2010. NDOT considered four different superstructure types for the interchange's 15 grade separations: CIP post-tensioned box girders; steel plate girders; precast concrete I-girders; and precast concrete U-girders. Structure types were evaluated based on cost, aesthetics, constructability, and durability. For the three large flyover structures, the selected structure type needed to accommodate spans longer than 200 ft, horizontally and vertically curved alignments (horizontal radii less than 850 ft), and superelevations of 6%. Historically, the CIP post-tensioned box girder is the most cost-efficient bridge type in Nevada, with typical costs under $150 per square foot of deck. At the time of type selection, steel plate girders and precast concrete girders cost $200 per square foot and $220 per square foot, respectively.

The geometric constraints of the interchange favored the CIP and plate girder solutions, with CIP ultimately selected based on cost, aesthetics, and durability. The use of CIP post-tensioned box girders for the interchange will result in an estimated 14% total project cost savings, lowering the cost for total structures by $30 million compared with steel plate girders.

Design

The WS Flyover is a fourth-level ramp, crossing over the major U.S. 95 and CC 215 roadways, six future interchange ramp movements, one existing local road, and a 36-in.-diameter high-pressure gas line. Due to these horizontal constraints, the layout of the flyover uses three frames with lengths of 858 ft, 862 ft, and 645 ft, for a total length of 2365 ft. The WS Flyover's alignment crosses the southbound lanes of U.S. 95 at a sharp skew, necessitating

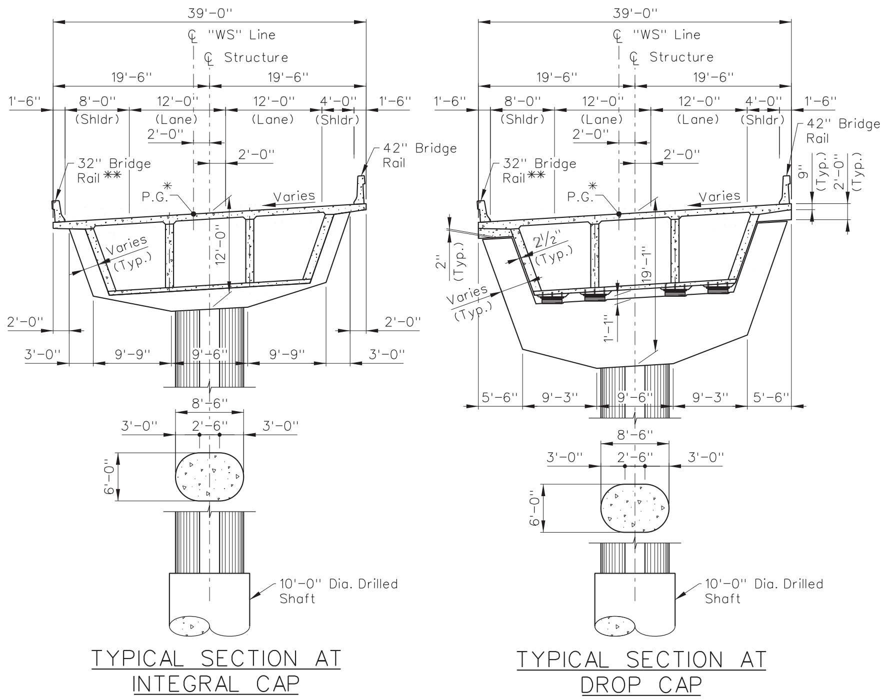
profile

CENTENNIAL BOWL INTERCHANGE (WESTBOUND CLARK COUNTY ROUTE 215 TO SOUTHBOUND U.S. ROUTE 95 FLYOVER BRIDGE) / LAS VEGAS, NEVADA

BRIDGE DESIGN ENGINEER: Nevada Department of Transportation, Carson City, Nev.

PRIME CONTRACTOR: Las Vegas Paving Corporation, Las Vegas, Nev.

POST-TENSIONING CONTRACTOR: DYWIDAG Systems International, Long Beach, Calif.



Typical cross sections of three-cell concrete box girder and oblong column at integral and expansion piers. Figure: Nevada Department of Transportation.

a maximum span of 250 ft. The remaining 10 spans were configured to balance span lengths to maximize the efficiency of the superstructure cross section while working around the horizontal constraints imposed by the ramps and utilities.

Early in the design phase of the WS Flyover, NDOT performed Osterberg load testing on the large-diameter drilled shaft foundations. NDOT invested $600,000 in the load-testing program and realized a savings of $2.6 million in foundation costs for the WS Flyover by using the higher American Association of State Highway and Transportation Officials (AASHTO) resistance factors and higher soil capacities. Additional savings will be realized in future foundation construction phases.

The superstructure of the WS Flyover consists of a 39-ft-wide, three-cell CIP box girder, with 4-ft-wide overhangs and a 10 ft web spacing. The box-girder depth of 9.5 ft is dictated by a maximum bridge span of 250 ft, resulting in a span-to-depth ratio of 26.3.

The substructure consists of oblong-shaped single-column bents (6 ft by 8.5 ft) with integral pier caps. Per standard NDOT practice, drop-cap (or stepped-cap) expansion piers were used between frames instead of in-span hinges.

Durability considerations for the concrete included the use of 6.5-ksi high-performance concrete in the bridge deck. The project specifications mandated the use of a shrinkage-reducing admixture to limit deck shrinkage as well as the use of a three-aggregate blend in the deck concrete to reduce permeability.

The superstructure design resulted in a total post-tensioning force of 18,984 kips, achieved using twenty-seven 0.6-in.-diameter strands per tendon and four tendons per web for each of the four webs of the three-cell box. The tight 847-ft horizontal radius warranted additional design considerations. The girders were designed to resist lateral web bending due to post-tensioning and live loading, as well as global shear and torsion. Tendon confinement in the webs is provided by tie reinforcement proportioned to resist local web-bending effects, tendon deviation forces, and internal splitting forces due to the draped vertical tendon profiles.

All pier caps in the WS Flyover use post-tensioning to limit congestion of mild reinforcement, improve seismic joint performance, and increase the

Completed Westbound Clark County Route 215 to Southbound U.S. Route 95 Flyover Bridge. Photo: Nevada Department of Transportation.

NEVADA DEPARTMENT OF TRANSPORTATION, OWNER

BRIDGE DESCRIPTION: 2365-ft curved, cast-in-place post-tensioned concrete box girder on an 847-ft minimum horizontal radius (11 spans with a 250-ft maximum span)

STRUCTURAL COMPONENTS: 39-ft-wide by 9.5-ft-deep cast-in-place post-tensioned three-cell concrete box girder superstructure with 6.5-ksi, high-performance concrete deck; 10-ft-diameter drilled shaft foundations; 6-ft-by-8.5-ft oblong columns; post-tensioned integral and expansion pier caps

BRIDGE CONSTRUCTION COST: $10.1 million ($110/ft^2)

AWARD: Associated General Contractors/Nevada Contractors Association Nevada Civil Project of the Year, 2017 durability of the structure. The 3340-kip transverse post-tensioning force in each of the expansion pier caps is provided by nineteen 0.6-in.-diameter strands in each of the four tendons. Shear keys at the expansion caps restrain lateral movement of the superstructure and force composite behavior in the transverse direction between adjacent frames. These shear keys are designed to remain elastic under seismic loading, which is accomplished by permitting the supporting columns to fuse, and designing the shear keys for the associated column plastic hinging forces.

Falsework in place for frame 2. The maximum falsework height was 58 ft. Photo: Nevada Department of Transportation.

The transverse post-tensioning is a primary component of the seismic detailing, ensuring that the expansion pier caps have adequate capacity to resist lateral forces transferred through the shear keys. The 1932-kip post-tensioning in the integral pier caps is provided by 11 tendons with four 0.6-in.-diameter strands each running through the deck slab. The use of transverse post-tensioning in the integral caps was done primarily to promote durability of the bridge deck by limiting tensile stresses, but it has the added benefit of improving the joint shear performance by precompressing the joint region.

Falsework of frame 2 being lowered after completion. Photo: Nevada Department of Transportation.

Concrete box girder with reinforcement and post-tensioning ducts in place for soffit and webs. Photo: Nevada Department of Transportation.

Frame 3 tunnel bent over U.S. Route 95 with bracing. Photo: Nevada Department of Transportation.

Due to the size and complexity of the WS Flyover, seismic design was a key aspect of the design process. The flyover is the first bridge in Nevada to use the *AASHTO Guide Specifications for LRFD Seismic Bridge Design,* 2nd edition, with 2012 Interims. In addition to these specifications, NDOT investigated the use of performance-based seismic design by evaluating the structure's performance at 500- and 1000-year-event seismic hazard levels. The performance criteria used to evaluate the columns are based on an essentially elastic response at the 500-year event, with light damage permitted at the 1000-year event. The 1000-year design response spectrum has the following site-adjusted spectral accelerations: peak ground acceleration of 0.235*g*, S_s = 0.59*g*, and S_1 = 0.33*g* (Site Class D). These values place the structure in Seismic Design Category C. The long frame length and variable column height due to the curved vertical profile result in a significant difference in stiffnesses between frames and adjacent columns. The use of isolation casings to lengthen three of the shorter columns provides a more uniform seismic response.

Construction

NDOT delivered the first phase of the Centennial Bowl Interchange via the design-bid-build delivery method in October 2015. The construction of the WS Flyover was completed in 375 working days, for a total cost of $110 per square foot.

Construction of long-frame CIP post-tensioned box girders in an urban setting poses several challenges. Due to the use of falsework, maintenance of traffic becomes an issue for construction. Careful planning during design, and coordination in construction, allowed the contractor to successfully manage traffic. For U.S. 95 traffic, adjacent collector/distributor roads served as detours for the erection and removal of falsework and during concrete placement. For daytime traffic, a minimum of three lanes in each direction was maintained. Due to the sharp skews of the WS Flyover alignment and U.S. 95, the contractor used a tunnel bent with a tiered-falsework system to maintain the minimum number of lanes.

Grouting of the longitudinal post-tensioning ducts proved to be another construction challenge, due to the vertical geometry, frame length, and the hot climate of southern Nevada. NDOT requires the use of prebagged thixotropic grout. Grouting pressures regularly exceeded 100 psi, with typical grouting durations of more than one hour for the long frames. The contractor used chilled water and took precautions to protect the grout bags from sun exposure. The grout material proved to be forgiving with respect to grouting times and temperature, maintaining its fluidity despite the long grouting durations and ambient temperatures greater than 100°F.

Conclusion

The WS Flyover, which opened to traffic in July 2017, demonstrates how CIP post-tensioned concrete bridges can provide an economical and resilient structure type for modern urban interchanges in Nevada. The CIP, post-tensioned concrete box girder structure type is well suited for the curved geometrics while maintaining a strong aesthetic appeal. Careful planning, coordination, and partnering through the construction phase successfully mitigated the typical risks associated with CIP construction. Use of high-performance deck concrete with a shrinkage-reducing admixture, as well as transverse post-tensioning at the pier caps, helps maximize the durability of the structure. With the second phase of the interchange preparing to advertise and the final phase beginning design, NDOT is confident in the choice of CIP post-tensioned box girders as the best solution for the Centennial Bowl Interchange.

Michael Taylor is a senior structures engineer with the Nevada Department of Transportation in Carson City, Nev.

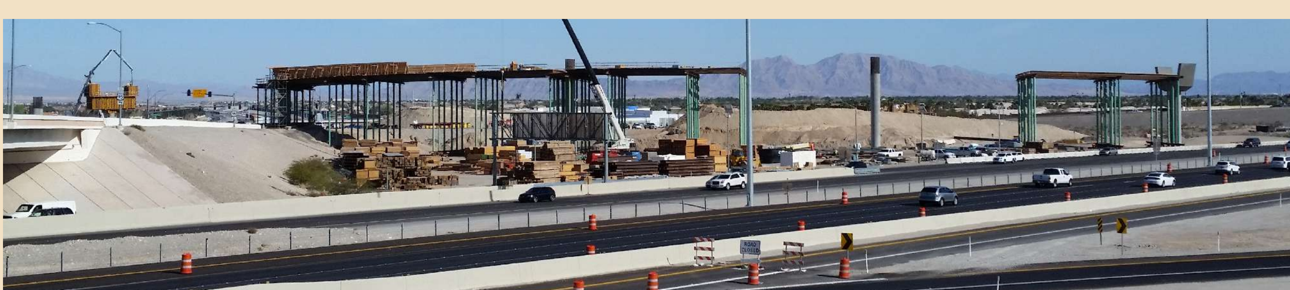
AESTHETICS **COMMENTARY**

by Frederick Gottemoeller

For aesthetic as well as structural and economic reasons, box girders are an eminently suitable structural type for the flyover ramps of complex interchanges. All the lines of the superstructure—the tops of the parapets, the edges of the slabs, and the corners of the box girders—are parallel to each other and to the trajectories of the vehicles traveling across the bridge. The thin, single-shaft piers with hidden or minimal pier caps impose no cross lines or barriers to this visual flow. The pier shafts themselves are thin enough to keep the space of the interchange visually open and unencumbered, allowing drivers to see through to converging ramps and merge areas.

Although the use of cast-in-place concrete box girders can be problematic in locations where construction has to be done over existing roadways, this project successfully met the challenge. With a great deal of ingenuity, the team found methods to build the girders while still keeping traffic moving underneath. Hopefully, their experience will reassure others who are considering applying this structural type.

The slightly dropped pier caps at all of the intermediate piers add an intriguing rhythm to this structure. They punctuate the eye's progress along the curves, while at the same time creating some visual consistency with the dropped pier caps at the expansion joints. Coloring the box girder itself with a hue that contrasts with the colors of the slabs, parapets, piers, and pier caps makes obvious the visual ribbon of the girder. When viewed in the Las Vegas sunshine, the ramp is a memorable structure.



Progress photo of placement of falsework for frame 2. Photo: Nevada Department of Transportation.

Chief Joseph Dam Bridge
Replacing the old timber truss bridge

By Jason B.K. Pang, KPFF Consulting Engineers

The Chief Joseph Dam Bridge crosses the Foster Creek ravine just upstream of where it meets the Columbia River, below the western crest of the Chief Joseph Dam in Bridgeport, Wash. Since 1959, the bridge has served as an important freight route and primary access to the agricultural and recreational region upstream of the dam. In 2016, the existing bridge was replaced with what was the longest single-span precast concrete spliced-girder bridge in Washington state at the time of construction.

