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THE CONCRETE BRIDGE MAGAZINE
WINTER 2021
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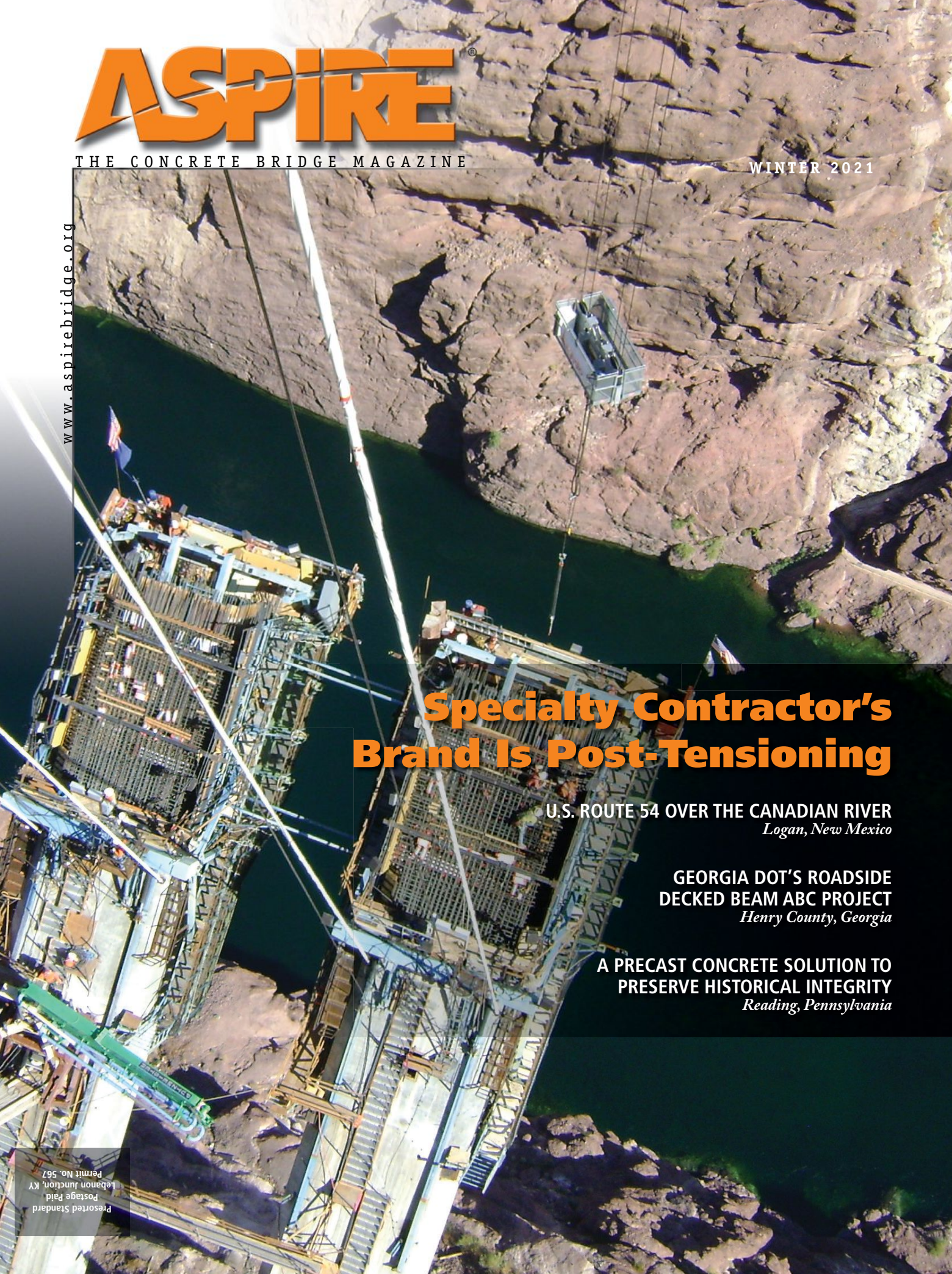

Specialty Contractor's Brand Is Post-Tensioning

U.S. ROUTE 54 OVER THE CANADIAN RIVER
Logan, New Mexico

GEORGIA DOT'S ROADSIDE DECKED BEAM ABC PROJECT
Henry County, Georgia

A PRECAST CONCRETE SOLUTION TO PRESERVE HISTORICAL INTEGRITY
Reading, Pennsylvania

Presorted Standard
Postage Paid
Lebanon Junction, KY
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Courses on Design and Fabrication of Precast, Prestressed Concrete Bridge Beams

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

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

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

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

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Preliminary Precast, Prestressed Concrete Design (T110)

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Extending Spans (T310)

Bearing Pads (T450) and (T455)

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

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Stability Calculations and Sensitivity Analysis (T527)

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AASHTO THE VOICE OF TRANSPORTATION

PCI Precast/Prestressed Concrete Institute

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EDITORIAL

Photo: PCI.

Allowed but Appalling—The Importance of Maintaining High Principles

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

For readers new to *ASPIRE*®, The Concrete Bridge Magazine, welcome to the best concrete bridge periodical for the engineering professional. Okay, I'm a little—no, I'd better make that very—biased. To the tried-and-true returning readers, thanks for returning, and enjoy this issue.

When I began this journey as the Editor-in-Chief of *ASPIRE*, my goal was to highlight innovative concrete bridge and precast concrete technologies. The idea of sharing new, exciting, and emerging solutions, with the goal of advancing our profession to meet future challenges and demands, is what drives me as a civil engineer.

Along this journey, I've shared stories of triumph and failure, and on occasion, a saying or two, picked up from time to time, from colleagues across a wide range of professions. I ran across one of those sayings a few weeks ago, and it resonated with me. I thought I'd share it, as it seems to fit several topics that we've highlighted in this past year's issues of *ASPIRE*.

"Allowed but appalling." This phrase came up in a conversation with an associate, who shared a story about how her work at an upper level of state government required a great deal of finesse and more than a few negotiated solutions. She described a one-line press release that came to her for review; it met all public information requirements but the messaging to the community was appalling. The additional time, research, review, and editing that were required to prepare an appropriate communiqué delayed the start of the project but were absolutely necessary to get the project off on the right foot.

How does "allowed but appalling" fit into your business practice? Is it a personal or company practice that, while legitimate, does not show you or your organization in the best light or circumstance? Is the engineering solution you selected code compliant but, in the long run, not the best idea for your project, when viewed by the end user or owner? Is the public information officer advising the state highway engineer, "Technically you are correct, but your messaging is tactless"? It absolutely may be "allowed," but, ugh, it's just plain "appalling."

As we head into 2021, the challenges we face do not seem to be taking the year off; in fact, I'd argue that they're picking up the pace. How will the projected and realized gas tax revenues impact our efforts? Toss in a tablespoon of COVID-19, mix with seemingly limitless extreme environmental events (think record wildfires in California and a historic hurricane season—which ran out of names!), add a pinch or two of societal stressors, all while observing a slowing of the traditional project workflow. While it's clear that 2021 will challenge our community at every turn, our call as engineering professionals is to meet these demands by pushing the boundaries of innovation while still maintaining the high principles of our profession. We must get back in front, outpace the norm, and develop the engineering solutions for the future we now find ourselves in.

For this issue, the *ASPIRE* team has selected another full slate of practical technical topics, from structural fundamentals that need rekindling to innovative concrete and reinforcing materials, as well as the continuing Perspective series to promote discussion on our bridge community's duties and workflow.

Buckle up!

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Schwager Davis supplied the temporary stay-cable system and post-tensioning materials for the iconic Hoover Dam Bypass Bridge whose twin arches meet at midspan over the Colorado River.

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If you have a suggestion for a project or topic to be considered for *ASPIRE*, please send an email to info@aspirebridge.org

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CONCRETE CALENDAR FOR WINTER 2021

The events, dates, and locations listed were accurate at the time of publication but may change as local guidelines for gatherings continue to evolve. Please check the website of the sponsoring organization.

January 5–29, 2021
100th Transportation Research Board Annual Meeting
Virtual Event

January 12–15, 2021
International Symposium on Pavement, Roadway, and Bridge Life Cycle Assessment
Virtual Event

February 4 or 5, 2021
PTI Level 1 & 2 Multistrand and Grouted PT Specialist Workshop
Miami, Fla.

March 28–April 1, 2021
ACI Spring 2021 Convention
Virtual Event

April 2021
ASBI Grouting Certification Training Course
On-demand webinar

April 18–21, 2021
PTI 2021 Convention & Expo
Westin Indianapolis
Indianapolis, Ind.

Week of April 26, 2021
PTI Level 1 & 2 Multistrand and Grouted PT Specialist Workshop
Seattle, Wash.

May 18–22, 2021
PCI Convention with the Precast Show and National Bridge Convention
Ernest N. Morial Convention Center
New Orleans, La.

June 7–8, 2021
ASBI 2021 Construction Practices Seminar
Marriott Seattle Airport
Seattle, Wash.

June 7–10, 2021
World of Concrete
Las Vegas Convention Center
Las Vegas, Nev.

June 7–10, 2021
2021 International Bridge Conference
Gaylord National Resort & Convention Center
National Harbor, Md.

July 11–15, 2021
AASHTO Committee on Bridges and Structures Annual Meeting
Indianapolis, Ind.

July 19–22, 2021
Bridge Engineering Institute Conference 2021
Singapore

August 1–5, 2021
AASHTO Committee on Materials and Pavements Annual Meeting
Location TBD

September 22–25, 2021
PCI Committee Days and Technical Conference
Loews Chicago O'Hare Hotel
Rosemont, Ill.

November 8–10, 2021
ASBI 33rd Annual Convention and Committee Meetings
Westin La Paloma Resort and Spa
Tucson, Ariz.

Reader Response:

The *ASPIRE* team received a reader comment on the 3-part series on sweep by Dr. Bruce Russell that appeared in the Spring 2019, Fall 2019, and Summer 2020 issues. The comments and editor's response are summarized below.

The reader felt that the two most important points were not addressed:

1. If a product is out of tolerance, the piece should be considered defective until the cause of the sweep is determined.
2. An out-of-tolerance piece due to unknown causes should not be incorporated because it may have an unknown deleterious effect on the structure's performance.

Editor's Response:

The author addressed the first point in the first article, indicating that the fabricator must investigate any unusual sweep. From this evaluation, reasonable strategies can generally be developed to minimize or mitigate sweep. The author addressed the second point in subsequent articles.

Overwhelming experience from years of production of pretensioned girders is that sweep can almost always be accommodated or corrected, and that most girders with sweep can be used successfully. The methods and procedures used to manufacture pretensioned girders make it unlikely that an extreme sweep could occur that would require rejection.

Successfully Pumping Lightweight Concrete for Bridges

Pumping is often the preferred method used by contractors to place concrete for bridges, and especially decks. But some contractors and engineers are reluctant to pump lightweight concrete for bridges, thinking that it is not practical. However, there are many examples of projects where pumping has been successfully used for bridges, and even for many buildings over 500 ft tall.

To successfully pump lightweight concrete, the same considerations should be followed that are used for normal weight concrete, with one significant addition: the lightweight aggregate must be prewetted before batching because the porous lightweight aggregate has a higher absorption than normal weight aggregate. Prewetting prevents water from the mixture being driven into the lightweight aggregate during pumping, which would cause slump loss and difficulties in pumping. For more information, see the ESCSI publication *Go with the Flow – Pumping Structural Lightweight Concrete*, which is available on www.escsi.org.

An article in the Fall 2019 issue of *ASPIRE* discusses the construction of the Shasta Viaduct Arch Bridge in California, where lightweight concrete was pumped vertically 100 ft for bent caps and the box girder superstructure. This project demonstrates that that lightweight concrete can be successfully pumped for bridges at rates from 50 to 100 yd^3/hr.

Photo Credit: R. Maggenti

www.escsi.org

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Rotary Kiln Structural Lightweight Aggregate

Specialty Contractor's Brand *Is* Post-Tensioning

Family-owned Schwager Davis Inc. thrives on the toughest projects and energizes the post-tensioning industry with their creative approach

by Monica Schultes

The post-tensioning (PT) industry has had a major role in the development of long-span concrete bridge construction in North America. One firm stands out in the continued growth and development of the industry: specialty contractor Schwager Davis Inc. (SDI).

SDI's reputation stems from founder Guido Schwager, who loved a challenge and pursued the most difficult jobs while also devising new techniques and materials.

With the changes in seismic requirements brought about by the 1989 and 1994 earthquakes in California, the need for PT grew there and across the country. SDI thrived as well. Although the majority of their projects are on the West Coast, "we follow the work wherever that might be," explains Marcus Schwager, who handles business development at SDI.

Investing in Research and Development

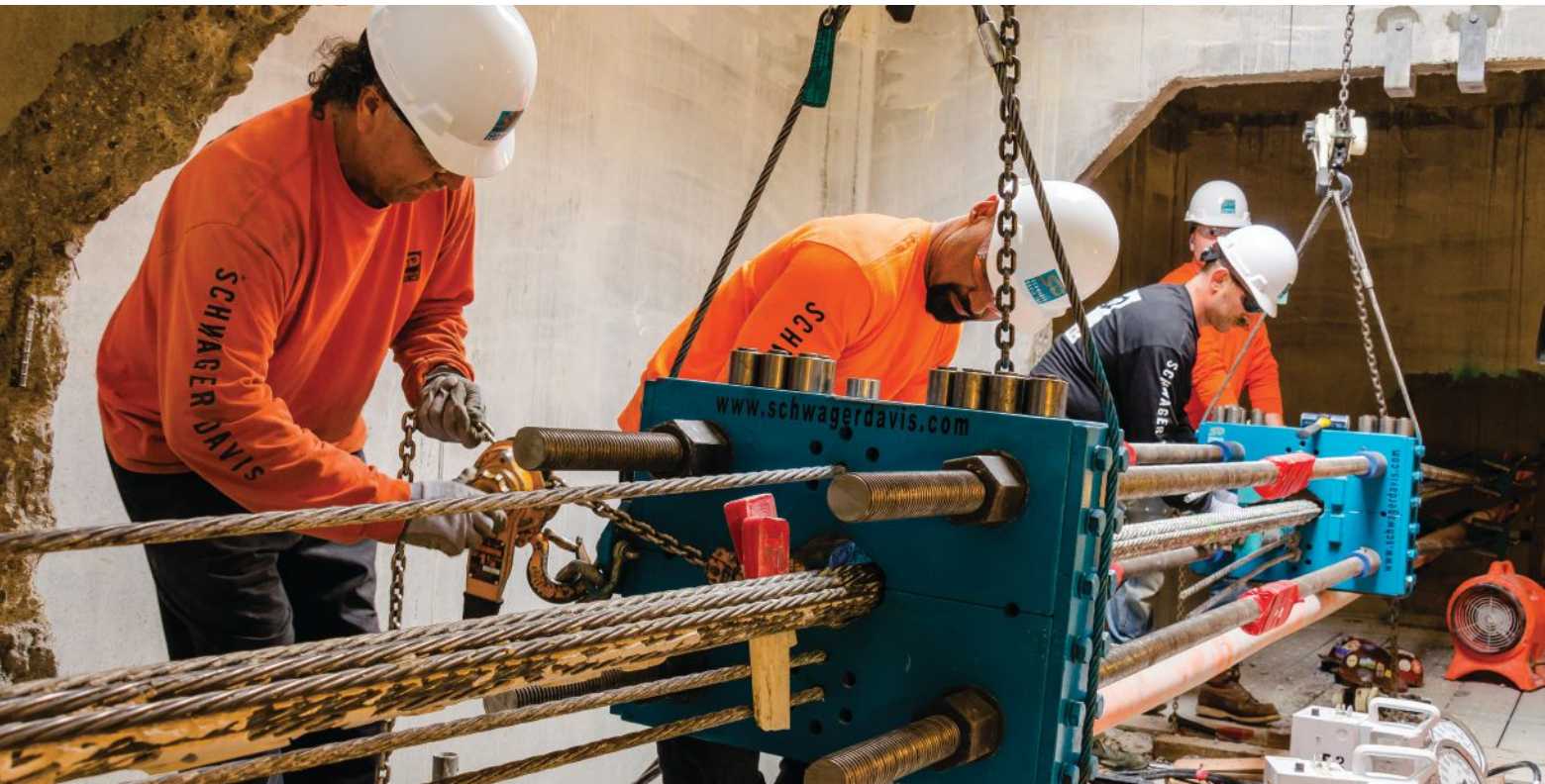
Great companies invest in innovation. SDI invests a considerable amount in research and development. "That gives us an edge over other specialty contractors," says Mike Schwager, president of SDI. "While we don't designate a specific target, we spend as much as 25% of our annual profit researching, developing, and testing new techniques and equipment."

SDI makes substantial investments in finding solutions for challenging projects. Mike Schwager explains, "We are always trying to unravel a new knot or increase the efficiency of our current methods." Making improvements helps SDI keep the focus on the safety of their labor force and the integrity of the structure.

The SDI Difference

Active in technical associations like the Post-Tensioning Institute (PTI) and the American Segmental Bridge Institute (ASBI), SDI engineers participate in developing best practices for the entire PT industry. "It is good for our people, our firm, and helps move the industry forward," suggests Mike Schwager. Since

Because tendons could not be safely cut in the typical fashion, Schwager Davis developed a new procedure for use on the State Route 520 Evergreen Point Floating Bridge Project in Seattle, Wash. After months of troubleshooting and testing, the firm's crews carefully exposed the strands in a tendon near midspan and then detensioned one strand at a time. After detensioning, the strand was anchored at the temporary plates and retensioned. After all strands were tensioned and anchored to the temporary plates, the entire tendon could be detensioned in stages. All Photos: Schwager Davis Inc.



On the Hoover Dam Bypass Bridge, one of the largest concrete arch bridges in the world, Schwager Davis supplied the temporary stay-cable system and post-tensioning materials for this iconic bridge's twin arches that meet at midspan over the Colorado River. The firm's crews worked post-tensioning two temporary towers on both sides of the bridge. The peak of these towers served as the location for cables to be attached.