Background

Chief Joseph was a prominent Nez Perce Native American leader. The dam was named in honor of him, and the bridge was subsequently named after the dam.

Before it was replaced, the existing 309-ft-long bridge consisted of a unique 130-ft-long and 20-ft-deep Howe deck-truss main span constructed with glued laminated Douglas fir members and five timber approach spans. Designed by the U.S. Army Corps of Engineers, the bridge was certainly an unusual structure, even for its time. It was constructed during an era when steel replaced wood as the preferred material. It was the only bridge of its kind designed and constructed for highway use in Washington state in the 1950s, and, for many years, it was the only Howe deck-truss bridge remaining in the Washington state bridge inventory. Because of its rare structure type, age, and association with the Chief Joseph Dam, the main span was placed on the National Register of Historic Places.

Despite being retrofitted in 2003, the bridge had major structural deficiencies and needed to be replaced. After evaluating several alternatives, the designers selected a single-span precast concrete, post-tensioned spliced-girder bridge as the preferred solution to address the unique challenges of the project.

Major Project Challenges

The nearly 45-degree side slopes of the ravine, coupled with the presence of large boulders and sharp drop-offs of the bedrock, made site access and foundation construction major challenges. Access was further complicated by strict environmental requirements. No structural or construction activities were allowed in the creek between the existing piers. Despite the creek being dry most of the year, it was highly restricted because of its environmentally fragile nature. Because Chief Joseph Dam is not equipped with fish ladders, Foster Creek is the last creek along the Columbia River that naturally supports the spawning of wild salmon and steelhead. Another issue was the historical significance of the site.

Reinforcing steel and post-tensioning ducts installed in the 100 in.-deep WSDOT "supergirder." For the end segments, the girder web was widened at the end to accommodate the post-tensioning anchorages. All Photos: KPFF Consulting Engineers.

The completed Chief Joseph Dam Bridge in Bridgeport, Douglas County, Wash.

profile

CHIEF JOSEPH DAM BRIDGE / BRIDGEPORT, DOUGLAS COUNTY, WASHINGTON

BRIDGE DESIGN ENGINEER: KPFF Consulting Engineers, Seattle, Wash.

PRIME CONTRACTOR: Cascade Bridge LLC, Vancouver, Wash.

PRECASTER: Concrete Technology Corporation, Tacoma, Wash.—a PCI-certified producer

POST-TENSIONING CONTRACTOR: DYWIDAG Systems International, Long Beach, Calif.

OTHER SUPPLIERS: Structural Earth Walls: The Reinforced Earth Company, Englewood, Colo.

The existing timber Howe deck truss was moved to the temporary work trestle and used as the platform to erect the girder segments. Detailed inspection and analysis were performed to ensure that the existing truss could safely support the loads.

The entire reach of Foster Creek has a history of human use dating back to precontact Native Americans. Archeological sites in the ravine near the existing bridge limited the bridge replacement to the same alignment.

The biggest challenge was the owner's requirement to salvage the historically significant truss. The design and construction sequences had to address preserving the truss by removing it whole or dismantling it without entering the creek in the steep ravine.

Alternative Bridge Designs

Designers first considered a three-span continuous girder bridge. A 170-ft-long mainspan and two 70-ft-long back spans were required to clear the existing piers and remain outside the sensitive, restricted areas. Construction of the two intermediate piers would have been difficult and costly, given the geotechnical conditions and lack of access in the ravine. Furthermore, transporting a 170-ft-long girder to the site would have been problematic, if not impossible.

Constructing a single-span bridge on tall fill abutments was the most economical and feasible option. A single-span steel plate girder bridge was considered. However, the owner had concerns about the cost of long-term maintenance of a steel bridge and firmly preferred a precast, prestressed concrete bridge.

A single-span concrete girder bridge using the Washington State Department of Transportation (WSDOT) "supergirder" was the best option. The precast, prestressed concrete, post-tensioned spliced girder allowed designers to extend the span range of conventional precast concrete girders to eliminate costly intermediate piers on the steep-sloped ravine and avoid affecting the environmentally sensitive area, while satisfying the owner's requirement for an all-concrete bridge. Spliced girders also allowed for shorter and manageable precast concrete components to be transported to the remote site, and designers could resourcefully use the existing piers as temporary supports for splicing the girders.

"Supergirder" Bridge

The new two-lane, single-span bridge is 240 ft long and 32 ft wide. The bridge is framed by five girder lines, spaced at 6.25 ft, each consisting of three precast, pretensioned concrete segments (49, 136, and 49 ft in length). The splice locations were strategically located so that the span could be erected on falsework supported on the intermediate piers, avoiding the need for a temporary structure.

Girder segments use the 100-in.-deep WSDOT WF100PTG "supergirder" section and were constructed with high-performance concrete with a 28-day design compressive strength of 10.8 ksi. The segments were pretensioned to support self weight and ensure zero tension during shipping and erection.

Segments were joined together on site with a 2-ft-wide cast-in-place closure pour, where longitudinal mild steel reinforcment was lap spliced and post-tensioning ducts were coupled. Each girder was post-tensioned with four 19-strand tendons using 0.6-in.-diameter, low-relaxation Grade 270 strand.

The completed girders after the post-tensioning tendons were installed and tensioned. Stirrups were prebent to provide clearance during hauling.

DOUGLAS COUNTY, WASHINGTON, OWNER

BRIDGE DESCRIPTION: The two-lane bridge is 32 ft wide and composed of a single post-tensioned 240-ft long span and a 69-ft earth-filled approach. The superstructure is framed by five girder lines, each consisting of three precast, pretensioned concrete segments erected on existing bents and post-tensioned together. The abutments consist of cast-in-place concrete and structural-earth walls.

STRUCTURAL COMPONENTS: WSDOT WF100PTG precast, prestressed spliced, post-tensioned concrete girders (five girder lines, three segments each: 49-ft, 136-ft, and 49-ft); 7.5-in-thick cast-in-place concrete deck; cast-in-place concrete and structural-earth-wall abutments

BRIDGE CONSTRUCTION COST: \$3.8 million (\$495/ft^2)

BRIDGE CONSTRUCTION COST: 2018 PCI Design Award - Transportation: Best Main Span More than 150 feet; WSDOT Local Programs 2017 Award of Excellence – Best County Project; Washington Aggregates & Concrete Association 2016 Excellence in Concrete Award – Public Works: Bridges

The existing truss bridge was lifted onto the completed superstructure and dismantled.

Instead of a conventional single parabolic curve, the tendon profile consisted of two parabolic curves in the end segments and a tangent straight profile in the middle segment. This was done to reduce the high stresses at the closure pours and allowed for concrete with a lower strength than used in the girder to be placed in the closure pour. Designers opted to use a lower-strength concrete because higher-strength mixtures were of limited availablity and would cost more to deliver to the remote location. The closure pours required a strength of 6 ksi at post-tensioning and a 28-day strength of 7.5 ksi.

Each tendon was tensioned to 835 kips and had a final effective force of 730 kips. Post-tensioning was applied to the girders before the deck was placed for two reasons. First, the extra weight of the concrete deck would have overloaded the existing piers that were used as temporary supports. Second, this sequence allows the deck to be extensively rehabilitated or replaced in the future, if needed, without concerns about overstressing the girders. An even longer span could have been achieved if the composite deck were in place when the girders were post-tensioned.

Precamber

Vertical deflection of the spliced girder from self weight and superimposed dead load was estimated to be 15 in., which is considerable. To ensure a smooth road profile and no sag, and to minimize the girder haunch, the girder profile was precambered. This was achieved by elevating the temporary supports at the closure pours 5 in. above a straight line connecting the ends of the girder. After casting the closure pour and post-tensioning the girder, a final camber of 3 in. was achieved in the girder soffit profile.

Girder Transportation

The distance from the precaster to the project site was more than 230 miles. The longest girder segment weighed more than 190 kips and was 9 ft 9 in. high, including the length of stirrups extending from top of the girder. To clear the lowest bridge along the haul route, the hauler required the girder stirrups to be prebent with only 5-in. extensions, which reduced the total girder height to 105 in. A special stirrup detail using a hat-shaped bar was used to accommodate the variable haunch depth. These bars formed a splice with the prebent stirrups and extended into the 7.5-in.-thick cast-in-place concrete deck slab. Both the top and bottom layers of the deck-slab reinforcement were epoxy-coated. All reinforcement extending from the precast concrete girders was coated with a zinc-rich primer.

Substructure

The abutments were placed on benches cut into the steep-sloped ravine to accommodate the girders' maximum span. This resulted in the abutments being upward of 37 ft in height. The east and west abutments are supported on 21 and 15 HP16 steel piles, respectively, under a 5-ft-thick pier cap. Overexcavation of the footings was required to remove large cobbles and boulders in the ravine that obstructed pile driving. Structural earth walls were used behind the abutment wing walls to retain the roadway approach fill. To access the site, the contractor constructed a pile-supported work trestle next to the existing bridge. The trestle was used to demolish the existing bridge, construct the abutments, and erect the new girders.

Salvaging the Historic Truss

Extensive analysis of the existing truss was performed, and several construction sequences were developed to either dismantle the truss in place (without entering the creek) or lift it out whole.

The innovation in the ultimate construction sequence was moving the entire existing truss to a temporary trestle and using it as the girder-launching truss. To do so, the strength of the truss was checked against the launching demands, which included the weight of the girder and hauling truck. The recent inspection report of the existing bridge was referenced to establish reasonable strength reduction factors for each structural element. After the new bridge superstructure was completed, the existing truss was lifted onto the new bridge to be dismantled.

Conclusion

The precast concrete, post-tensioned spliced girders were a cost-effective and durable solution that addressed the unique challenges and met the goals of this project, which is an innovative example of stretching the practical use of precast concrete girders to longer spans by splicing girder segments with manageable weights and lengths for transportation. In this project, precast concrete was the best solution to minimize disruption to the natural environment, while promoting constructability, providing durability, and facilitating the salvage of a historic structure.

Jason B.K. Pang is a senior project engineer and project manager in the bridge and infrastructure group at KPFF Consulting Engineers in Seattle, Wash.

View of completed bridge from below. Piers from existing bridge remain, which were used to support girder segments at splice locations.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PROJECT

Faunce Corner Road Bridge Over Interstate 195

by Paul W. Berthiaume and John F. Watters, GPI

The Massachusetts Department of Transportation (MassDOT) recently completed construction of a two-span bridge that carries Faunce Corner Road over Interstate 195 (I-195) in Dartmouth, Mass., as part of a larger highway and intersection improvement project. This section of Faunce Corner Road is a heavily traveled area, which, prior to the project, had become congested with vehicle queues extending from the I-195 exit ramps back onto the mainline of I-195.

East elevation of completed Faunce Corner Road Bridge Over Interstate 195. All Photos and Figures: GPI.

Background

Constructed in 1966, the existing bridge was a prestressed concrete girder superstructure supported by concrete gravity abutments and multicolumn bent piers. It comprised five simple spans totaling 244 ft and had a curb-to-curb distance of 40 ft with a single 5-ft-wide sidewalk. The bridge superstructure was performing well, but the deck joints had failed and the piers were heavily deteriorated because the pier caps were directly exposed to roadway salts from the failed joints above.

In response to the vehicular demand in the area, a traffic study was carried out, and it indicated that more travel lanes and sidewalks were required across the bridge. A bridge study was performed to determine whether to undergo a bridge rehabilitation or complete replacement. Because the existing bridge superstructure was doing well, a bridge rehabilitation and widening option was strongly considered. However, the study concluded that—because the existing piers were constricting future I-195 expansion and the existing overhead clearance was only 14 ft 5 in.—a complete bridge replacement that improved these deficiencies was a better overall investment for MassDOT.

The bridge study focused primarily on precast, prestressed concrete and steel plate girder solutions because of the anticipated 120-ft span range. The bridge's history showed that precast concrete girders could perform well in the environment, which made it difficult to not be biased toward that solution. There were some initial concerns that the weight of the concrete girders would create large reactions and potentially require large and uneconomical substructure elements. However, this bridge site had the benefit of high bedrock, which allowed the footings to bear either directly on rock or on a lean concrete fill to rock. This fact alleviated concerns about needing large footings or deep foundation elements as the project moved further into the design phase. Based on cost and service-life considerations, the study ultimately concluded that the bridge should be completely replaced and precast, prestressed concrete girders should be used.

Precast, prestressed concrete girders were selected over weathering-steel girders because the project is located within 5 miles of the ocean and in a corrosive environment. However, the use of prestressed concrete girders involved certain design constraints. To reduce the formation of cracks in the girders, the bottom flange was designed to meet MassDOT's most stringent requirements and to have minimal tensile stresses in the precompressed tensile zone under the Service III limit state after prestressing losses have occurred. Reducing the formation of cracks lowers the risk that chloride contamination might reach the

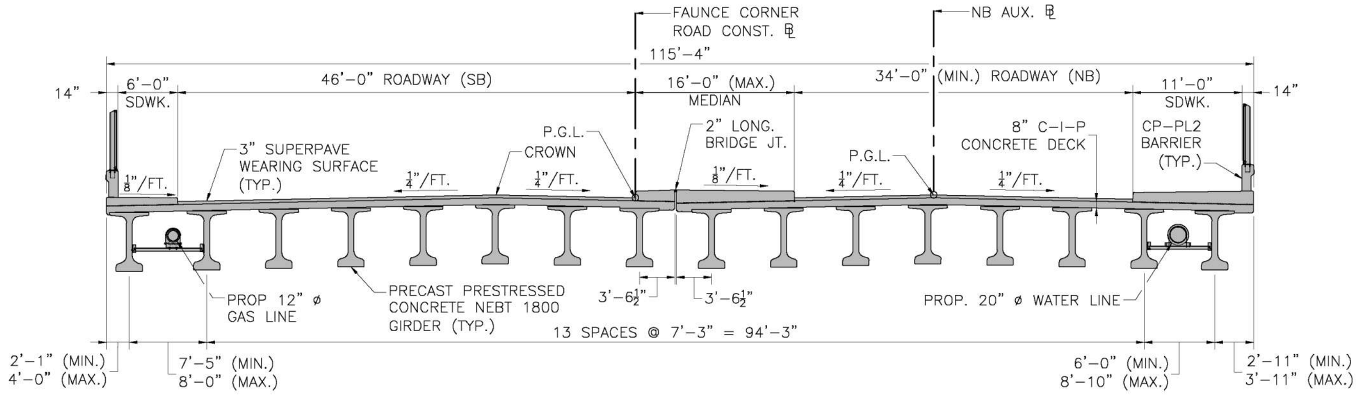
profile

FAUNCE CORNER ROAD BRIDGE OVER INTERSTATE 195 / DARTMOUTH, MASSACHUSETTS

BRIDGE DESIGN ENGINEER: GPI, Wilmington, Mass.