Guido Schwager established SDI nearly 25 years ago, the PT industry has seen significant improvements in techniques and materials such as better quality control; stronger corrosion protection; improved anchorages and injection ports, caps, ducts, grouts and grout caps; and installation advancements.

Looking ahead, Mario Salice, vice president of the post-tensioning division, predicts that the industry will see more electrically isolated tendons and monitoring systems (see the article on electrically isolated tendon technology in the Spring 2019 issue of *ASPIRE*®). The PT industry is developing standards to encourage their use. "I think these methods will help us monitor and foresee any issues. As a PT supplier, we want to preserve the simple and cost-effective system that we believe in, but there is always room for improvement."

"As a PT supplier, we want to preserve the simple and cost-effective system that we believe in, but there is always room for improvement."

Many public projects are design-bid-build, which makes it difficult for SDI to modify design requirements and bring the latest technology to a project. They turn to industry association committees to update specifications to reflect the most current methodology, so that bridge engineers have the latest information.

"While we price our services competitively, there are occasions where contractors self-perform the PT work. It makes sense on small cantilever projects where construction sequencing prohibits us being on the site all the time, or where the PT work is not continuous," explains Salice.

The company culture is to be as efficient as possible. "Whether it is a stay-cable, segmental, or standard cast-in-place box-girder bridge, safety, quality, and service are the core values we insist on maintaining," Salice says.

"Whether it is a stay-cable, segmental, or standard cast-in-place box-girder bridge, safety, quality, and service are the core values we insist on maintaining."

Duct Work

The company is constantly looking for ways to move PT technology forward. For example, they manufacture their own plastic duct. "It might not seem like a big deal, but our projects often have long runs and we needed to demonstrate effective grout pumping without voids. There were limits to what was available commercially," says Mike Schwager.

According to Salice, not all agencies on the West Coast are using plastic ducts. In his opinion, plastic ducts have brought an additional level of protection in other regions of the country, especially in harsh environments. "We tout the longevity benefits of using it and promote its use to owners," Salice explains.

SDI's signature fusion duct hardware details were incorporated into the longitudinal PT tendons for the Honolulu Rail Transit Project in Hawaii, with over 7000 connections installed. (Segments 1 and 2 of this project were featured in a Project article in the Summer 2015 issue of *ASPIRE*.) The elevated guideway carries the Honolulu commuter rail system from Kualakai to the east end of the island (20 total miles of dual-track structures and 21 stations).

SDI also fabricates their own PT couplers, which is critical for projects with long duct runs. "Our assembly is easy for the ironworkers to assemble. It makes installation simpler and virtually foolproof," describes Mike Schwager. That is an improvement over other versions that require multiple steps, any of which could cause failure or grout leakage.

Precast concrete box-girder segments on the new Marc Basnight Bridge in North Carolina's Outer Banks incorporated SDI's anchorage systems and required

nearly 6000 of their segmental duct couplers. The North Carolina Department of Transportation specified the use of stainless steel post-tensioning materials up to 12 ft above the water and stainless steel reinforcement in all cast-in-place concrete for this project. SDI supplied PT materials and equipment as well as technical assistance for the precast concrete segmental section of the project (see the Project article in the Fall 2019 issue of *ASPIRE*).

Talent Scout

Headquartered in San Jose, Calif., SDI employs up to 150 people, including engineers, project managers, ironworkers, and support teams. That number can fluctuate depending on the local repair projects they take on, as well as project backlog. SDI is committed to "work safely, deliver quality, and provide excellent service. That is of paramount importance to us," says Marcus Schwager.

"Work safely, deliver quality, and provide excellent service. That is of paramount importance to us."

The Bay Area's high cost of living due to its booming technology industry puts a strain on finding talent in the engineering and construction industry. According to Mike Schwager, "The number one challenge that we have here is finding and keeping young talent. The draw of start-ups and tech companies is very strong and skims the pool of employees, making it difficult to compete with their entry-level compensation."

While labor shortages are felt across the country, PT is very technical, making those highly skilled workers even more difficult to find. However, the company's reputation for seeking the most complicated and stimulating projects helps SDI attract and retain those who are passionate about their industry.

Evergreen Point Floating Bridge

"We have done 34 segmental bridge projects and over 1000 PT structures, which is our bread and butter," says Mike Schwager. "Almost every project features a challenging or difficult aspect, but the Evergreen SR 520 project was our most memorable."

The Evergreen Point Floating Bridge in Seattle, Wash., was built in 1963 at a staggering 7580 ft long and underwent a PT seismic retrofit in 1997 that added both grouted and external tendons that were each 3600 ft long. (The low-rise portion of this bridge was the subject of a Project article in the Spring 2016 issue of *ASPIRE*.)

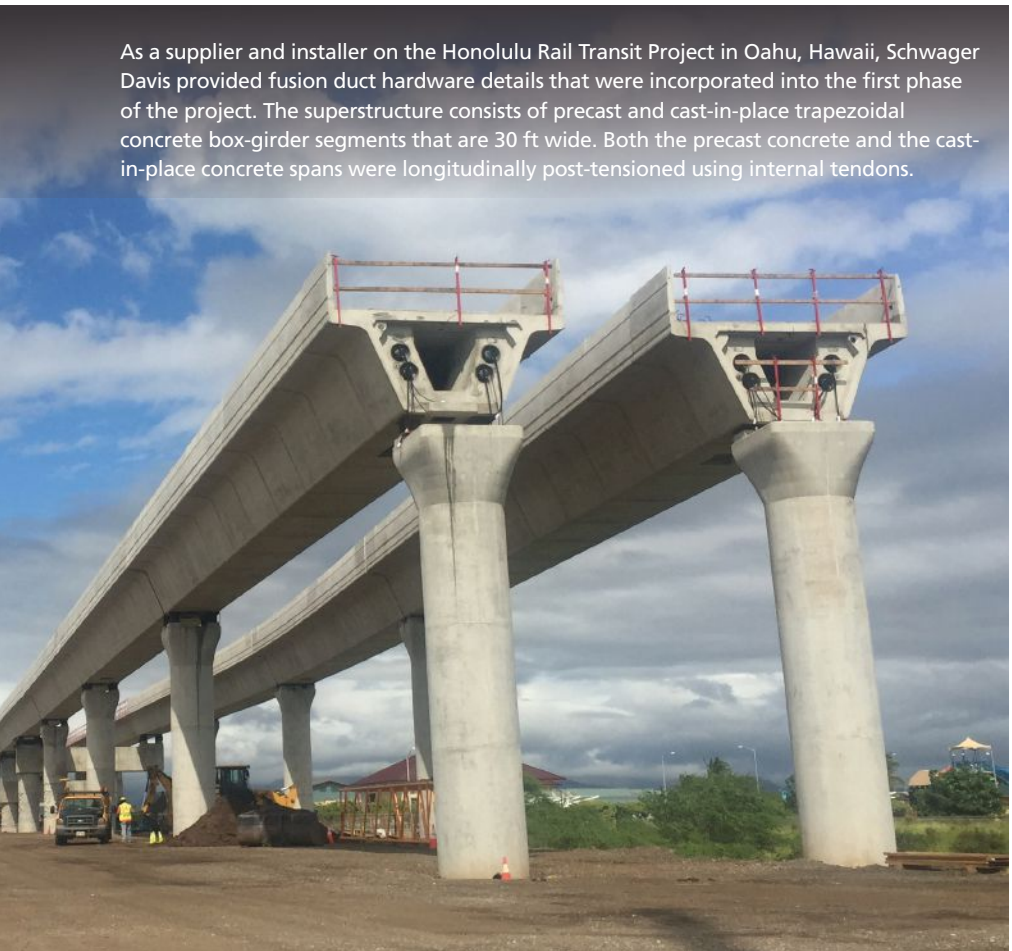
When the Washington State Department of Transportation decided to build a new structure to withstand a 100-year storm, engineers were faced with the challenge of how to safely detension the retrofitted tendons while protecting the old pontoons for use as floating docks. Guido Schwager describes the Evergreen Point Floating Bridge as the "most complicated PT challenge of my career." Even after decades of experience on countless PT projects, this project presented a conundrum.

"Some of the early ideas for detensioning were not safe or feasible," recalls Guido Schwager. Traditional methods could release forces that might injure workers or possibly sink the bridge pontoons. Guido Schwager and his team at SDI considered and designed alternative solutions despite no assurances that they would be awarded the project. After months of troubleshooting and testing, SDI resolved to reanchor and retension the strands transferring loads until they could safely be detensioned hydraulically one stroke at a time, releasing 24 ft of elongation in each 15-strand tendon. Ultimately, SDI was awarded the contract and began work in 2016.

"That problem-solving exercise is a great example of how much we love a challenge. In this industry, you can't repeat the same approach. That helps keep our crews sharp and engaged from job to job and year to year," describes Guido Schwager.

Homer M. Hadley Floating Bridge Retrofit

Accustomed to technical challenges and with their previous experience in floating bridges, SDI was consulted for the retrofit of the Homer M. Hadley Memorial Bridge on Interstate 90 (I-90) in Washington state, which is one of the longest floating bridges in the world. Westbound traffic from Mercer Island to downtown Seattle travels on the northern lanes of the



As a supplier and installer on the Honolulu Rail Transit Project in Oahu, Hawaii, Schwager Davis provided fusion duct hardware details that were incorporated into the first phase of the project. The superstructure consists of precast and cast-in-place trapezoidal concrete box-girder segments that are 30 ft wide. Both the precast concrete and the cast-in-place concrete spans were longitudinally post-tensioned using internal tendons.

Schwager Davis crews worked high above the Pacific Ocean on the Pitkins Curve Bridge on the Pacific Coast Highway (State Highway 1) in Monterey County, Calif. Schwager Davis served as form traveler and post-tensioning supplier on the project.

bridge, while the southern lanes have been designated to carry the Sound Transit East Link light rail extension. A companion span carries eastbound vehicular traffic.

Sound Transit agreed to add longitudinal post-tensioning to the pontoons to increase the load-carrying capacity of the bridge by minimizing movement during adverse weather. At approximately 3600 ft long, the tendons are some of the longest PT tendons in the world.

To add to the construction challenges, the work included fabricating, delivering, and installing 1800 pieces of anchor frame steel collectively weighing 41,000 lb into the confined spaces of the pontoons and hand transporting them to each end for the erection of 20 anchor frames. The pontoons were cored and 20 tendons containing a total of over 1 million ft of strand were installed through the pontoons to each anchor frame. "In my opinion this project solidified our reputation as excelling at these challenging projects," recalls Salice.

Technical Support

From one high-profile and challenging project to the next, SDI has changed the landscape of the country. Employees point to projects like the Pitkins Curve Bridge in Monterey County, Calif. SDI takes pride in playing their part to keep the Big Sur coastal highway accessible. Their crews were frequently seen on the job high above the Pacific Ocean, as SDI served as form traveler and PT supplier and installer.

As an example of their teamwork, SDI frequently consults with engineers and contractors to plan erection sequencing. Such was the case at the Hoover Dam Bypass Bridge, the longest concrete arch span in North America (see the Project article in the Spring 2010 issue of *ASPIRE*). Involved from the early planning stages, SDI supplied the temporary stay-cable system as well as all PT materials and technical support.

Future Generations

Guido Schwager is deeply invested in the family business that he created, and he passes on that reputation and philosophy. While it would be difficult to match his level of passion about work, SDI has excelled at finding like-minded colleagues. Like many family-owned companies, the passion of one spreads to many. "Our challenge is to continue to grow and diversify but stay true to our core values," says Salice.

Mike Schwager took the reins as president of SDI in 2018. "Guido's humble, passionate, and insightful leadership built this company and prepared me and our team to forge on to the next chapter of our company's life. I am thankful to still be working with him, I am proud to further what he so brilliantly began, and I am honored to lead the excellent engineering and construction team that SDI is today."

History and Growth of SDI

Guido Schwager, founder and chairman of Schwager Davis Inc. (SDI), grew up in Switzerland and studied engineering. After immigrating to the United States, he joined VSL, where he quickly rose to CEO of the company's USA West Division. He joined Baker Davis in 1993 and quickly transformed the tiny construction operation into one of the nation's leading specialty contractors. The small, local firm ultimately became Schwager Davis Inc. (SDI). It was clear that Guido wanted to grow the business and enter more challenging markets. He found ways to improve a design, implement an unexpected solution, and even keep a historic building from demolition. He played a key role saving buildings and shoring up local infrastructure after the 1989 Loma Prieta earthquake in California.

Guido Schwager's career of masterful engineering and business integrity provided the reputation for SDI to take on projects of dramatically increased scale and scope in a relatively short time frame. Whether the project involved post-tensioning installation or repair, bridge girders, stay cables, people movers, form travelers, parking structures, nuclear energy, or natural energy projects, he made a place for SDI at the table of regional and national infrastructure.

Guido Schwager was recently recognized by the Post-Tensioning Institute as a "Legend of Post-Tensioning": an individual who has made a significant, long-term contribution to the development of the post-tensioning industry in North America.

PERSPECTIVE

Evolution of the Caltrans *Seismic Design Criteria*

by Mark Yashinsky and Tom Ostrom, California Department of Transportation

Figure 1. Bridge damage from the 1971 San Fernando earthquake. All Photos and Figures: California Department of Transportation.

California's bridge inventory is mainly composed of cast-in-place and precast concrete structures. Of the 25,000 state and local agency bridges in California, about 88% are made of concrete. California's bridge inventory can be categorized by expected seismic performance. "Important bridges" are expected to provide immediate access to emergency facilities after an earthquake. "Recovery bridges" serve as vital links for rebuilding damaged areas shortly after earthquakes and should be available for public use a few days after a seismic event. All other bridges are designated as "ordinary bridges," which comprise 99% of California's bridge inventory.

Three damaging earthquakes, the 1971 San Fernando earthquake, the 1989 Loma Prieta earthquake, and the 1994 Northridge earthquake, were the impetus for California's bridge seismic safety program. The San Fernando earthquake caused the collapse of five bridges (**Fig. 1**) and the death of two people. The Loma Prieta earthquake caused the collapse of one span of the San Francisco–Oakland Bay Bridge and a one-mile segment of the Cypress Street Viaduct in Oakland, resulting in the death of 43 people. The Northridge earthquake resulted in the collapse of seven bridges and one death. These damaging earthquakes spurred the California Department of Transportation (Caltrans) to focus on life safety by minimizing the probability of a bridge collapse during earthquakes.

The modern history of seismic design began after the 1933 Long Beach earthquake and the passing of the Field Act, which required California to design schools for seismic hazards. California's Division of Highways (now Caltrans) followed the state's lead by requiring bridges to be designed for a horizontal coefficient of 0.06 times the weight of the structure. At the time, California was the only state addressing seismic hazards.

In the aftermath of the 1971 San Fernando earthquake, engineers began increasing bridge column strength to resist ground shaking hazards. However, they soon discovered that adding strength increased the bridge's stiffness, which in turn increased the seismic demands on the bridge. **Figure 2** illustrates how the initial stiffness determines the structure's period and the load it attracts during strong ground shaking.

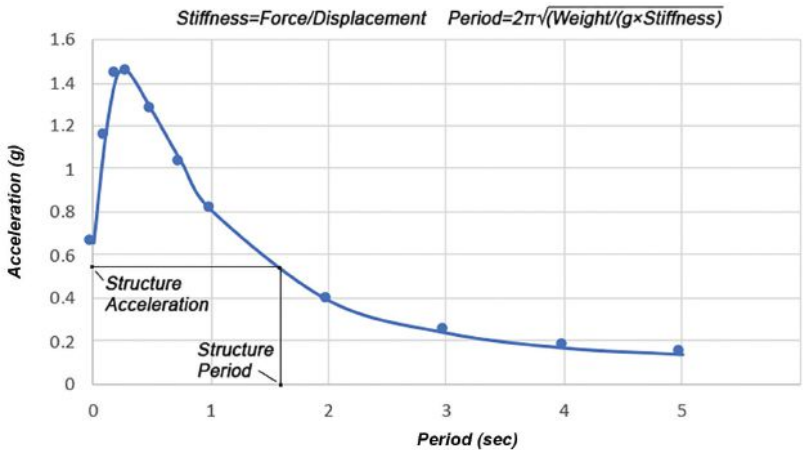


Figure 2. Typical Caltrans response spectrum for bridge design. The initial stiffness of a structure determines its period and the load it attracts during strong ground shaking.

In the mid-1970s, Caltrans began adopting the principles of ductile design of reinforced concrete columns developed by John Blume—cofounder of Earthquake Engineering Research Institute and recognized as the "father of earthquake engineering"—and capacity-protected design, in which the members adjacent to the ductile columns are designed to be stronger than the columns to ensure all damage occurs in the ductile element, developed by Robert Park and Tom Paulay. These principles provided the basis for ductile column design and strong connecting elements that formed the Caltrans seismic design philosophy for concrete bridges.

The geometry and component design of concrete beam and girder bridges lent themselves to incorporating fuses and capacity-protected elements by increasing the transverse reinforcement in columns and shafts and fusing abutment shear keys and backwalls. However, accurately determining earthquake demands and structure capacities turned out to be a more difficult task.

After the San Fernando earthquake, Caltrans adopted elastic-dynamic analysis for estimating the inelastic displacement demands for bridges. This was based on Nathan Newmark's observation that long-period linear and nonlinear systems with the same initial period would have about the same maximum displacement for a given ground motion. This design philosophy was used to address the inelastic demands imposed by a large earthquake on a structure with proper structural detailing. The large moments and shear forces obtained from the

elastic analysis were reduced by *Z* factors to obtain the inelastic moments and shear forces on the structure (**Fig. 3**). This method of design and analysis is still allowed in the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[1] and is often referred to as the "force method."