PRIME CONTRACTOR: J.H. Lynch & Sons Inc., Cumberland, R.I.

PRECASTER: J.P. Carrara & Sons Inc., Middlebury, Vt.—a PCI-certified producer



Typical bridge cross section.

prestressing strands and extends the service life of the girders.

Bridge Configuration

The new bridge is composed of Northeast Bulb Tee (NEBT) 1800 (1800-mm-deep [70.9-in.-deep]) precast, prestressed concrete girders with a cast-in-place, 8-in.-thick, reinforced composite concrete bridge deck. The girders are supported by cast-in-place concrete cantilever abutments and a single cast-in-place concrete multi-column pier.

The bridge pier is centered in the median of I-195, and the abutments are located 30 ft back from the edge of the closest lane, which set the spans at 120 ft each. The new bridge has an overall out-to-out width of 115 ft 4 in., with slight variations due to the horizontal roadway alignment. The roadway consists of a 6-ft-wide sidewalk, an 11-ft-wide sidewalk, a variable-width median (16 ft maximum), a 46-ft curb-to-curb width southbound, and a 34-ft (minimum) curb-to-curb width northbound. The bridge also carries a 12-in.-diameter gas line on one exterior bay and a 20-in.-diameter water line on the opposite exterior bay.

Because of the width of the bridge, a longitudinal joint was constructed down the center of the bridge, which essentially created two independent superstructures. To limit thermal movement of a bridge and stress on the bearing assemblies, MassDOT's general policy is to provide a longitudinal bridge joint in the median if the bridge width exceeds 72 ft.

The bridge profile is on a 300-ft-long vertical curve with 2.51% entry and -3.87% exit grades, and the roadway alignment over the bridge is on a 3800-ft-radius horizontal curve. The new design removed multiple piers from the I-195 shoulders, which allows for future expansion and also provides 16 ft 6 in. of minimum vertical overhead clearance for I-195.

The roadway vertical curve crest was located in the second span and required close geometric coordination with the anticipated residual girder camber to lessen the depth of haunches at time of deck placement. This geometric coordination allowed the design to reduce additional haunch dead load and avoided complications of installing variable-depth stirrups to create composite connections with the deck slab. The bridge seat elevations and sole plate slopes had to be determined through computer-aided design modeling to ensure that the final beam would provide full contact to the 24-in.-diameter reinforced elastomeric bearing pads. MassDOT prefers to use elastomeric bearing pads where possible; in this case, they were sized and designed to meet all requirements of the American Association of State Highway and Transportation Officials (AASHTO). The bearing pads were located at both ends of the girders.

With the 120-ft span lengths and an average girder spacing of 7 ft 3 in., NEBT 1800 girders were selected because

Completed stage I bridge in service while stage II is under construction.

MASSACHUSETTS DEPARTMENT OF TRANSPORTATION—HIGHWAY DIVISION, OWNER

BRIDGE DESCRIPTION: A 240-ft-long, two-span continuous for live load, precast, prestressed concrete bulb-tee girder bridge

STRUCTURAL COMPONENTS: Thirty-two (16 per span) precast, prestressed concrete NEBT 1800 girders, 8-in. cast-in-place composite concrete bridge deck, cast-in-place concrete cantilever abutments, and cast-in-place concrete multi-column pier

BRIDGE CONSTRUCTION COST: $6.35 million bid cost (approximately $230/ft^2)

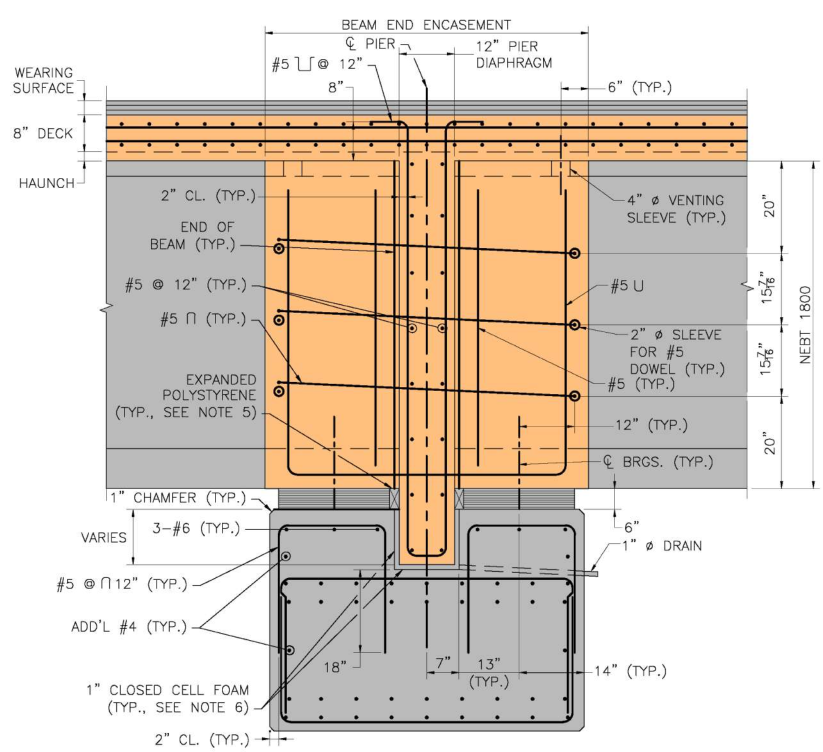
View beneath completed bridge showing the center pier and abutment.

they were the only MassDOT standard precast, prestressed concrete girder that had the load-carrying capacity for such a span and beam spacing. The NEBT 1800 girders also have a wide top flange, which facilitated the use of splayed-girder framing for the exterior girders, with girder spacings that varied from 6 ft 0 in. to 8 ft 10 in. The straight bulb-tee girders also accommodated the horizontally curved roadway deck slab that required variable width overhangs ranging from 2 ft 1 in. to 4 ft 0 in.

The girders were designed using the 2012 *AASHTO LRFD Bridge Design Specifications* methodology and were analyzed as simply supported for dead loads and continuous for live loads.The bridge was constructed in two stages, with the first stage being built immediately adjacent to the existing bridge. This construction method allowed the existing bridge to stay completely in service during stage I construction. The second stage of construction occurred within the approximate footprint of the existing bridge, while traffic was shifted to the new stage I structure.

Girder Design Features

The girders were made continuous for live loads through the use of reinforcing steel in the deck slab combined with a full-depth beam end encasement detail, which eliminated the need for an expansion or control joint over the pier.



Beam end encasement detail at pier, providing continuity for live loads and eliminating joints over piers.

This was a key element to this structure, given that one of the major deficiencies of the previous structure was leaky deck joints over the piers. The concrete girders also have a lower coefficient of thermal expansion, as compared with steel girders, which allowed for the use of simpler expansion joint details at the abutments. The simpler joint detail will require less long-term maintenance and offer better protection of the girder ends, where corrosion can typically be a problem.

Due to the long span length and bridge loading, the girder design used all 10 available harped strand locations within the girder. The girder design also required a concrete strength of 8 ksi, as opposed to MassDOT's standard 6.5-ksi concrete strength for prestressed concrete girders. MassDOT permits use of an increased concrete strength to avoid going to deeper sections or creating closely spaced girders. In addition to harped strands, the girder design required debonding for the first 18 to 24 in. for specific strands at the girder ends to reduce compressive stresses at the time of prestressing force transfer.

Providing durability

All elements of the structure, with exception to the approach slabs, are reinforced with epoxy-coated reinforcing bar, which is a MassDOT standard. MassDOT also requires the use of high-performance concrete in all bridge decks, end diaphragms, and precast, prestressed concrete beams.

MassDOT specifications require that high-performance concrete include the use of a corrosion inhibitor such as calcium nitrite and be air-entrained for freezing-and-thawing resistance. The use of these materials is specified in Massachusetts to provide structures that are resistant to the high-chloride environment to which they are exposed during long, cold winter months throughout their service life. These materials, combined with careful detailing, will provide a durable, economical structure with immediate and long-term cost savings.

Paul W. Berthiaume is a project manager and John F. Watters is vice president/ director of structural engineering with GPI in Wilmington, Mass.

Admixtures for Concrete

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

The American Concrete Institute (ACI) defines an admixture as "a material other than water, aggregates, cementitious materials, and fiber reinforcement, used as an ingredient of a cementitious mixture to modify its freshly mixed, setting, or hardened properties and that is added to the batch before or during its mixing."[1]

Admixtures are used in concrete for bridge structures to provide air entrainment, reduce water content, improve workability, retard setting times, accelerate strength development, provide flowable or self-consolidating concrete, reduce shrinkage, reduce the potential for corrosion, and prevent washout of concrete placed under water. This article summarizes the more frequently used admixtures for concrete used in bridge structures. Further details are given in *Report on Chemical Admixtures for Concrete (ACI 212.3R-16)*.[1]

Air-Entraining Admixtures

Air-entraining admixtures are used primarily to increase the resistance of concrete to damage from freezing and thawing. They may also be used to increase workability and facilitate handling and finishing. The relevant specification is American Association of State Highway and Transportation Officials' AASHTO M 154.[2]

Chemical Admixtures

Chemical admixtures are classified in AASHTO M 194[3] as follows:

- Type A: Water-reducing
- Type B: Retarding
- Type C: Accelerating
- Type D: Water-reducing and retarding
- Type E: Water-reducing and accelerating
- Type F: Water-reducing, high-range
- Type G: Water-reducing, high-range, and retarding

Water-reducing admixtures and high-range water-reducing admixtures are used to allow for a reduction in the water–cementitious materials ratio of the concrete, while maintaining or improving workability. Accelerating admixtures are used to decrease the setting time and increase early strength development. Retarding admixtures are used to increase the setting time to allow more time for transportation and placement. According to a 2012 survey, not all seven admixture types are permitted by every state department of transportation. Types A and F were identified as the most frequently used types.

Shrinkage-Reducing Admixtures

Shrinkage-reducing admixtures have the potential to reduce long-term shrinkage by 25% to 50%. The admixture works by reducing the surface tension effects that contribute to drying shrinkage of the hardened concrete. They are not covered by any AASHTO or ASTM standard.

Corrosion Inhibitors

Corrosion-inhibiting admixtures either extend the time to corrosion initiation or significantly reduce the corrosion rate of embedded metal, or both, in concrete containing chlorides in excess of the accepted corrosion threshold value for the metal in untreated concrete. Performance requirements and test methods are described in ASTM C1582.[4]

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Dr. Henry G. Russell is an engineering consultant and former managing technical editor of ASPIRE®.


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Using Fully Bonded Top Strands in Pretensioned Concrete Bridge Girders

by Dr. Bruce W. Russell, Oklahoma State University

Fully bonded top strands provide significant benefits for precast, prestressed concrete bridge girders. By using fully bonded top strands, several serviceability issues are improved relevant to the engineer's ability to control stresses in the end regions and eliminate or mitigate unwanted cracking:

- Harped (or deflected) strand patterns can be eliminated for most designs.
- The need for debonded strands is eliminated in many designs, and, where debonding is required, the required length and number of debonded strands are significantly reduced.
- Girder camber is reduced in both the short term and long term.
- Cracks that occur in end regions at detensioning can either be eliminated or reduced in both number and size.

Many engineers, however, do not take advantage of this reasonable approach because of concerns that fully bonded top strands add tension to the bottom fiber. There are also concerns that top strands may reduce the flexural strength of the cross section. This article asserts that increases to bottom-fiber tensile stresses are small, easily overcome, and usually inconsequential; moreover, the flexural strength of the cross section is largely unaffected. It is the opinion of the author that the positive benefits of using fully bonded top strands far outweigh any adverse effects on service load stresses.

Background

My interest in this topic grew out of my research (while at the University of Texas at Austin) with Dr. Ned H. Burns to determine rational design guidelines for debonding strands. The overarching principle that we followed in our recommendations,[1-4] which was also presented in an earlier paper by Horn and Preston,[5] is that debonding strands should be minimized in number and length because the pretensioned concrete beam or girder is inherently stronger with bonded strands than with unbonded strands. While the topic of fully bonded top strands was not part of our original conclusions, a paper published in 1994 indicated that fully bonded top strands improve the efficiency of the prestressing strand pattern, and that one of those efficiencies was the elimination or minimization of the need for debonding.[6]

In 1997 and 1998, Dolese Bros. Co. of Oklahoma City, Okla., hired me to redesign strand patterns—from harped to straight—for two sets of bridges. The first, the Interstate 35 (I-35) Bridge over Rollercoaster and Pine Roads in Logan County, Okla., is featured in the example in this article. In this design example, the middle span, with 103.43-ft center-to-center bearings, was constructed from American Association of State Highway and Transportation Officials (AASHTO) Type IV girders spaced at 8.2 ft.

This I-35 bridge is the first bridge in Oklahoma designed to use straight strand patterns in lieu of harped strands. Also, and as part of the redesign, this bridge became the first in Oklahoma designed using the new (first edition) *AASHTO LRFD Bridge Design Specifications*[7] in place of AASHTO's *Standard Specifications for Highway Bridges*.[8]

There have been changes to the AASHTO LRFD specifications since that first edition—most notably the fact that the relatively new code provisions for prestress losses reduce the required number of prestressing strands. Another change is that the allowable compressive stress for temporary stresses is now $0.65f'_{ci}$ instead of $0.60f'_{ci}$.