During the 1970s, Caltrans adopted a response spectrum analysis to determine the seismic demand on bridges based on the factors *A*, *R*, and *S*. A response spectrum is a plot of the maximum response (acceleration, velocity, or displacement) of single degree-of-freedom systems having different natural periods that are all excited by the same base motion. The *A* factor was the peak rock acceleration obtained from seismic hazard maps, and *R* was the normalized 5%-damped rock spectra response factor. By multiplying *A* by *R*, an elastic spectrum for rock was obtained. The rock motion was converted to a soil spectrum by multiplying the spectrum by an *S* factor. The product of these three factors obtained at different periods provided a smooth, elastic, 5%-damped response spectrum at any bridge site (Fig. 2). However, these spectra were based on a deterministic "maximum credible earthquake" of each fault. The adoption of probabilistic response spectra, which addressed the likelihood of larger earthquakes over longer time periods, by AASHTO and later Caltrans took another 20 years.

Caltrans's first *Seismic Design Criteria* (SDC) was published in 1973. **Figure 4** is reproduced from that publication and shows the elastic force reduction due to the nonlinear response for bridge columns, abutments, and restrainers.

After the Loma Prieta earthquake, Caltrans began retrofitting its vulnerable bridges for seismic events. However, researchers and bridge engineers realized that better tools that incorporated nonlinear capacities of bridge elements were needed. At the University of California–San Diego, Nigel Priestley and Frieder Seible pioneered column and cased-column testing and new tools for capturing the post-elastic concrete column response that contributed greatly to the success of the seismic retrofit program.

Caltrans engineer Mark Mahan developed software to obtain moment and deformation capacities of new and retrofitted columns and shafts and "pushover" software that generated the displacement capacities of bridge frames. In 1999, Caltrans adopted displacement-based design principles for all bridges.[2] However, it would take many years before Caltrans developed procedures to reliably obtain the inelastic displacement demands for ordinary bridges.

The Northridge earthquake showed engineers that large variations in superstructure mass and substructure stiffness in adjacent elements, such as variations in columns of a bent, bents of a frame, or frames of a bridge, increased earthquake damage. Shortly after Northridge, Caltrans required that the stiffnesses of columns and bents be balanced to prevent damage to stiffer elements during strong shaking.

After the Northridge earthquake, Caltrans also developed "ARS Online," which allowed engineers to obtain both probabilistic and deterministic ground motions for any bridge site in California. Although AASHTO adopted probabilistic seismic hazards in 1994, it wasn't until 2009 that Caltrans adopted probabilistic spectra. At that time, Caltrans started using the larger of deterministic and probabilistic ground shaking spectra;

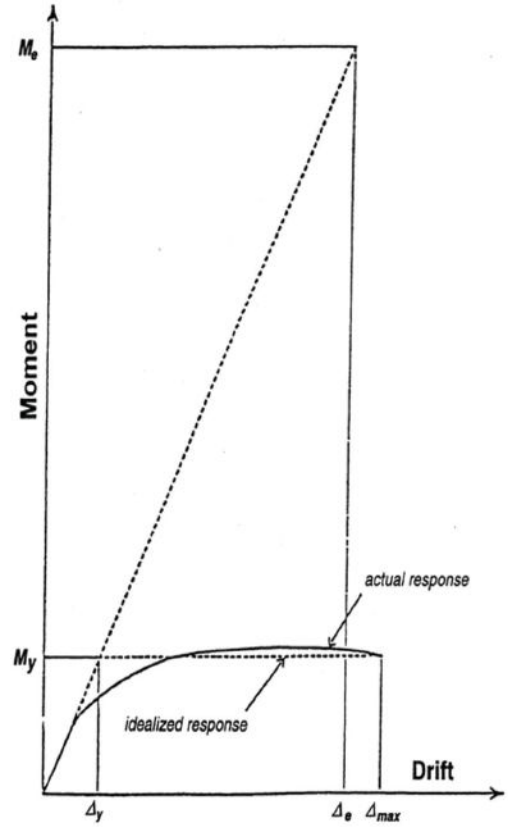


Figure 3. Moment *M* compared with drift *Δ* for a typical bridge element showing the theoretical elastic and the actual behavior. The *Z* factor was defined as the reduction of the seismic moment (or force) due to the nonlinear response of the bridge: $Z \approx M_e/M_y$.

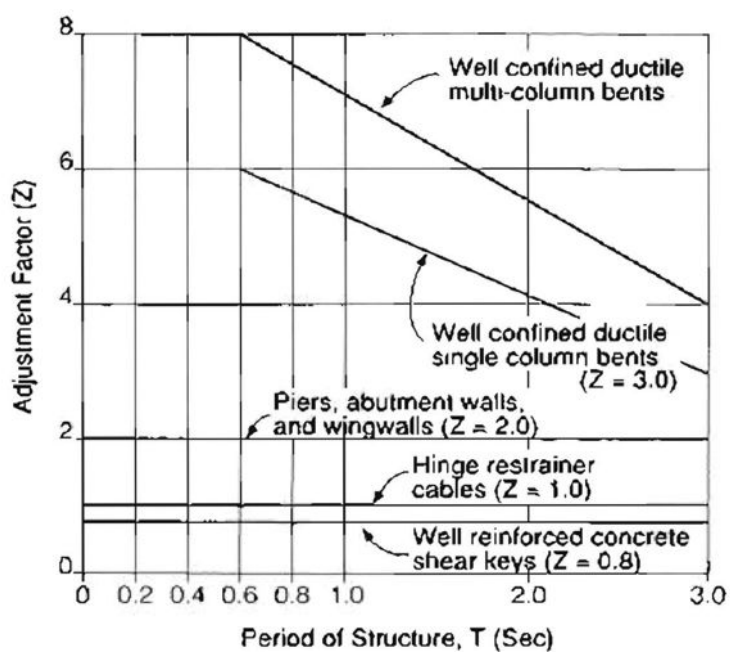


Figure 4. *Z* factor chart for different types of elements from the 1973 Caltrans *Seismic Design Criteria*.

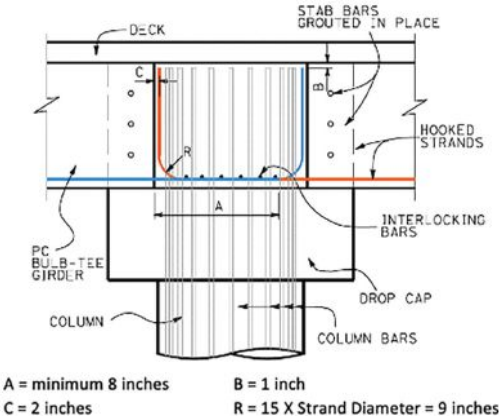


Figure 5. Draft details from Caltrans for accelerated bridge construction of a precast concrete girder–to–bent cap connection using extended, hooked strands for continuity.

eventually, after finding that probabilistic spectra controlled in active seismic regions, Caltrans abandoned the use of deterministic ground shaking spectra. These probabilistic spectra allowed Caltrans to begin using probabilistically derived (through spectral matching) suites of time histories that could be used to analyze nonlinear bridge models. In addition, Caltrans also started using other probabilistically derived seismic hazards such as fault offsets. Adopting probabilistic seismic hazards enabled Caltrans to develop guidance and tools for performing nonlinear time history analyses for typical bridges.

Rather than always relying on column plastic hinges as the earthquake-resisting element on bridges, Caltrans developed criteria for isolation devices, damping devices, and rocking foundations. These alternative earthquake-resisting elements had been used for retrofits but are now being applied to the design of new bridges.

In support of the accelerated bridge construction initiative, Caltrans has developed a suite of capacity-protected details for precast concrete girders to ensure the inelastic behavior is limited to the ductile column elements (**Fig. 5**).

Caltrans has incorporated several other hazards into the current version of its SDC,[3] including probabilistic methods for determining the fault offset and tsunami hazard along the coast. Updated rules for designing bridges for liquefaction and lateral spreading are being tested on new bridges and the retrofit of existing bridges. Because Caltrans bridges will be tested during large future earthquakes, the SDC will likely continue to evolve.

The greatest lesson learned from the three momentous California earthquakes is that bridge engineers will never be done improving the seismic resiliency and safety of our bridges through research and innovation. As aptly stated by the late Joseph Penzien, chairman of the Caltrans Seismic Advisory Board, "Earthquakes measure our actions, not our words."

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3. Caltrans. 2019. *Seismic Design Criteria*. Version 2.0. Sacramento, CA: Caltrans.

Reflections on 25 Issues of *ASPIRE*®

Photo: PCI.

by Dr. Reid W. Castrodale, Castrodale Engineering Consultants, PC

As I was beginning work on this issue of *ASPIRE*®, I realized it is the 25th issue for which I have been the managing technical editor. This issue is also the 57th issue since the first one appeared in Winter 2007, which means we are embarking on the 15th year of *ASPIRE*. So, it seemed like a suitable time to pause and reflect on the past six years during which I have been involved in collecting ideas for articles, then following them through editing and production to a completed issue. Fortunately, I get to stay involved in the technical editing and can leave the production in the capable hands of our great staff.

In 2014, I took over the role of the managing technical editor from Dr. Henry G. Russell—a giant in our field, and a good friend. He had done a terrific job working with John Dick, the executive editor, and others to get *ASPIRE* started and make it into a highly respected resource for the concrete bridge community. When I began work on *ASPIRE*, William Nickas was the editor-in-chief, having taken over that role from John Dick a number of years earlier. William is still the editor-in-chief, and his many strengths, varied experiences, and keen insights have suited him well to lead *ASPIRE* to continue to produce the high-quality content that has been the hallmark of the magazine. The *ASPIRE* team also works with several of the members of the National Concrete Bridge Council (NCBC), whose collective mission is, in part, to gather and disseminate information on the design, construction, and condition of concrete bridges. These members of NCBC are the sponsors of *ASPIRE* (see the list on page 2) and comprise *ASPIRE*'s editorial advisory board. As such, they provide direction and reviews for articles. We also have an outstanding staff who perform the editorial and production work to prepare each issue for publication.

In addition to being responsible for the technical review and editing of articles, my main role is to develop content for the magazine, so I am always looking for ideas for articles. *ASPIRE* is "the concrete bridge magazine," so I endeavor to provide articles on concrete bridges of all types and sizes, from cast-in-place, short-span slab structures to pretensioned concrete girders and post-tensioned segmental box-girder bridges. Articles also discuss concrete decks, foundations, and substructure elements, as well as topics related to design, construction, rehabilitation, and repair of concrete bridges.

In seeking topics for articles, I make a conscious effort to distribute projects and topics around the country and give many engineers opportunities to provide content for *ASPIRE*. I have been impressed and pleased that engineers, contractors, and others have indeed been willing to write articles, and that the articles are generally well written when we receive them. Indeed, the task of recruiting authors has been quite enjoyable. It has been good to see authors deliver interesting and informative articles, and that the articles provide an opportunity to recognize companies, agencies, and engineers for the work they have done.

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Many very interesting articles have been published in *ASPIRE* over the years, so it is instructive to take a stroll back to the beginning and look at some of the early issues, which are all available on the *ASPIRE* website: www.aspirebridge.org. While you are on the website, you can also sign up for a print subscription (if you are not already a subscriber) or search for specific content across all issues if you are interested in learning more about a particular bridge or topic (use the link on the lower right corner of the home page to access the search function in the phone/tablet version). Additional information related to an article, such as extra photos for project articles or backup calculations for technical articles, may be provided in archived issues or on the website, so please check that resource.

We have a number of regular articles within *ASPIRE*. I'm sure many of you are familiar with them. One that has been especially popular is the LRFD feature that Dr. Dennis Mertz, late professor at the University of Delaware, wrote so capably from the first issue and up to his untimely death in 2016. Dr. Oguzhan "Ozzie" Bayrak, professor at the University of Texas at Austin, has taken over that series and has been doing a great job. We appreciate the contributions to our understanding of the *AASHTO LRFD Bridge Design Specifications* and its background, development, and application that we can obtain from these two outstanding engineers.

Another of the regular features that has been a highlight of the magazine is the brief Aesthetics Commentary written by Fred Gottemoeller about one of the issue's featured bridge projects. Over the years, he has done a terrific job giving us insights into aesthetics for bridges and has helped us learn how to make our basic, functional structures complement their surroundings without being extravagant or adding unnecessary ornamentation.

Unlike the *PCI Journal* or *PTI Journal*, *ASPIRE* is not a periodical with peer-reviewed research papers, but its articles do occasionally explore some more complex technical topics. Articles on several such topics have been included in the last few years, including ultra-high-performance concrete (UHPC), the effects of sweep in pretensioned concrete girders, and the current series on concrete anchorage. While these articles take considerably more effort to produce than our typical articles, we hope they have been valuable to our readers.

ASPIRE also provides articles that encourage us as engineers to think about more than just the production of our next bridge. In today's environment, we need to consider resilience, sustainability, and engineering professionalism. We also need to mentor younger engineers who will soon be taking our places.

There are so many good stories to be told about concrete bridges. I wish there were more pages in the magazine so more projects and concepts could be presented. (By the way, the length of the magazine is a function of the amount of ad revenue. *ASPIRE* delivers 14,000 paper copies and has approximately 6000 online readers for each issue. If your company would like to be exposed to over 20,000 bridge engineers, owners, contractors, and suppliers every quarter, please consider purchasing an advertisement in the magazine.) I think it is important to tell these stories so that others can be encouraged and can learn from our experiences, as we also learn from theirs. I encourage each of you to think about projects and topics you could contribute to *ASPIRE*, then put together a team, including younger engineers, to write an article.

It has been my pleasure to be involved with *ASPIRE* for these 25 issues. I look forward to working with many of you as you become authors of articles in future issues. Speaking for all of us on the *ASPIRE* team, we would certainly like the magazine to continue to be the "go-to" resource for concrete bridge concepts, design, and construction. We hope the magazine gives you many ideas to enable you to do a better and more efficient job of designing and building bridges using the amazing material that concrete is.

Redundancy and Ductility for Bridge Design

by Dr. Oguzhan Bayrak, University of Texas at Austin

My article in the Summer 2020 issue of *ASPIRE*® provided an introductory discussion on structural, load path, and internal redundancies. This article is the first of a series of articles that will continue to expand that discussion of redundancy. The series is intended to provide a context for discussions of different types of redundancies for concrete bridges that can be used when adopting new materials and technologies, as the art of concrete bridge design continues to advance. At the conclusion of the series, the resilience and robustness of concrete bridges will hopefully be clear to all of us. These attributes of concrete bridges, alongside their cost-effectiveness, architectural appeal, reduced maintenance costs, and durability, are the reasons bridge engineers and owners around the world have chosen, and continue to choose, concrete bridges as the preferred bridge solution. As we begin this series, it should also be noted that the topics covered in this article and the upcoming articles are not new; they are based on concepts that are well known by engineers designing concrete bridges.

Preamble

To facilitate this discussion, let us start with an example of a concrete beam that will introduce the key terminology used in this article and its relationship to the notation and concepts used in the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*.[1] **Figure 1** shows a typical structural test conducted on a reinforced or prestressed (pretensioned or post-tensioned) concrete beam. To simplify our discussion, let us focus on a case where structural performance is governed by flexure. For such a test specimen, we can consider its flexural capacity from several perspectives. When using the AASHTO LRFD specifications to calculate the nominal flexural resistance M_n of such a beam, we simplify our calculations and typically make conservative assumptions by, for example, using a rectangular stress block (that is, the Whitney stress block) to represent the distribution of compressive stresses on the compression side, and by assuming a bilinear response for the reinforcement by ignoring strain hardening. For this example, we assume that the design compressive strength of concrete is 4 ksi, the beam is reinforced with Grade 60 deformed reinforcing bars, and the strain hardening behavior of the flexural reinforcement, which may be significant, is ignored. The experimental capacity M_{exp} of the member is determined based on the actual strength of concrete, which in this example is 4.5 ksi, somewhat higher than the specified strength; the actual yield strength of Grade 60 reinforcing bars, which in this case is 67 ksi and also somewhat higher than their nominal strength; and the strain hardening behavior of the flexural reinforcement is considered, with f_{ult} = 85 ksi. Therefore, the actual or experimental capacity M_{exp} is significantly higher than M_n. This "overstrength" although present in many, if not all, bridge applications—is a margin that is typically ignored in our design calculations. In other words, our designs are based on what we specify, rather than what is likely to occur. The nominal flexural resistance of the beam M_n is then multiplied by a resistance factor ϕ that is less than or equal to 1.0 to obtain the factored flexural resistance

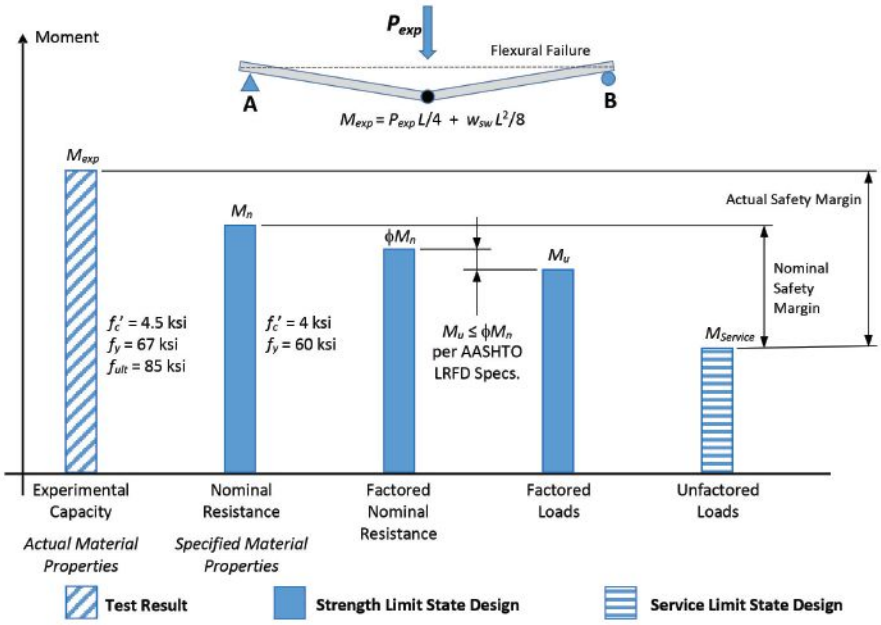


Figure 1. Design and experimental moments and associated safety margins for a reinforced or prestressed concrete beam. Note: f'_c = compressive strength of concrete; f_y = yield strength of flexural reinforcement; f_{ult} = ultimate strength of flexural reinforcement; L = span length; M_{exp} = experimental capacity; M_n = nominal resistance; $M_{service}$ = moment due to unfactored (service) loads; P_{exp} = experimental load; w_{sw} = beam self weight; ϕ = resistance factor; ϕM_n = factored flexural resistance. Figure: Dr. Oguzhan Bayrak.