Design

Figure 1 illustrates the strand pattern

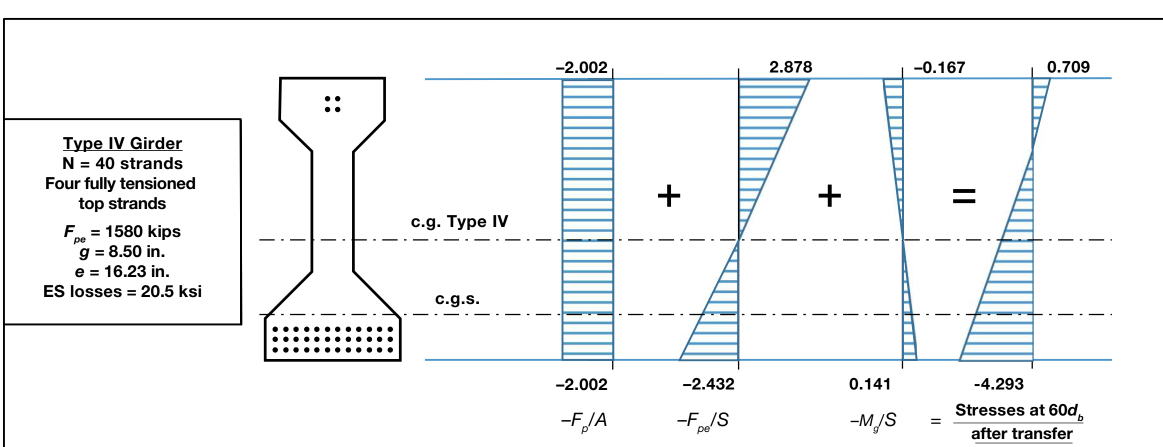

Figure 1. Calculated stresses at transfer at $60d_b$ from end of AASHTO Type IV bridge girder with straight strand pattern and four fully bonded top strands but with no debonded strands. Note: Compressive stresses are negative, stresses shown are in ksi, and figure is not to scale. All Figures: Dr. Bruce Russell.

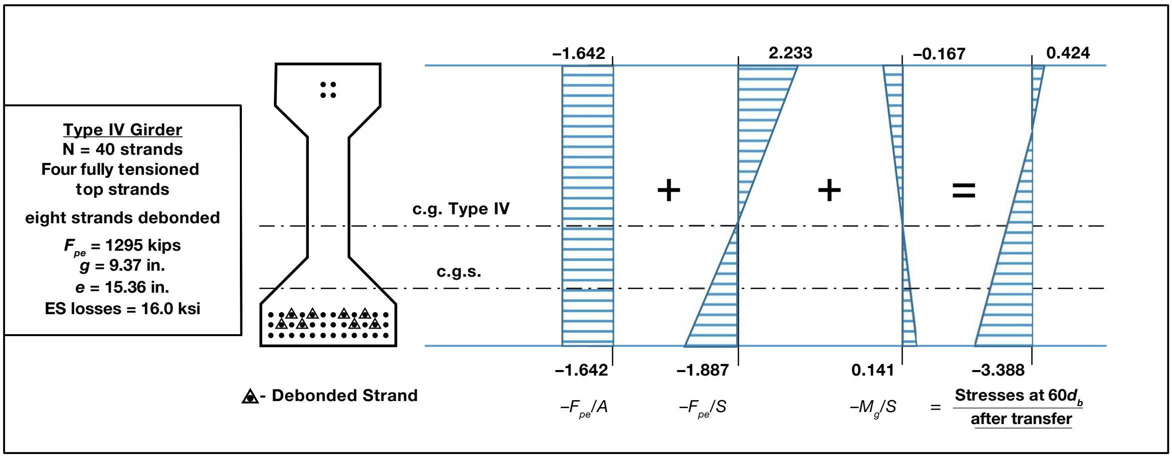

Figure 2. Calculated stresses at transfer for same girder and location as Fig. 1 but with eight of the bottom strands debonded.
Note: Compressive stresses are negative, stresses shown are in ksi, and figure is not to scale.

for the cross section that contains forty 0.6-in.-diameter, Grade 270 low-relaxation strands conforming to ASTM A416. Thirty-six strands are located in the bottom flange, and four fully bonded straight strands are located near the top of the cross section. The specified concrete strengths were 6.5 ksi at transfer and 8.7 ksi at 28 days. The strand pattern shown in Fig. 1 does not include debonded strands. The beam stresses shown in the figure are computed after transfer at $60d_b$ (36 in.) from the end of the beam, as if debonding were not applied. Please note that d_b is the diameter of the strand, compressive stresses are shown as negative stresses, and stress units are ksi.

Figure 2 shows the cross section, including the eight strands that were debonded, and illustrates the stresses after transfer at the location $60d_b$, but with eight of the bottom strands debonded. Significant differences from debonding eight bottom strands include the following:

- Reduces the prestress force BEFORE transfer P_{pi} by 25%.
- Reduces the effective prestressing force AFTER transfer F_{pe} by approximately 18%.
- Decreases the eccentricity of the center of gravity of the strands by 0.88 in. Engineers often do not anticipate this factor or appreciate its effects. The change in eccentricity is an important benefit derived directly by combining fully bonded top strands with debonding of bottom strands.
- Reduces the elastic shortening loss ES by 22%.
- Significantly reduces both the maximum tensile and compressive stresses at the extreme fibers.

Not fully discussed nor illustrated is the significant reduction in girder curvature brought about by the inclusion of top strands. This factor, perhaps more than any other, helps mitigate the incidence of web cracking in end regions of prestressed concrete beams upon detensioning.

Tensile Stress in Bottom Fiber at Service III Limit State

Figure 3 illustrates the concrete stresses at midspan for the non-composite girder and the composite girder (Type IV girder with cast-in-place slab) and includes the effect of pretress losses. Since the stresses are computed at midspan, debonding has no effect. The figure shows that the tensile stress at the bottom fiber is less than the allowable stress in the current AASHTO LRFD specifications. Stresses in beams with fully bonded top strands were also compared with stresses in beams with no top strands. **Table 1** shows the tabulation. For this design case, with AASHTO Type IV girders at 8.2 ft spacing and spanning 103 ft 5 in., the tabulation shows that the addition of four fully bonded top strands added 137 psi of tension to the bottom fiber at midspan. There will be some design cases where the addition of fully bonded top strands may require a designer to add another pair of strands to the bottom strand pattern. This option is an economical alternative that the engineer may wish to consider.

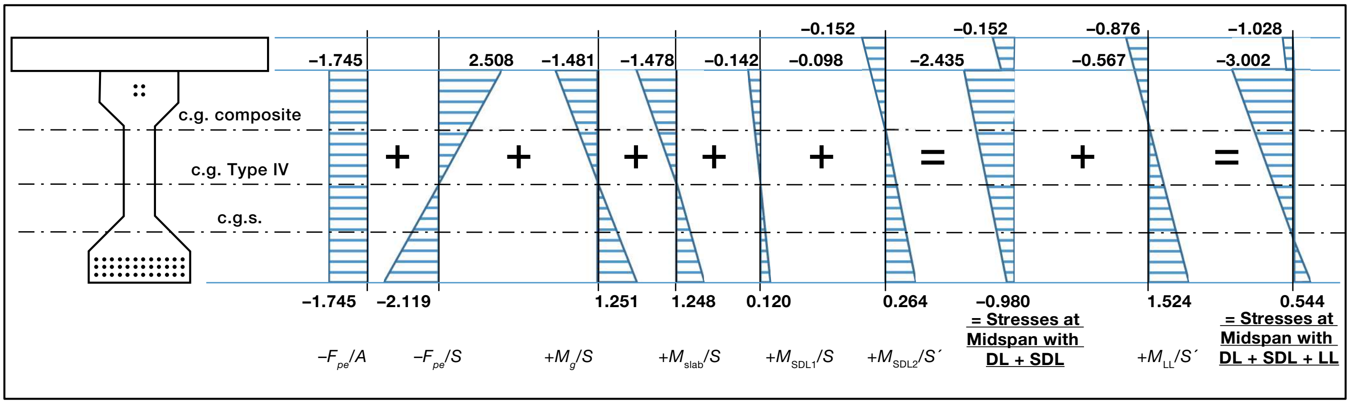

Figure 3. Stresses at midspan at service load for same girder as Fig. 1, with or without debonded strands.
Note: DL = dead load; LL = live load; SDL = superimposed dead load. Compressive stresses are negative, stresses shown are in ksi, and figure is not to scale.

Discussion and Conclusions

The intention of this article is to document the practice of using fully bonded top strands in the precast, prestressed concrete bridge industry. My hope is that engineers, owners, and precasters will read this article and consider implementation of these basic principles. I have asserted that the inclusion of fully bonded top strands can provide benefits that improve the serviceability of bridge beams. These improvements will be manifested in reduced concrete stresses in end regions, and by reducing cambers and decreasing the incidence of cracking near end regions that may occur during detensioning.

This article outlines the basic design principles for the design of straight strand patterns. Some engineers may find the assertions and conclusions challenging. As a response, I encourage readers to work out the controlling stresses, "play" with different strand patterns, and see whether they come to the same or similar conclusions. The design of the I-35 bridge over Rollercoaster and Pine Roads and its subsequent maintenance-free performance over the past 20 years illustrates that fully bonded top strands can be integrated into the design and construction of prestressed concrete bridge girders with favorable results.

Finally, I believe there is conclusive evidence from both theory and practice that suggests that bridge designers and bridge builders should consider the regular inclusion of fully bonded top strands along the whole length of a bridge girder. It is my opinion that this practice will help control stresses in end regions, work symbiotically with the practice of debonding strands, limit cambers, and improve the overall serviceability of precast, prestressed concrete bridges.

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Dr. Bruce W. Russell is the director of the Bert Cooper Engineering Laboratory and associate professor of Civil and Environmental Engineering at Oklahoma State University in Stillwater, Okla.

Table 1. Concrete stresses at midspan at service load comparing designs with straight strands. Case I: thirty-six bottom strands as in example girder; Case II: Case I plus four fully bonded top strands. The values show that the addition of the top strands adds 137 psi tension to the bottom fiber of the composite section.

Concrete Stresses at Midspan at Service Loads After All Prestress Losses (ksi) Both designs contain 36 fully tensioned bottom strands				
DL + SDL				
	Case I No Top Strands		**Case II Four Top Strands**	
	f_{top}	f_{bot}	f_{top}	f_{bot}
Prestress-Axial (F_{pe}/A) Prestress-Bending (F_{pe}/S)	–1.568 2.880	–1.568 –2.433	–1.745 2.508	–1.745 –2.119
Stresses due to Prestressing Only ([F_{pe}/A] + [F_{pe}/S]) (Difference: Case II - Case I)	**1.312**	**–4.001**	**0.763** (-0.549)	**–3.864** (+0.137)
Girder Self Weight (M_g/S) Slab Weight (M_{slab}/S) SDL on Non-Composite (M_{SDL1}/S) SDL on Composite (M_{SDL2}/S')	–1.481 –1.478 –0.142 –0.098	1.251 1.248 0.120 0.264	–1.481 –1.478 –0.142 –0.098	1.251 1.248 0.120 0.264
Total Stresses with DL + SDL	**–1.887**	**–1.118**	**–2.435**	**–0.980**
Add Effect of LL: DL + SDL + LL				
LL on Composite	–0.567	1.524	–0.567	1.524
Total Stresses @ Midspan @ DL + SDL + LL (Difference: Case II - Case I)	**–2.454**	**0.407**	**3.002** (+0.548)	**0.544** (+0.137)

Note: DL = dead load; LL = live load; and SDL = superimposed dead load. Compressive stresses are negative, and stresses shown are in ksi.

West Seventh Street Bridge, Fort Worth, TX
photo: Liam Frederick

Using Maturity Systems to Evaluate Concrete Strength

by John Gnaedinger, Con-Cure, and Danielle Shultz, CTL Engineering

Many people in the bridge-building community may be surprised to learn that the maturity method, introduced 20 years ago as ASTM C1074 *Standard Practice for Estimating Concrete Strength by the Maturity Method,*[1] can be incredibly beneficial for every stakeholder in the building process, including contractors, departments of transportation (DOTs), concrete producers, and testing agencies. The maturity method (often just called "maturity") is a vital supplement to traditional methods of testing concrete that are required by codes and specifications.

Applications in Bridges

Maturity is recognized by many state DOTs as an alternative to field-cured sample testing for determining in-place concrete strength for low-risk applications such as slab-on-grade fast-patch pavement repairs and mainline paving,[2] but its use in bridge construction is not nearly as universal. Using maturity can have substantial benefits for all aspects of concrete bridge construction, especially for high-risk projects where post-tensioning and cold-weather operations are commonplace.

Estimating concrete strength using maturity has been in place for more than 60 years and is based on the fact that concrete gains strength at a rate that is generally proportional to the temperature history of the concrete during the curing process. Maturity/strength relationships are mixture-specific (relationships must be determined for each mixture's proportions), but as long as the mixture constituents and proportions do not change, maturity estimates are remarkably accurate.[3,4]

There are several different ways to collect the field data needed to estimate concrete strength using maturity. Traditionally, sensors embedded in the fresh concrete record concrete temperatures over time via a data-logging device; then, at various ages, the data are "read"; and an assessment of the strength is made with software using the procedures of ASTM C1074. Today, the most innovative maturity-collection systems provide data to the user in real time via the internet.

Regardless of the type of maturity data-logging device used, the results are the same: in-place concrete strength estimates that are far more accurate than traditional field-cured samples could ever be.

Stakeholder Benefits

Each stakeholder in any bridge project has unique criteria to monitor, record, and achieve. The contractor wants to build quickly and safely while turning a profit. The owner (usually a DOT) wants the project done correctly to maximize the service life of the bridge while minimizing disruption to the traveling public. The engineer wants to ensure that the entire structure meets the minimum design requirements. The concrete producer wants to ensure consistent quality and eliminate cylinder breaks that come back too low (often due to improper curing and handling of the cylinders in the field). The regular use of the maturity method on any concrete project can help all stakeholders achieve their goals simultaneously, easily, and reliably, and this is especially true for time-sensitive and high-risk endeavors.

Contractors have reported many benefits from routine use of maturity on their projects. They experience lower project costs because construction schedules can be dramatically shortened. They can use knowledge obtained during other projects to estimate their time and staffing requirements more efficiently, which allows for more competitive bids. Their energy costs are significantly lower in cold weather, and their reliance on third-party testing is reduced. Finally, they can earn early-completion bonuses, if available, when they finish work ahead of schedules.

Black Ankle Valley Bridge for Interstate 69 in southern Indiana. Maturity and temperature were wirelessly monitored for the cast-in-place piers, saving the contractor the costly process of using a lift to collect recorded data. All data were sent wirelessly to the contractor's job-site trailer. All Photos: John Gnaedinger.

Sensors at the Black Ankle Valley Bridge were also used to monitor test cylinders for curing compliance and verification of strengths.

Conclusion

As more contractors become aware of the value of the data generated by a maturity system and the benefits that accrue almost immediately, we predict that maturity will soon earn the position it deserves as a vital quality control and process improvement tool for the entire construction community. Despite being around for decades, maturity is not yet used by the entire concrete industry. However, those who have embraced the methodology are reaping its benefits.

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John Gnaedinger is the founder of and engineering services director for Con-Cure, in Pioneer, Ohio. Danielle Shultz, is project manager for CTL Engineering, in Columbus, Ohio.

Firsthand Account

Danielle Shultz, coauthor of this article, began using maturity systems in 2012. The following recounts a few of her notable experiences using maturity systems on large projects.

My employer was the testing agency for the Interstate 75 rebuild near Lima, Ohio, and we were responsible for all the fresh concrete testing. Initially, we relied exclusively on field-cured cylinders, and it was taking between 10 and 14 days for those test cylinders to reach sufficient compressive strength for the contractor to begin stripping forms and backfilling against the structures. In order for the contractor to backfill, the cylinders had to reach 85% of the design concrete compressive strength (f'_c) per Ohio Department of Transportation (ODOT) specification 511.14-1A. This time-consuming process of waiting for test cylinders to indicate sufficient strength was one reason that the project fell six months behind schedule.

I promoted a maturity system to the contractor, who was unfamiliar with the concept but soon began to appreciate what it could do. Once the maturity curve was established and the sensors were being used on site to estimate in-place concrete strength, we were able to prove that the in-place concrete was achieving 85% of f'_c within 48 hours of placement. Every stakeholder for this important project realized immediately what a huge difference this new approach would make. By using the maturity system to monitor field-cured results and in-place strength data, we were able to get back the six months that the contractor had lost and shorten the project timeline by another three months, which allowed the contractor to receive a $1 million incentive from ODOT for finishing ahead of schedule.

Another instance in which maturity highly benefited a contractor was during winter construction season. The winter prior to implementing maturity, the contractor was required to blanket, tent, cover, and heat concrete until it reached sufficient strength. However, because the contractor was using an excessive amount of heat and covering the freshly placed concrete during the winter months to retain as much moisture as possible, the concrete was often being overheated, which was found to be the root cause of excessive shrinkage cracking and other problems that only appeared much later.

Once maturity was implemented, we were able to demonstrate that extra heat did not need to be added to the tented area because the concrete itself was generating sufficient heat to cure on its own. By reducing the amount of waiting time for the concrete to cure, not having to tent the entire structure, and not using a gas-powered salamander to heat under the tent, the contractor saved more than $50,000 in the first month. The contractor was also able to get ahead of potential problems with the concrete because the maturity system would provide the data to the project manager, technician, contractor, engineer, and owner on how the in-place concrete was behaving and how well it was curing.

We have also used a maturity system to assess the temperature differential and the overall strength for mass concrete placements. Using an innovative wireless maturity system and reusable sensors has allowed us to provide accurate and real-time in-place concrete information for our clients and project owners. Maturity has proven useful on projects that have a tight schedule or strict specifications.

EDITOR'S NOTE

The opinions expressed in this article are those of the authors. Some engineers that reviewed this article stated that using maturity as the only source of strength verification for critical operations is not always advised.

Example of mass concrete temperature and maturity monitoring of a structure in cold weather. Sensors embedded at the core and the surface record temperatures and differentials. All data from the devices are viewable on the internet in real time, saving the contractor valuable time that would have been required to collect and report this information to the engineers.

Precast, Prestressed Concrete Bent Caps: A Faster, Safer, and More Durable Alternative

by Christopher P. Miller, Texas Department of Transportation

The Texas Department of Transportation (TxDOT) has been developing and implementing prestressed, precast concrete bent caps (PPBC) as an efficient, durable, and safe alternative to conventionally reinforced cast-in-place (CIP) concrete bent caps.

Construction Projects

TxDOT first used PPBCs in 2014, on a project located in Bexar County on Texas State Highway Loop 1604. In this project, twin-bridge structures, approximately 2100 ft in length, contained 36 bent caps designed with the same dimensions and reinforcement. The original design called for CIP bents, but the contractor requested post-letting to add a precast concrete option to help reduce construction time. Because of the similarity of the bents, the fabricator could cast them rapidly. Once installation of the PPBCs began, it took only three days to install all of the caps for one of the bridges.

PPBCs were also used for an emergency project to replace a bridge on Fischer Store Road in Hays County that washed out in a major flood event in 2015. For this project, PPBCs were included in the original plan set as an alternative to CIP bent caps. The contractor elected to use PPBCs to expedite construction. Reopening the bridge quickly was important because the nearest detour added nearly an hour to commuters' drives. The contractor installed all of the PPBCs in one day, and the overall bridge was completed in 80 days, opening to traffic in February 2016.

Design

The design philosophy for the initial PPBCs was to match or exceed the ultimate moment and shear capacities of the original CIP design. In Texas, CIP bent caps are designed with Grade 60 reinforcing steel and TxDOT's Class C concrete with a compressive strength of 3.6 ksi. PPBCs are designed with Grade 270, low-relaxation, 0.6-in.-diameter prestressing strands and TxDOT's Class H concrete. The compressive strength of Class H concrete can range from 5 to 8.5 ksi, depending on what is specified in the plans.

The number of strands was selected based on the moment demand, and the strands were placed symmetrically in the cap so that there would be no camber. The transverse reinforcement in PPBCs was designed to match the shear reinforcement in the CIP caps and to provide sufficient resistance to bursting stresses at the cap ends. Because the compressive strength of Class H concrete is higher than that of Class C concrete, the shear capacity of the PPBCs exceeds the shear capacity of the CIP caps with the same transverse reinforcement. Once moment and shear designs were completed, stress checks were performed at different loading situations, including casting yard, transportation/lifting, and service load, to ensure that the caps will not crack.

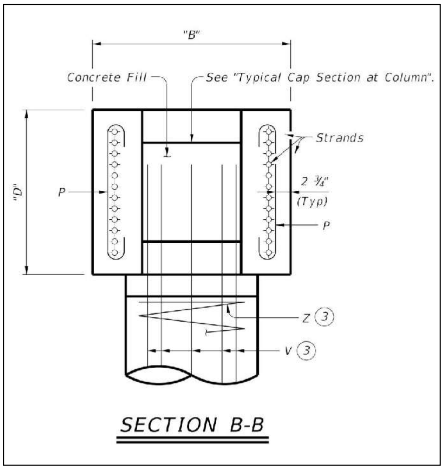
TxDOT PPBC Standard

To date, each PPBC project by TxDOT has been a custom design. In an effort to increase the use of PPBCs, TxDOT published a PPBC standard[1] for use with round columns in April 2017. This standard provides PPBC designs for use as an alternative to TxDOT's standard CIP bent caps. The PPBC standard provides details to support the following TxDOT beam types: TxGirder, box beam, slab beam, decked slab beam, and spread slab beam. These standards account for nearly 2000 different bridge configurations and can be found on the TxDOT Bridge Standards web page.

The design philosophy for the PPBC standard varies slightly from the previous custom designs. Instead of designing the caps to have equal or greater capacities than their CIP versions, the standard PPBC designs are based on the loading from the superstructure. The number of strands and the amount of transverse reinforcement are optimized, and a single design is provided for each different beam type.

Another difference from the earlier designs is the column connection details. The earlier PPBCs were connected to the column

Prestressed, precast concrete bent caps at northbound loop of Texas State Highway 1604. Prior to grouting the column connection, precast concrete caps are supported on shim packs placed on the top of each column. Photo: Texas Department of Transportation.



Cross section of prestressed, precast concrete bent cap at column cap pocket connection.[1] Figure: Texas Department of Transportation.

using grouted vertical ducts. In this older-style connection, column reinforcement was terminated at the top of the column, and dowels extended from the top of the column through vertical ducts that were cast into the PPBC. Once the PPBC was placed on the columns, the ducts were grouted to complete the connection. In the TxDOT standard, the connection has been changed to a cap pocket. In the cap pocket connection, the column reinforcement is extended into a pocket in the PPBC that is formed with a corrugated metal pipe. Once the PPBC has been placed, the pocket is filled with TxDOT Class C or Class S concrete. This change was made so that connecting the cap to the columns would be simpler.

Research

In 2015, TxDOT sponsored a research project to develop standardized details for PPBCs and verify their performance compared with conventional CIP caps. The project tested full-scale PPBCs and CIP caps to compare performance at load levels up to and including failure. It also experimented with different reinforcing steel and connection details to improve structural performance and make construction easier. The final report on the project, which ended in November 2017, has not yet been published. Moving ahead, TxDOT intends to implement findings from research and construction projects and increase the use of PPBCs in bridge projects.

Reference

1. Texas Department of Transportation. 2017. Prestressed, Precast Bent Cap Option for Round Columns (PPBC-RC). ftp://ftp.dot.state.tx.us/pub/txdot-info/cmd/cserve/standard/bridge/ppbcstd1-17.pdf.

Christopher P. Miller is a transportation engineer with the Texas Department of Transportation, Bridge Division, in Austin.

Prestressed, precast concrete bent caps (PPBCs) at Fischer Store Road in Hays County. The contractor installed all of the PPBCs in one day. Photo: Texas Department of Transportation.

SECONDARY MOMENT EFFECTS

in Continuous Prestressed Concrete Beams

by Dr. Andrea Schokker, University of Minnesota Duluth

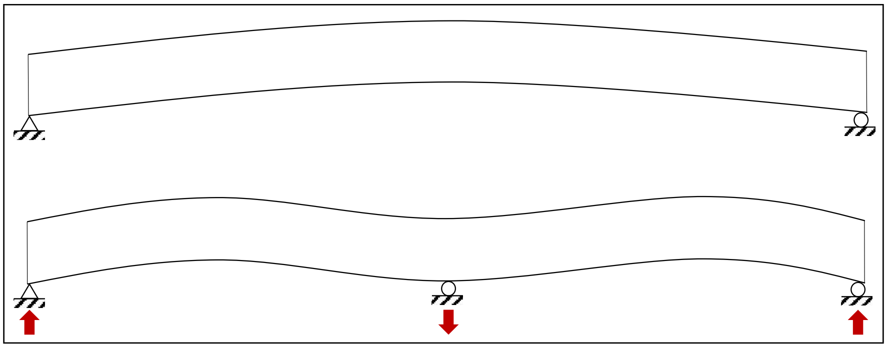
The topic of secondary moment effects in continuous prestressed concrete beams is one that can be tough to understand at first (or so it seems, based on the looks I get when I first introduce the topic in class). Perhaps we need some solid, practical understanding of how prestressed concrete beams behave before we can really have a feel for how these moments are induced. In this column, I will do my best to give a short introduction to the topic for those familiar with prestressed concrete. Most prestressed concrete textbooks and post-tensioned multispan design examples cover the topic in more detail.

What are secondary moments, and why do I have to deal with them in continuous beams but not simply supported beams?

To answer this question, consider a simply supported prestressed concrete beam such as the one shown in Fig. 1 (top). You can visualize the deflected shape due to prestressing directly or think of the prestress force as a uniform load acting upward on the beam. Either way, this beam is determinant, which means the reaction forces can be calculated by equilibrium alone, and when the prestressing force (load) is applied without external loads, the reactions remain zero.

However, the case is different for an indeterminate structure such as a continuous beam. Figure 1 (bottom) shows the same beam with an intermediate support added. Now, the reactions are affected and the center support keeps the beam from taking its preferred deflected shape. This hold-down reaction induces a moment—specifically, a secondary moment.

Figure 1. Deflected shapes of determinate and indeterminate beams under prestress. The top figure shows the curvature due to prestress for a simply supported member. The bottom figure shows the curvature and induced reaction due to prestress for a continuous beam. All Figures: Dr. Andrea Schokker.



So, what does a secondary moment mean for my calculations?

The secondary moment M_2 can be found by subtracting the primary prestressing moment M_1 (the moment due to the prestressing force applied at an eccentricity) from the total moment due to the prestressing force $M_{total\ PS}$:

$$M_2 = M_{total\ PS} - M_1$$

Figure 2 shows a generic illustration of each of the components of moment due to the prestressing force. The top beam shows a typical tendon profile for a multispan, post-tensioned concrete beam. A uniform load representing this profile can be developed, but a simplified tendon profile is used in this case. We can find the total moment due to the prestressing force directly by applying the equivalent uniform load due to the tendon profile and constructing the moment diagram.

The primary moment M_1 is simply the prestressing force multiplied by the eccentricity, or:

$$M_1 = Pe$$

where

P = prestressing force

e = eccentricity of the tendon relative to the center of gravity of the cross section

For the example shown in Fig. 2, the ends of the tendon are located at the center of gravity of the section. As expected, the secondary moment diagram has the shape of a moment diagram from a point load (the reaction). The moment diagrams can be taken from an analysis program, but the moment-distribution method can be a useful tool for hand calculations (this is when my students groan, but hand calculations really are a quick method for continuous beams).

For stress calculations, the total moment due to prestressing force is all we need. However, when we consider the required capacity of the section, ϕM_n, we need to consider the effect of the secondary moment separated out because this will influence the factored moment M_u:

$$\phi M_n \geq M_u - M_2$$

The secondary moment can reduce the required capacity of the section if its effect is the opposite of the effect due to the external loads (such as live and dead loads). As with any calculation, do not use the equations blindly without considering the effect of signs.