$M_r = \phi M_n$. In other words, the nominal resistance is reduced by the resistance factor to account for variability of material properties, structural dimensions, and other uncertainties. Also, the AASHTO LRFD specifications make explicit accommodations for brittle and ductile failure modes by assigning an appropriate ϕ factor for flexural resistance (see the discussion related to variable resistance factors later in this article). As explained in the following section, material properties and the resistance factor ϕ are not the only sources of safety margin in our designs.

Vehicular Live Load

For bridge design, engineers use the AASHTO LRFD specifications' HL-93 vehicular live load, which consists of the design truck or design tandem and the design lane load. This vehicular live load is a notional load and does not represent a specific type of vehicle (see the LRFD articles in the Summer 2009 and Fall 2009 issues of *ASPIRE* for a more complete discussion of the development and significance of the HL-93 vehicular live load). As described in AASHTO LRFD Article C3.6.1.2.1, the load was "developed as a notional representation of shear and moment produced by a group of vehicles routinely permitted on highways of various states under 'grandfather' exclusions to weight laws."

As traffic patterns change, the vehicular live load model continues to be monitored and evaluated using weigh-in-motion measurements taken over past decades.

Safety Margin

While the safety margin for bridges is not explicitly addressed in the AASHTO LRFD specifications, the concept is incorporated by requiring the moment M_u from factored load effects to be less than the factored resistance ϕM_n, as stated in a general form in AASHTO LRFD Article 1.3.2.1. A margin of safety is achieved in the context of load and resistance factor design methodology by calibrating the load and resistance factors for the strength-limit state using a statistical approach to ensure a "probability of exceedance of 2/10,000 during the 75 year design life of the bridge" (AASHTO LRFD Article C1.3.2.1), which corresponds to a target reliability index of $\beta = 3.5$ (see the LRFD article in the Winter 2007 issue of *ASPIRE* for more information on the calibration approach). This concept will be discussed further in a future article in this series.

Design for the strength-limit state, according to AASHTO LRFD Article 1.3.2.4, is intended "to ensure that strength and stability, both local and global, are provided to resist the specified statistically significant load combinations that a bridge is expected to experience in its design life." Using the terminology of the AASHTO LRFD specifications for flexural design, the demand M_u from the factored load effects must not exceed the factored resistance ϕM_n. Load factors for the strength-limit state are greater than 1.0, unless the use of a smaller load factor results in a more severe effect for a particular load combination. When designing the typical reinforced or prestressed concrete beam of Fig. 1 for the Strength I limit state load combination, we would use a dead load factor of 1.25 and a live load factor of 1.75. To simplify the discussion for this beam, other types of loads and the multiple presence of live loads will not be considered. As illustrated in Fig. 1, by requiring ϕM_n to be greater than or equal to M_u, a nominal safety margin is provided. This safety margin is not explicitly discussed in the AASHTO LRFD specifications, nor is it "AASHTO-sanctioned" terminology. Furthermore, this safety margin is not constant because it depends on the dead load (DL) to live load (LL) ratio (DL/LL) and other factors.

Design for the service-limit state, according to AASHTO LRFD Article 1.3.2.2, is intended to provide "restrictions on stress, deformation, and crack width under regular service conditions." The service-limit state represents the day-to-day operation of the bridge, usually with the maximum expected vehicular load as discussed earlier, so service-limit-state design provisions address serviceability criteria. Load factors for service-limit states are 1.0 (AASHTO LRFD Article 1.3.2.1), except in special situations, like the Service III limit state for prestressed concrete design. Initially, service-limit-state provisions were experience based and were not calibrated as had been done for the strength-limit states. Also, the consequence of exceeding a service-limit-state requirement is damage with a potential for decreased performance or service life rather than excessive yielding and cracking, as is generally the case when strength-limit-state requirements are exceeded.

The concept of a safety margin for the service-limit state is difficult to develop, but it is instructive to compare service loads to the nominal resistance or experimental capacity as shown in Fig. 1 to obtain a functional or operational feel for the safety margin for an element. For the concrete beam tested in flexure, as shown in Fig. 1, the actual safety margin between the service load $M_{Service}$ and the experimental capacity M_{exp} is typically greater than the nominal safety margin between the service load and the nominal capacity M_n. While this discussion has been focused on flexural behavior, analogous discussions can be formulated for shear and other behavioral modes.

Structural Redundancy

With this background, we can now move to the discussion of structural redundancy. To simplify our discussion, let us focus on actual behavior and nominal capacities. Certainly, as previously discussed, factored loads and resistances in compliance with the AASHTO LRFD specifications, which set the minimum standards employed in bridge design, will provide sufficient safety margins. More sophisticated analyses resulting in better estimates of strength are permitted but are not mandated. However, using such methods will typically reduce the safety margin, with due credit given to sophisticated analyses.

Structural redundancy is directly related to structural indeterminacy. A simply supported beam will form a collapse mechanism after the formation of one plastic hinge (**Fig. 2a**), whereas a continuous beam will require the formation of two plastic hinges (**Fig. 2b**) before a collapse mechanism can form. This means that when the yielding of flexural reinforcement occurs at midspan, the load effects can be redistributed within an indeterminate structure from the span with the initial hinge to the

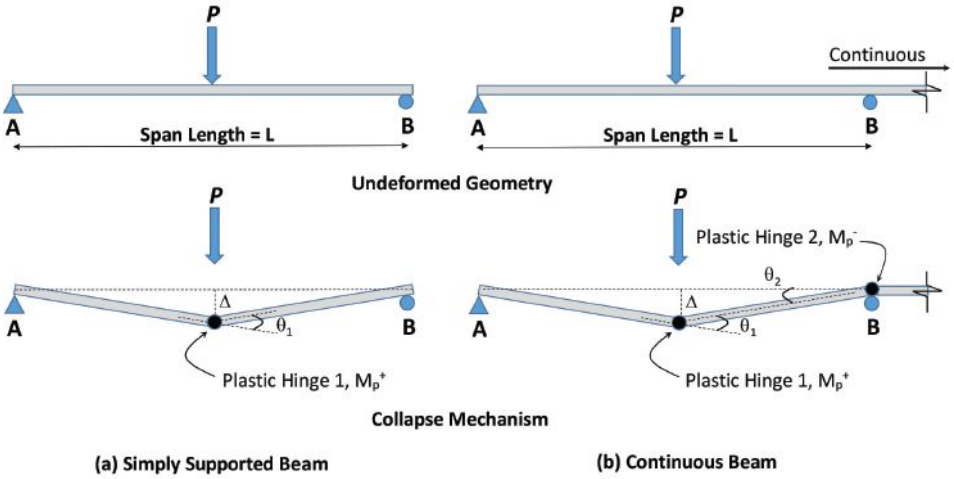


Figure 2. Collapse mechanisms for (a) simply supported beam and (b) continuous beam. Note: L = span length; M_p^+ = positive plastic moment; M_p^- = negative plastic moment; P = applied load; Δ = deflection at plastic hinge 1; Δ_1 = rotation at plastic hinge 1; θ_2 = rotation at plastic hinge 2. Figure: Dr. Oguzhan Bayrak.

remaining portion of the member. While not typically used in designs, the collapse load can be calculated using energy equilibrium equations. For the example, in Fig. 2, external work is equal to the product of the applied load and the deflection, that is $P\Delta$. Internal work is equal to the product of the plastic moment and the rotation, which is $M_p^+ \theta_1$ for the simply supported beam, and $M_p^+ \theta_1 + M_p^- \theta_2$ for the continuous beam. Because the load is acting at midspan, $\theta_2 = 0.5\theta_1$, and further, $\theta_2 = 2\Delta/L$, where L is the span length. Combining these expressions by equating the internal work to external work, we can calculate the collapse load for each case. That is, for a given geometry we can relate the collapse load P to M_p^+ for the simply supported beam, and to M_p^+ and M_p^- for the continuous beam.