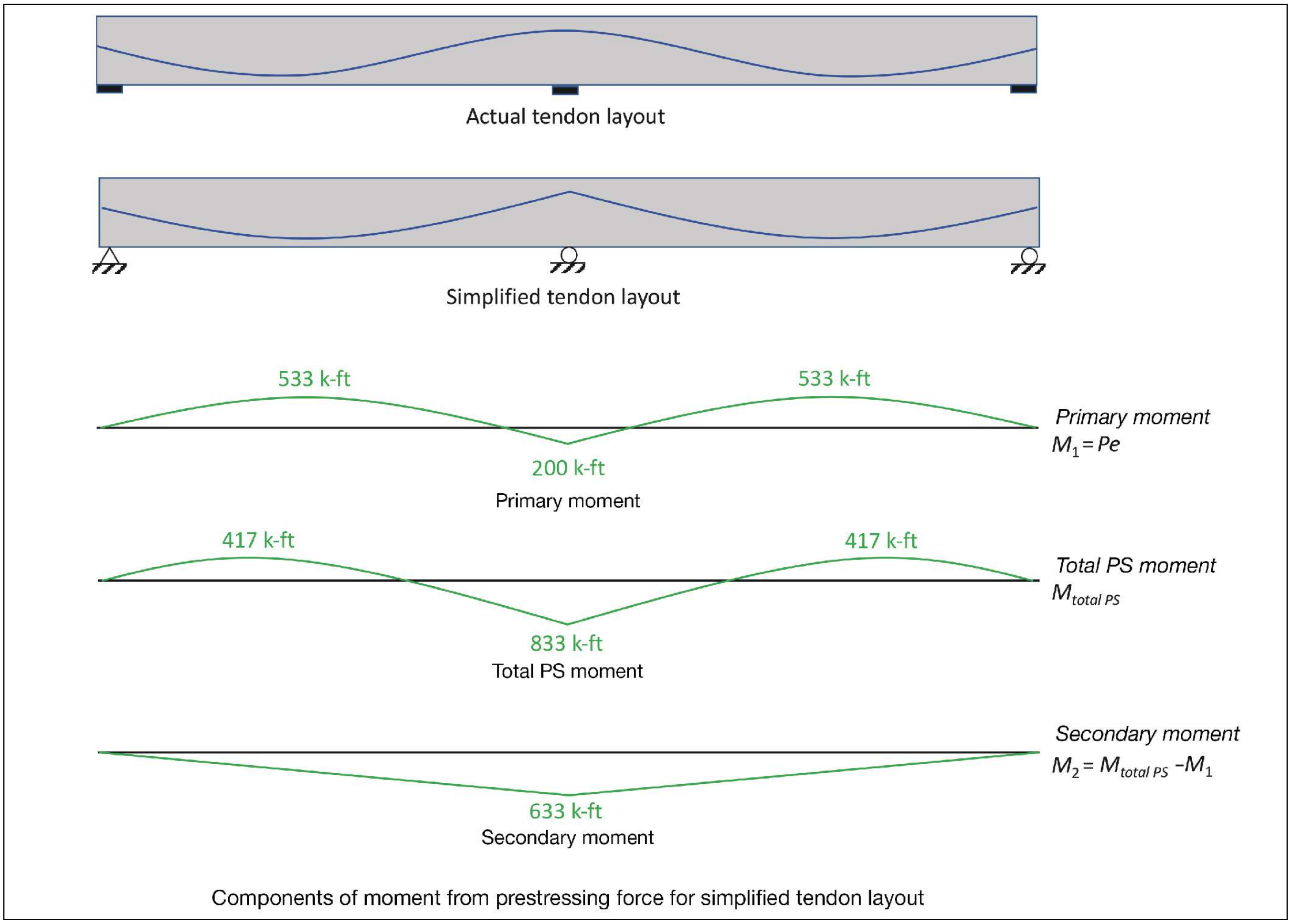
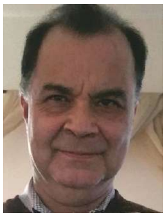


Figure 2. Example of a two-span post-tensioned beam to demonstrate secondary moment. Moments are plotted on tension side, and figures are not to scale.

Foundation Reuse: An Option for Bridge Reconstruction Projects



by Frank Jalinoos, Federal Highway Administration Office of Infrastructure R&D

According to Federal Highway Administration (FHWA) National Bridge Inventory 2017 data, 9% of bridges are structurally deficient and 15% are older than the average design life of 70 years for bridges.[1] Widening, replacement, or significant rehabilitation of these bridges, particularly those in urban areas, is challenging because of mobility and traffic demands. In many cases, bridges will require superstructure replacement, while the foundation still has significant functional value. Therefore, reuse of these foundations can result in significant time and cost savings.

Reconstruction Options

Foundation reuse is herein defined as using existing foundation or substructure of a bridge, as whole or in part, when the existing foundation has been evaluated for new loads. Foundation reuse includes reuse of substructure components both above and below ground, including rehabilitation of existing substructure and foundation elements when the superstructure has been replaced.

Figure 1 illustrates different foundation construction options with the following descriptions:[2]

- In option 1, a new foundation is constructed that avoids the existing foundation. In this case, reconstruction is carried out at a new location without affecting the mobility on the bridge during construction, although switching to the new alignment may involve mobility impacts.
- In option 2, the existing bridge foundation is demolished prior to construction of a new foundation.
- In option 3, the existing foundation is reused as is, with or without minor repairs such as patching or chloride removal.
- In option 4, the existing foundation is reused but with some form of retrofitting or strengthening.

Options 3 and 4 illustrate foundation reuse. The remaining substructure elements of bridges in these two options may also be suitable for reuse, with or without rehabilitation.

The forthcoming *FHWA Best Practice Manual for Bridge Foundation Reuse* (available summer 2018 at www.fhwa.dot.gov/research/publications/technical) addresses critical issues encountered during decision-making on foundation reuse, such as assessment for structural integrity, durability, load-carrying capacity, repair, and strengthening. To highlight significant benefits of foundation reuse, the manual includes numerous case examples.

Integrity Assessment

Existing substructure elements being considered for reuse have been exposed to the environmental elements, and they were not necessarily constructed with quality assurance/control practices consistent with modern code requirements. To redesign existing foundations that are compliant with the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*,[3] there must be confidence in the material properties and the current condition of the substructure.

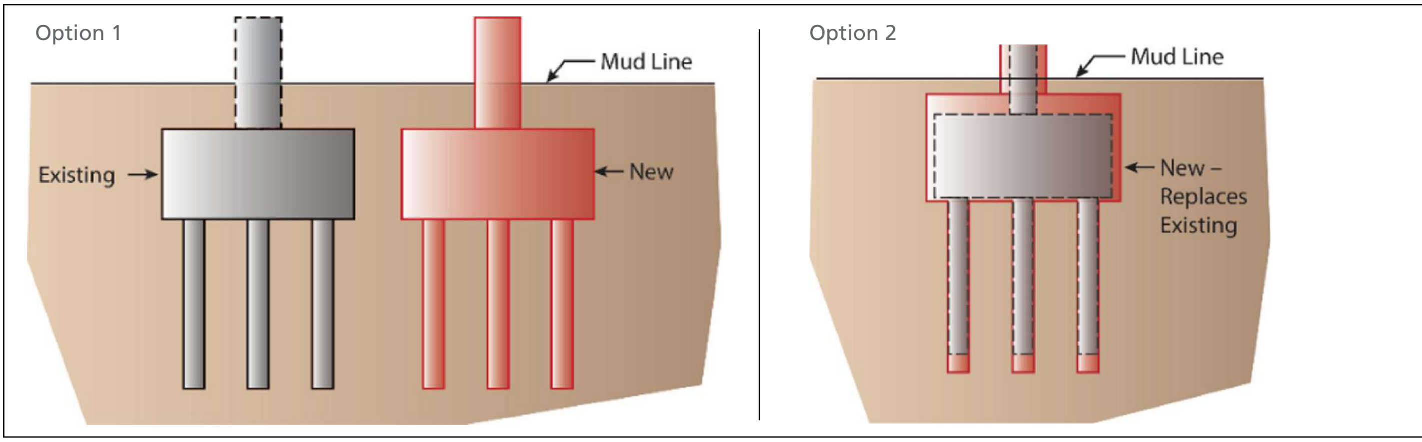
Durability and Remaining Service Life

At the forefront of foundation reuse are the following two questions: First, "How much remaining service life does the foundation have?" Second, "Will the advanced age of the reused components increase life cycle costs?" Some issues, such as chloride ingress or carbonation in concrete, may have reduced the service life remaining without yet creating noticeable damage. Repairs performed on issues identified during the integrity evaluation, such as spalling or delamination, can have service lives, or expected service life costs, unlike those of an intact structure. In many cases, strengthening is employed to simultaneously aid with both durability and capacity issues by repairing existing damage or planning for future deterioration that lowers the capacity of the damaged element.

Capacity Assessment

The overall goal of capacity assessment is to prove that a desired level of capacity exists

Figure 1. Foundation reconstruction options. Figure: Federal Highway Administration.



within the context of current AASHTO LRFD specifications and guidelines from state departments of transportation. The capacity assessment covers scenarios ranging from verifying original design capacity to determining load-and-resistance-factor design capacity for a foundation designed using allowable stress design or load factor design, and determining whether an increased nominal capacity is available (if there is reserve capacity). The capacity assessment would then be useful for determining the extent of strengthening required, if necessary.

Innovative Materials and Foundation Enhancement

The integrity, durability, and capacity assessments provide a list of deficiencies associated with the substructure being proposed for reuse. The selection of foundation-strengthening measures considers the issues identified during analyses to produce an acceptable reuse design. A different approach could be to lower the weight of the superstructure with the usage of lightweight concrete. For example, using lightweight concrete for the bridge deck can lower its weight by 10% or more.

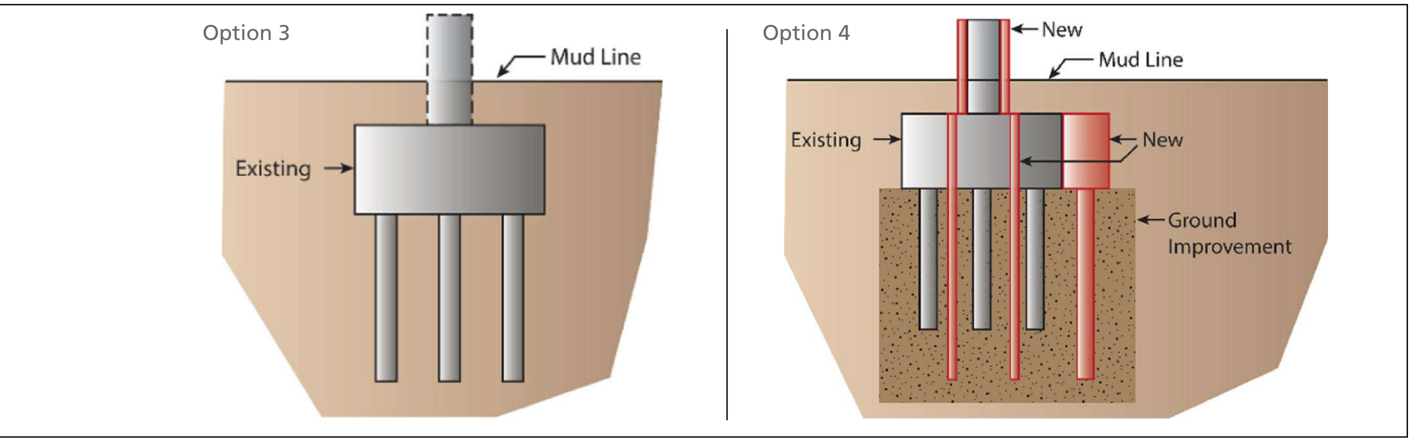
Example: I-95 Replacement

The Interstate 95 (I-95) corridor replacement project replaced 11 aging and deteriorated bridges located near the junction with Interstate 64.[4] These bridges were subjected to high traffic volumes that could not be accommodated through diversion to local roads or other highways.

This ambitious project was a success, in part due to foundation reuse. The dead load on the foundations was reduced by about 7% with the use of lightweight concrete decks in the replacement superstructures. Substructures were repaired by providing corrosion protection for the existing foundations and reused; substructures for 10 of the 11 bridges were reused after installation of cathodic protection, while two required electrochemical chloride extraction. The use of prefabricated bridge elements meant that bridge and lane closures could be limited to weekday nights. The four-year project was completed more than three months ahead of schedule and at a savings of nearly $16 million compared with fully replacing the bridges.

References

1. Federal Highway Administration (FHWA). 2017. National Bridge Inventory: Deficient Bridges by Highway System in 2017. https://www.fhwa.dot.gov/bridge/nbi/no10/defbr17.cfm.
2. Collin, J. G. and F. Jalinoos. 2014. *Foundation Characterization Program: TechBrief #1—Workshop Report on the Reuse of Bridge Foundations.* FHWA-HRT-14-072. Washington, DC: FHWA. https://www.fhwa.dot.gov/publications/research/infrastructure/structures/bridge/fcp/14072/14072.pdf.
3. American Association of State Highway and Transportation Officials (AASHTO). 2017. *AASHTO LRFD Bridge Design Specifications.* 8th ed. Washington, DC: AASHTO.
4. LeGrand, L. 2015. "Keeping Virginia Moving." *Public Roads* 79(1). FHWA-HRT-15-005. https://www.fhwa.dot.gov/publications/publicroads/15julaug/01.cfm.



Vermont

by Robert S. Young, Vermont Agency of Transportation

Vermont is a mostly rural state, and many of its bridges are one-span structures less than 150 ft in length. Because bridge closures can lead to long detours, the Vermont Agency of Transportation (VTrans) often uses accelerated bridge construction (ABC) techniques to minimize traffic interruptions. Additionally, VTrans works with local communities to set construction schedules that accommodate their time frames.

NEXT Projects and ABC

A key component in most of Vermont's concrete bridges is the Northeast Extreme Tee (NEXT) precast concrete beam, which features a top flange that forms the structural bridge deck, eliminating construction steps so bridges can be quickly opened to traffic.

VTrans first used NEXT beams in 2011, for the construction of a 65-ft span over the Williams River on State Road 103 in Chester. With NEXT beams, the bridge opened to traffic in four weeks. The alternative design would have required a temporary bridge and taken approximately nine months to complete. The minimal disruption to traffic proved that NEXT beams provide benefits, and they are now used regularly.

NEXT beams provided an important tool when VTrans established its Accelerated Bridge Program (ABP) in 2012. ABC offers several benefits in the state, such as eliminating environmental concerns related to temporary bridges, limiting the impact of construction on rights-of-ways and utilities, and reducing burdens on detour routes.

Since ABP's inception, VTrans has constructed or rehabilitated the superstructures on 79 bridges, with another three projects underway and 17 in the planning stage. Since 2012, about 50% of VTrans projects have been undertaken via ABP. ABP emphasizes prefabrication of components and the use of incentive/disincentive contract provisions that encourage new techniques and efficiencies. VTrans's Project Initiation and Innovation Team (PIIT), which was developed in conjunction with ABP in 2012, has developed many techniques to create efficient, consistent, and programmatic alternatives for rehabilitating or replacing deteriorated bridges and culverts.

Tropical Storm Irene

In 2011, Tropical Storm Irene damaged or destroyed many Vermont bridges, including some that had not been scheduled for replacement. The creation of ABP and PIIT and a boost in state funding helped return these bridges to service quickly.

During the storm recovery process, Vermont began using a "corridor" construction technique, in which a series of neighboring bridges are replaced at once. One of the first corridors was in Rochester, where Irene destroyed two of the four bridges and damaged another. VTrans's plan to replace all four bridges allowed one contractor to efficiently mobilize crews and materials. Two of the "Rochester Fast 4" bridges used single-span pretensioned concrete NEXT beams with a precast concrete curtain wall, while another featured an open-bottom arch with a cast-in-place (CIP) concrete subfooting and a precast concrete pedestal wall and concrete arch. (See "Rochester Fast 4 on VT 73," which appeared in the Fall 2016 issue of *ASPIRE*®.) VTrans now uses this corridor approach wherever feasible.

Planning and Feedback

A rolling five-year plan helps determine the state's construction schedule (which Irene upended for several years). Over time, VTrans has become more efficient at setting key milestones in project timetables. To this end, VTrans uses scheduling tools to track every element, including environmental permitting and design progress.

VTrans involves community members in every aspect of projects, including scheduling. For example, VTrans consults trucking firms that may be affected by detours and collaborates

In 2011, Vermont Agency of Transportation built a 65-ft-long bridge over the south branch of the Williams River on State Road 103 in Chester, marking the state's first use of Northeast Extreme Tee (NEXT) precast concrete beams. The NEXT beam has since become a standard design option in the state for fast construction. Photo: Vermont Agency of Transportation.

In an effort to replicate the aesthetics of the original Lime Kiln Bridge over the Winooski River Gorge, which was one of the first open-spandrel arch bridges in the United States, designers used precast concrete beams along with cast-in-place concrete arches, piers, floor beams, and deck. Special attention was paid to the visual depth of the arch spans, the depth of intermediate spans, and the post spacing on railings. Photo: VHB-Vanasse Hangen Brustin.

with towns to ensure that, within reasonable limits, schedules and designs are adaptable to their needs. Some towns in high-tourist areas want to avoid construction in the summer, while others want to build in the summer and avoid disruptions during the school year. Balancing those needs and using ABC helps shorten construction periods for all.

VTrans inspects and plans for local bridges under a Town Highway Bridge Program, and local communities are involved at every step. At about the 30% stage of design, town officials are invited to offer input on the plan and schedule. If a road closure and ABC are chosen, the local share in the project is about 50% less than if the community requires a temporary bridge. Officials are updated one year ahead of construction, and, starting about a month before construction begins, signs and announcements are used to keep communities informed.

VTrans also works closely with contractors to address their scheduling needs. Because ABC creates a higher demand for precast concrete elements, contractors have worried that they might not be able to maintain the same, consistent workforce as the one that typically worked with CIP concrete in the summer. In response to their concerns, VTrans now permits contractors to fabricate simple, nonpretensioned precast concrete elements, such as pile caps and approach slabs. They often cast these elements at the project site, which reduces transportation costs and keeps crews busy. The ABC approach also allows contractors to continue working through the winter. By prefabricating components, contractors can start and finish multiple bridge projects in one season. Previously, a conventional project—a CIP concrete bridge with a temporary bridge—might take two construction seasons to complete.

Design-Build Options

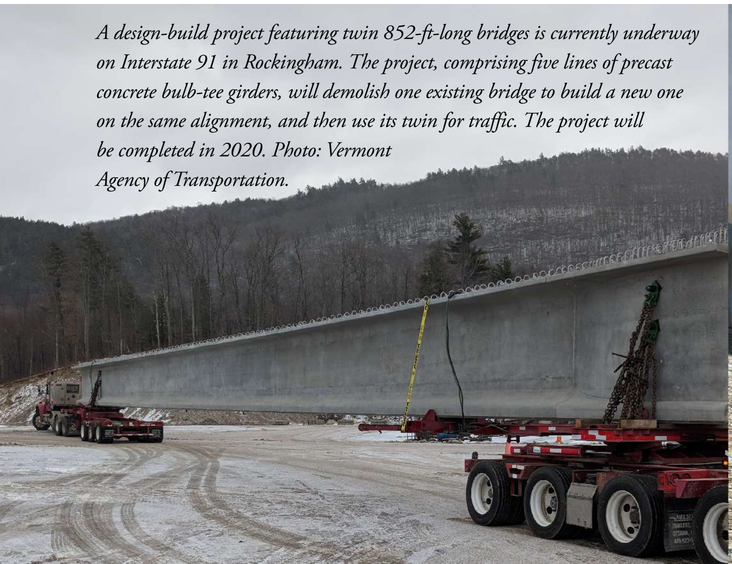
VTrans often works with consultants to develop designs that can be quickly executed. In some cases, a design-build delivery method is used. For example, on Interstate 91 over the Williams River and Green Mountain Railroad in Rockingham, twin narrow, four-span steel-truss bridges in poor condition are being replaced on the same alignment with two four-span structures comprising five lines of precast concrete bulb-tee girders in a 167-245-245-195-ft span configuration. The contract went to the design-build team whose proposal had the best combination of concept and bid price. The 852-ft-long bridges, scheduled for completion in 2020, will be built one at a time and feature two 12-ft-wide travel lanes, a 4-ft-wide left shoulder, and a 10-ft-wide breakdown lane.

An earlier design-build project was the 1036-ft-long Brattleboro Bridge, which consists of three spans of segmental CIP concrete box girders constructed with the balanced cantilever method. This "gateway" bridge over the West River and along the West River Trail features pedestrian viewing platforms and piers with Vermont-inspired stone-formed and stained concrete that blends with the local environment. (The Brattleboro Bridge was featured in the Winter 2018 issue of *ASPIRE*.)

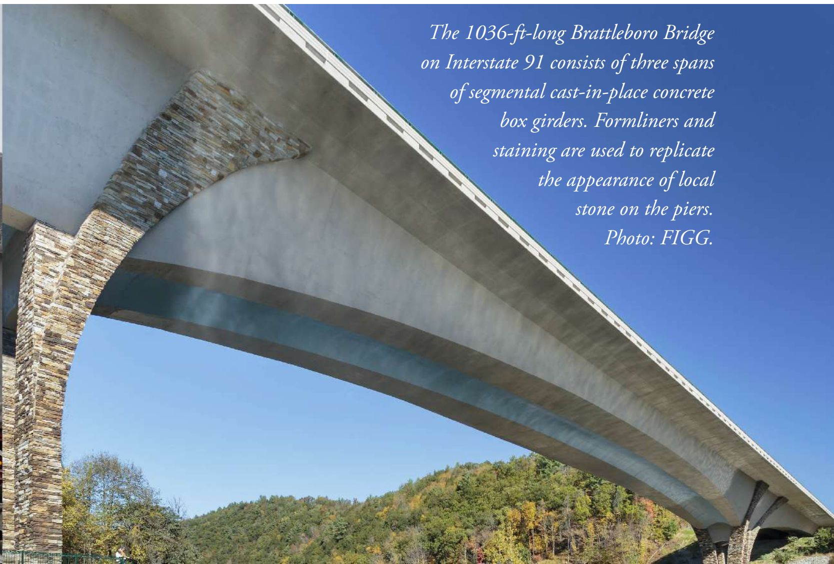
Typical bridges do not feature that level of aesthetic design. However, as older bridges are replaced, VTrans seeks new designs that reflect the original aesthetic concepts. For example, when the Lime Kiln Bridge over the Winooski River Gorge replaced one of last open-spandrel arch bridges in the United States, designers used an open-spandrel arch featuring 80 precast, prestressed concrete beams and CIP concrete arches, piers, floor beams, and deck. The design reflected the aesthetics of the original bridge while upgrading the structure and completing the project with an efficient schedule.

Going forward, Vermont will continue to prioritize techniques to efficiently construct projects and work to ensure communities are on board as projects are developed.

***Robert S. Young** is a project manager with the structures hydraulic section of the Vermont Agency of Transportation in Montpelier, Vt.*



A design-build project featuring twin 852-ft-long bridges is currently underway on Interstate 91 in Rockingham. The project, comprising five lines of precast concrete bulb-tee girders, will demolish one existing bridge to build a new one on the same alignment, and then use its twin for traffic. The project will be completed in 2020. Photo: Vermont Agency of Transportation.



The 1036-ft-long Brattleboro Bridge on Interstate 91 consists of three spans of segmental cast-in-place concrete box girders. Formliners and staining are used to replicate the appearance of local stone on the piers. Photo: FIGG.

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

IN THIS ISSUE

http://www.crab.wa.gov/Funding/Grants/Projects/activeNew.cfm?projectID=R1744
This is a link to the Douglas County, Wash., website about the Chief Joseph Dam Bridge. The construction of the replacement bridge is featured in a Project article on page 16.

http://www.intrans.iastate.edu/research/documents/research-reports/bridge_deck_removal_w_cvr.pdf
This is a direct link to *Methods for Removing Concrete Decks from Bridge Girders,* a 2014 report that reviews advantages and disadvantages of various deck removal methods. Hydrodemolition is a concrete-removal technique highlighted in a Concrete Bridge Preservation article on page 41.

http://www.aspirebridge.com/magazine/2018Winter
This is a link to the *ASPIRE*® Winter 2018 issue with an article on the Interstate 91 Brattleboro Bridge. The bridge is described in the State article featuring Vermont on page 38.

http://www.aspirebridge.com/magazine/2017Spring
This is a link to the *ASPIRE*® Spring 2017 issue with the article "Rochester Fast Four on VT 73." This article illustrates the use of accelerated bridge construction techniques in Vermont. Vermont is the featured state in the article on page 38.

ftp://ftp.dot.state.tx.us/pub/txdot-info/cmd/cserve/standard/bridge/ppbcstd1-17.pdf
This is a direct link to Texas Department of Transportation's standard drawings for prestressed concrete bent caps. These bent caps are the topic of a Concrete Bridge Technology article on page 32.

https://www.fhwa.dot.gov/pavement/pubs/006641.pdf
This is a direct link to *Guide to Nondestructive Testing of Concrete,* a Federal Highway Administration technical report that has information on the maturity technique for estimating strength gain in concrete. The use of the maturity method is discussed in a Concrete Bridge Technology article on page 30.

http://www.aspirebridge.com/magazine/2017Winter
This is a link to the Winter 2017 issue of *ASPIRE*®, which has three articles on inspection and quality of grout or flexible fillers in bridge structures. Quality of structures is the focus of the Editorial on page 2. That issue also features the state of Nevada and contains information on several Nevada Department of Transportation projects, including the Centennial Bowl project in Las Vegas, which is the topic of a Project article on page 12 of this issue.

http://lasvegas.cbslocal.com/2017/07/12/centennial-bowl-opens-wednesday-in-northwest-las-vegas
This is a link to a video of the opening day of the Centennial Bowl Flyover in Las Vegas, Nev. The project is featured in a Project article on page 12.

http://www.asbi-assoc.org/cfcs/cmsIT/baseComponents/fileManagerProxy.cfc?method=GetFile&fileID=688B8860-F51E-0459-FC816AEE9F38F555
This is a direct link to *Design and Construction of Concrete Segmental Bridges,* an American Segmental Bridge Institute publication outlining the history and construction methods of segmental concrete bridges. Segmental concrete bridge design and construction is the topic of the LRFD article on page 44.

OTHER INFORMATION

https://bookstore.transportation.org/collection_detail.aspx?ID=179
This is a link to the American Association of State and Highway Officials (AASHTO) website where one can view the table of contents or purchase the recently published AAASHTO *Manual for Bridge Evaluation*, 3rd edition. The manual has inspection procedures and evaluation practices that meet the National Bridge Inspection Standards.

http://publications.iowa.gov/27040/1/TR-683%20Final%20Report%20Use%20of%20Ultra-High-Performance%20Concrete%20for%20Bridge%20Deck%20Overlays.pdf
This is a direct link to the *Use of Ultra-High-Performance Concrete for Bridge Deck Overlays*, which reviews the application of a thin layer of ultra-high-performance concrete on top of normal concrete bridge decks.

Hydrodemolition:

A Preservation Strategy for Concrete Bridges in the United States

by Edward Liberati, Hydro-Technologies Inc. and Patrick Martens, consultant

Hydrodemolition uses a controlled, high-pressure water jet to safely and selectively remove portions of reinforced concrete from a bridge while leaving the reinforcing steel and surrounding concrete intact. It was developed in Europe in the 1970s as an alternative to jackhammers and has become an acceptable method of removing concrete throughout the world.

In the United States, hydrodemolition is predominantly used to remove concrete from bridge decks in preparation for a new concrete overlay. It can also be used to remove concrete from bridge abutments, piers, walls, and rails and to do full-depth deck removals.

Other uses of hydrodemolition to remove concrete from reinforced concrete structures include tunnels, factories, piers, dams, defense facilities, and even amusement park rides. Logistics, costs, and feasibility are important considerations for the use of hydrodemolition on these special projects.

Department of transportation specifications—such as those in Pennsylvania, Indiana, Kentucky, Ohio, Arkansas, and many more—are valuable resources when considering the use of hydrodemolition.

Hydrodemolition robot working on a bridge deck. All Photos: Edward Liberati.

Equipment and Process Overview

Equipment consists of a hydrodemolition robot, a pump unit, and cleanup equipment. The robot is computerized, self-propelled, and remote-controlled and has many safety features. To meet various project needs, there are hydrodemolition robots designed to cut horizontally, vertically, and overhead.

The hydrodemolition process begins when potable water is delivered from a hydrant or a tanker to the pump unit, where it is pressurized and the flow rate can be controlled. The high-pressure water is then delivered through hoses to the hydrodemolition robot, where it exits a secured nozzle and impacts the concrete surface. The computerized robot controls the movements of the water jet so the stream exposure time on the concrete surface is consistent over the removal area. A steel shell and skirting around the robot's cutting head that houses the water jet allows this operation to be performed safely.

While hydrodemolition work is underway, cleanup of all rubble created from the operation is required. Cleanup of the concrete debris and excess runoff water is performed with water pumps and vacuum-collection equipment. The vacuum-collection equipment can quickly cleanup all water and wet debris remaining on the deck.

As a final method of deck preparation before the placement of the overlay, the contractor performs a final high-pressure water blast and soaks the deck surface with water until it is at a point at which it will not dry out. Once the deck surface is saturated, the contractor covers it with plastic. This covering locks in the moisture, eliminates the need for bonding grouts, and prevents deck contamination from construction equipment.

Fast-Track Hydrodemolition

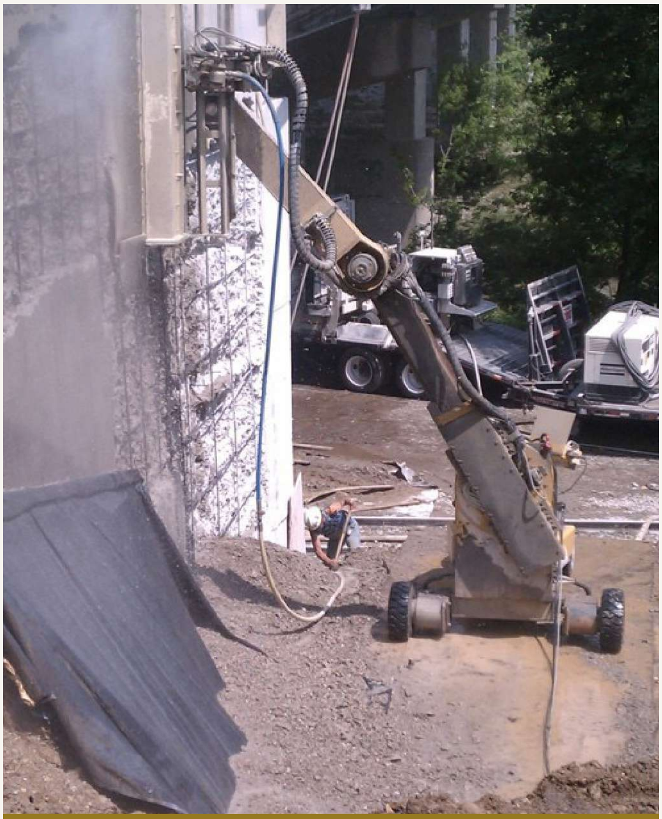
Fast-track hydrodemolition (FTH) is a technique used to prepare a bridge deck for a new latex-modified concrete

Full-depth deck concrete removal. Note the very clean condition of the reinforcement.

Vertical hydrodemolition of a bridge hammer-head pier.

overlay. It is the most commonly used hydrodemolition method in the United States. The FTH approach expedites the work, minimizes lane closure times, and provides a good surface for a high-quality concrete deck overlay that lasts for many years.

This process has numerous advantages for deck rehabilitation and restoration. Upon proper calibration of the robot, the FTH process takes advantage of a concept referred to as *selective removal;* the water jet traverses the deck surface selectively removing only weakened deck concrete in a single pass. The computer-calibrated equipment removes not only large and obvious areas of delamination but also areas with microfractures that are less evident. These microfractures could be the result of corrosion, past jackhammer work, or milling. It is imperative to ensure that no microcracks remain because they could later create a potential for delamination in the underlying deck.



Vertical hydrodemolition of a turnback bridge abutment.

The FTH process also etches the sound concrete that is left in place to provide a roughened and bondable surface for the new latex-modified concrete overlay. The weaker cement matrix and fine aggregates are washed away during the hydrodemolition process, exposing the angular faces of the coarse aggregate that provide mechanical interlock for the overlay. That bond is enhanced by the increased surface area available from the coarse aggregate and the general roughness from the process. With only a sound, roughened concrete deck surface remaining after the FTH process, a monolithic application of the new latex-modified concrete provides a dense structural repair that acts integrally with the remaining deck concrete.

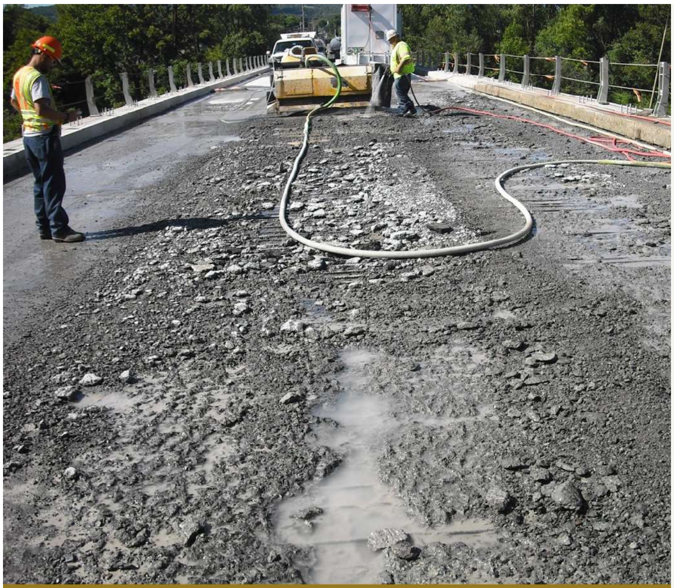
Hydrodemolition Advantages

When compared with jackhammering, hydrodemolition has many advantages. Hydrodemolition does not damage the existing reinforcing steel; in fact, it cleans the steel and removes chloride-ion concentrations. It also will not cause vibrations to the existing bridge deck steel or promote debonding of the otherwise firmly embedded portion of the bars.

With conventional concrete removal techniques using jackhammers, clearance around reinforcing steel must be provided for bonding of the new concrete. In contrast, the use of hydrodemolition, when combined with latex-modified concrete, may allow owners to waive the rebar clearance requirements in their specification, as long as the remaining concrete is found to be sound and no debonding of reinforcing steel with the existing interface is present. Thus, the use of FTH greatly reduces the required amount of concrete material removed from a bridge deck when compared to jackhammering.

Conclusion

In the United States, the use of hydrodemolition to



Hydrodemolition equipment performing fast-track hydrodemolition on a bridge deck.

Fast-track hydrodemolition surface showing selective removal of deteriorated concrete areas. The process provides a clean, etched, and bondable surface ready for an overlay.

selectively remove deteriorated concrete from bridge decks has become a popular technique to minimize the concrete removal required and more quickly address bridge deck needs. The success of the system has been validated through the thousands of installations across the United States. When combined with latex-modified concrete, the FTH surface preparation has the potential to deliver upward of 30 years of added service life to the bridge deck. Transportation agencies are finding that FTH is an efficient way to rehabilitate and preserve bridge decks quickly and cost effectively, while minimizing traffic disruptions.

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Segmental Bridges: Guide Specifications and AASHTO LRFD Bridge Design Specifications

by Dr. Oguzhan Bayrak, University of Texas at Austin

Concrete bridges constructed in short segments—also known as segmental bridges—are the subject of this article, which reviews some basic concepts and definitions, as well as the history of segmental concrete bridge design and construction, before turning to a discussion of specifications for this type of bridge.

Segmental concrete bridges can be constructed by either assembling precast concrete segments or casting the segments in place. The balanced cantilever technique can be used in both precast and cast-in-place concrete construction to minimize the unbalanced moments induced in the towers or pylons. Alternatively, in cases where environmental conditions or construction-related constraints necessitate the use of a top-down construction technique, span-by-span erection of the segments may be preferred.

Early examples of segmental concrete bridges can be found in Europe. To the best of my knowledge, such bridges were constructed in Germany and France in the 1950s and 1960s. Soon after segmental bridges were successfully designed and constructed in Europe, they were introduced in the U.S. as viable alternatives to long-span bridges. The first example of a long-span, segmental concrete bridge in the United States was built in the state of Texas over the Gulf Intracoastal Waterway in 1973. Since then, many cast-in-place and precast concrete segmental bridges have been built across the nation and are in service today.

Precast segmental concrete box girder for U.S. Route 183 elevated viaduct, Austin, Tex. Photo: Oguzhan Bayrak.

During the early days, as the technology of segmental concrete bridge design and construction was being transferred to the United States, first principles, fundamentals of structural engineering, and the concrete bridge design provisions of that time were employed in designing and constructing such bridges. This "technology transfer" period helped the bridge engineering profession identify areas that required research and additional discussion.

With the conclusion of early research efforts and successful completion of the first few segmental bridge projects, the profession developed the *Guide Specifications for Design and Construction of Segmental Concrete Bridges*.[1] It is important to appreciate the fact that American Association of State Highway and Transportation Officials (AASHTO) guide specifications, including the one that applies to the design of segmental concrete bridges, are commonly used for new technologies and emerging materials or design techniques and therefore do not carry the weight of conventional AASHTO specifications. The first edition of the AASHTO *Guide Specifications for Design and Construction of Segmental Concrete Bridges*,[1] published in 1989, was assembled in an environment in which very little additional formal guidance existed, except documentation from the prior European experience and early research done by the Texas Department of Transportation in conjunction with the University of Texas. The second edition,[2] published in 1999, reflects 10 years of further U.S. experience during which many segmental bridges were designed and constructed.

In the 8th edition of *AASHTO LRFD Bridge Design Specifications*,[3] design provisions that apply to segmental concrete bridges are covered in Article 5.12.5. The requirements listed within that section of the specifications are aimed at supplementing design requirements that apply to all concrete bridges. Separate treatment of segmental concrete bridges in Article 5.12.5 is necessitated by the segmental construction method, stages, and temporary support conditions during bridge construction. Article 5.12.5 covers "construction by free cantilever, span by span, or incremental launching methods using either precast or cast in place concrete segments which are connected to produce either continuous or simple spans." Although spliced-girder bridges share some characteristics with segmental bridges, their design is covered in Article 5.12.3.4 because, in many ways, they are closer to conventional concrete girder bridges. Thus, the reorganized Section 5 of the 8th edition of AASHTO LRFD specifications streamlines the design provisions applicable to segmental concrete bridges and provides a valuable resource in designing tomorrow's segmental bridges. With that stated, first principles will always remain primary method of choice for designing and constructing segmental concrete bridges.

References

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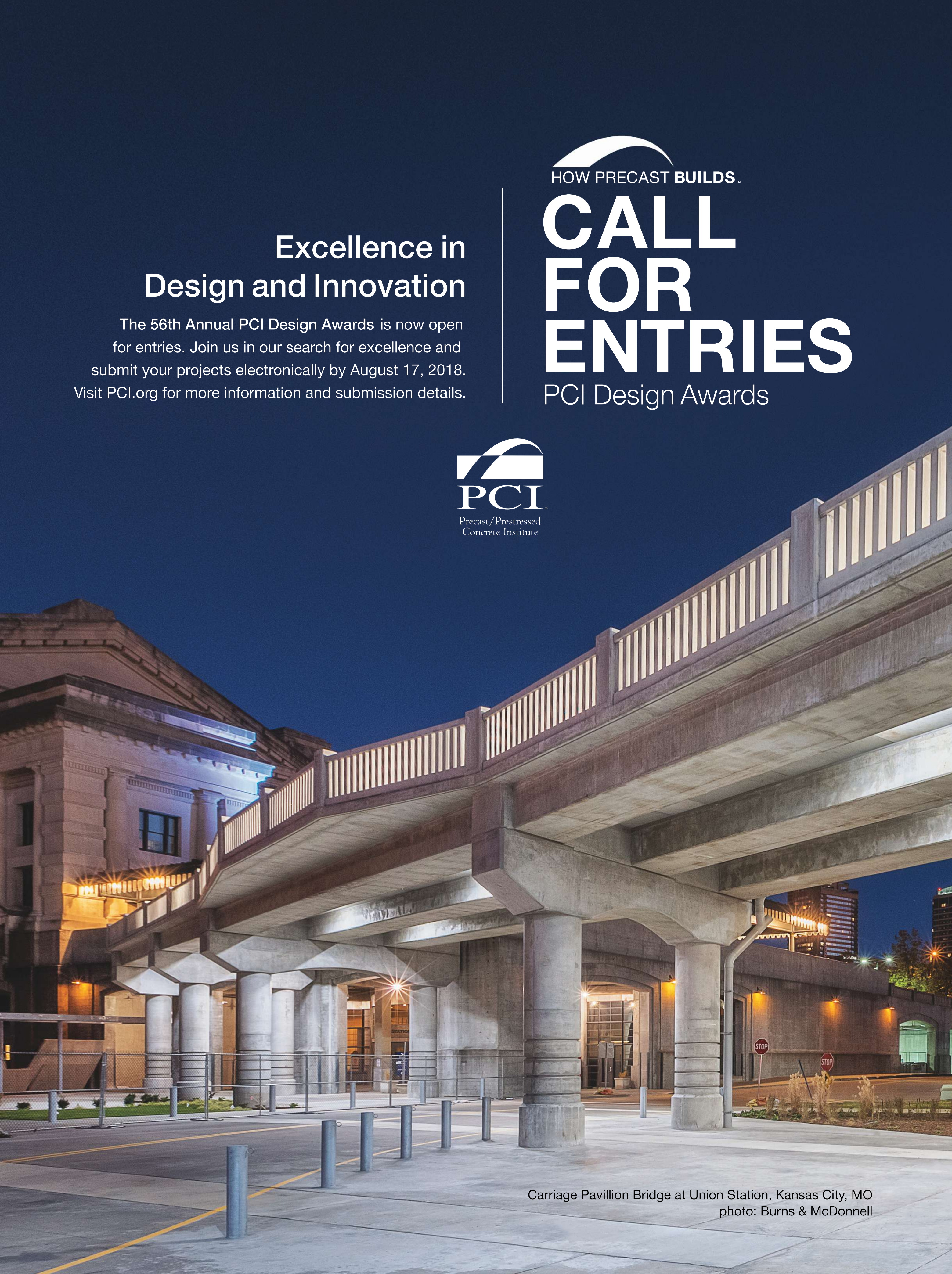

Carriage Pavillion Bridge at Union Station, Kansas City, MO
photo: Burns & McDonnell