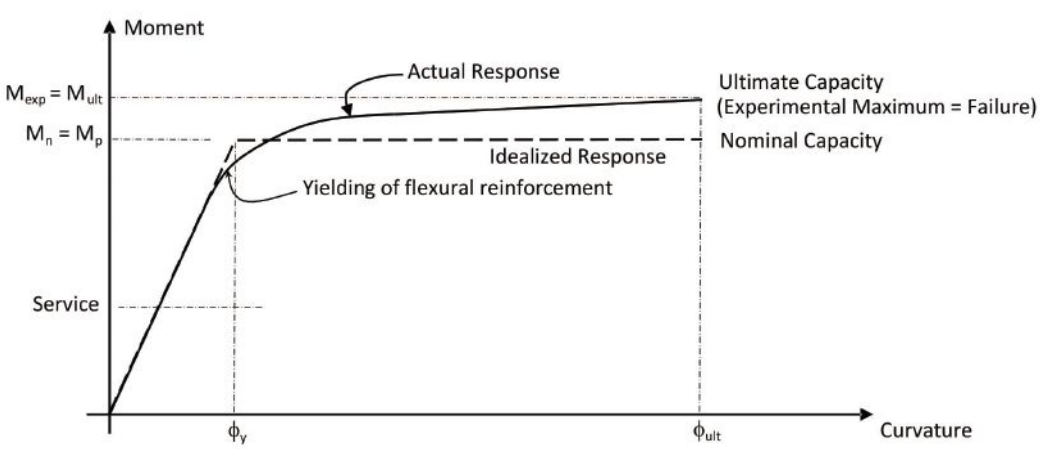
The concept of structural redundancy also highlights an important aspect of structural behavior that was not addressed in the calibration of the AASHTO LRFD specifications: the behavior of a structural system rather than a structural component. The calibration at the strength-limit state for the AASHTO LRFD specifications was based entirely on the performance of structural elements. However, if the interaction of elements in a structure is considered, there is often additional capacity that can be achieved as loads can be redistributed to adjoining elements when one element has reached its capacity. However, these benefits are not easily quantified, and much work remains before structural system behavior can be considered in the design specifications.

Ductility of Cross Section

To realize the benefits of structural redundancy, the member must have adequate ductility at the hinge locations. Evaluating the member ductility at the plastic hinge locations needs to be performed at the sectional level. For example, let us assume the beams shown in Fig. 2 are reinforced concrete beams detailed such that they possess ample shear strength and their response will be governed by flexural yielding. In this case, we can use sectional analysis software programs that perform layered-section analyses and determine the response of the beam section. **Figure 3** shows a typical moment-curvature response and its idealized form. While this figure represents positive bending response, such as at hinge 1 in Fig. 2, a similar response can be evaluated for negative bending, such as at hinge 2 in the continuous beam in Fig. 2.

In Fig. 3, the idealized form of the moment-curvature response has slightly lower moment capacity $M_n = M_p$ than the actual moment capacity M_{ult}. On the deformation side (the horizontal axis), the idealized yield curvature ϕ_y and ultimate curvature ϕ_{ult} are identified for the calculation of curvature ductility. The ratio of ϕ_{ult} to ϕ_y is known as the curvature ductility factor. This factor denotes the ability of the section to undergo plastic deformation in a stable manner, that is, without losing capacity. The moment-curvature relationship shown in Fig. 3 can be used to obtain the moment-rotation relationship, which can also be used as a measure

Figure 3. Actual and idealized moment-curvature responses for a reinforced concrete beam. Note: M_{exp} = experimental capacity; M_n = nominal resistance; M_p = plastic moment; M_{ult} = actual ultimate capacity; ϕ_{ult} = curvature at ultimate capacity (failure); ϕ_y = idealized yield curvature. Figure: Dr. Oguzhan Bayrak.



of overall ductility of the structure. To use this approach, the length of the plastic hinge must be known, or conservatively assumed. Mattock made a series of recommendations in this regard.[2] More recently, Bae and Bayrak analyzed a series of different conditions for columns and provided an expanded set of guidelines that reflect lessons learned since the 1960s.[3] When applying this concept to girders, it is important to appreciate that a traditional plastic hinge that develops in a well-confined region of a reinforced concrete column and one that develops in a beam (that is, one or a few wide cracks leading to yielding of reinforcement that crosses those cracks) are quite different from one another. The moment-rotation relationship for the member whose moment-curvature is shown in Fig. 3 would have a very similar shape to the moment-curvature behavior depicted in Fig. 3, but with the horizontal axis representing rotation. With this information, rotational ductility of the plastic hinge regions can now be explored. To take advantage of structural redundancy, a member should be detailed to permit the formation of a sufficient number of hinges and allow significant deflection that will signal impending failure. For details typically seen in our concrete bridges, it is expected that redundancy due to continuity provided at support B as in Fig. 2b will increase the collapse load and result in additional warning of impending failure.

Deformation Capacity

Detailing of the beam—material properties for the concrete, longitudinal reinforcement, and presence of confinement reinforcement (transverse reinforcement)—will determine the shape of the moment-curvature or moment-deflection curve, and therefore the ductility of the response. For reinforced concrete, plastic deformations can largely be attributed to the yielding of flexural tension reinforcement. The deformation capacity of the flexural tension reinforcement has a significant effect on the curvature ductility and therefore the rotational capacity of the section. For example, ASTM A615 Grade 60 reinforcing bars rupture at a minimum strain of 7% to 9%. ASTM A706 Grade 60 reinforcing bars are more deformable and rupture at a strain level of 10% to 14%. These high-rupture strains indicate that members reinforced with these materials can experience large rotations after cracking, thereby providing ample warning prior to failure as long as a compression failure is avoided.

Now we will consider prestressing strands and their effect on the behavior of prestressed concrete girders. Typical volume changes that prestressed concrete members experience due to creep and shrinkage are large enough to render the use of conventional reinforcement impractical in prestressed concrete applications. It is for this reason that the invention of high-strength steel wires made prestressed concrete viable. Since the first prestressed concrete girder bridge in the United States (Walnut Lane Memorial Bridge in Philadelphia, Pa.) was constructed in 1950, the strength of prestressing reinforcement (wires, post-tensioning bars, and strands) has gradually increased over the years. Today, the most common grade for prestressing strands is Grade 270. While this product was first produced using stress-relieving techniques, a strain-tempering process was subsequently introduced, and today low-relaxation strands are the industry standard. The minimum elongation capacity of Grade 270 seven-wire strands is 3.5% according to ASTM A416. This value is substantially smaller than the deformation capacity of Grade 60 deformed reinforcing bars, as previously discussed. Even with the reduced material deformation capacities of prestressing strand, prestressed concrete beams have been routinely used in our bridges, it has been demonstrated repeatedly that prestressed concrete girders have sufficient ductility to form wide cracks and significant deflections that signal impending failure.

This leads to a discussion of how much deformation capacity is needed in our bridges. While the answer to this question is somewhat subjective, it is commonly accepted that we would like to see sufficient levels of inelastic deformation (deflection) and associated cracking in our bridges to provide warning as they are loaded to failure. Based on decades of structural tests, the deformation capacity displayed by typical prestressed concrete bridges has been shown to be more than adequate.

As discussed earlier, the AASHTO LRFD specifications are focused on providing adequate strength but aspects of deformation capacity and ductility are included in the design provisions, which are intended to ensure yielding and cracking. However, the specificactions permit members with limited ductility based on the use of a variable resistance factor as shown in **Fig. 4**. Ductile girders are assumed to be governed by

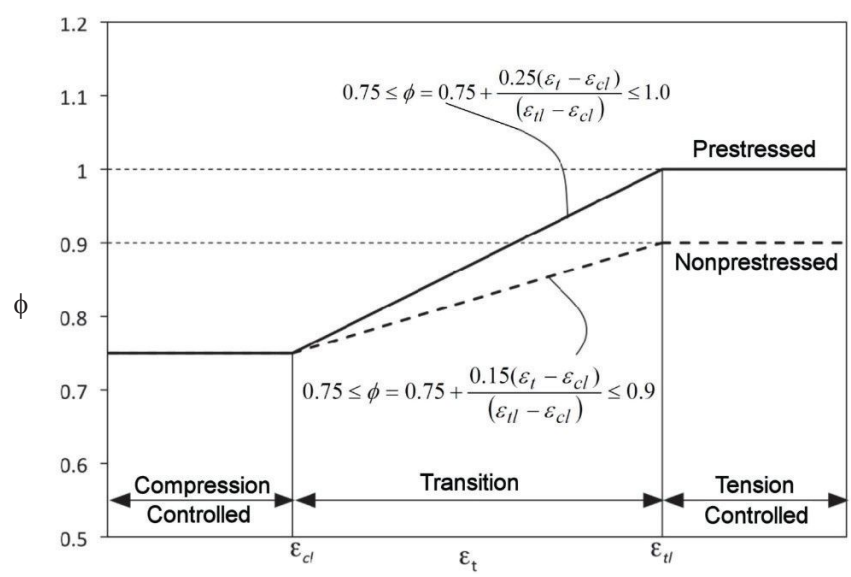


Figure 4. Variation of resistance factor ϕ with net tensile strain ε_t for prestressed and nonprestressed elements as behavior transitions from a tension-controlled section to a compression-controlled section. Figure: *AASHTO LRFD Bridge Design Specifications*, 9th ed., Figure C5.5.4.2-1.

tension-controlled behavior in which the tension reinforcement yields. However, the current design provisions also allow compression-controlled designs through the use of a lower resistance factor, as shown in the figure. While compression-controlled designs are very rare in flexural members, the specifications do allow them, albeit with lower resistance factors. With the introduction of some new materials as flexural reinforcement, it may become more common to see designs where the member capacity is compression controlled. The appropriate implementation of this concept will be discussed in future articles in this series.

The AASHTO LRFD specifications also require that members have a minimum quantity of reinforcement to avoid failure as soon as a member develops a flexural crack. These provisions, which are found in AASHTO LRFD Article 5.6.3.3, require that the minimum amount of prestressed and nonprestressed tensile reinforcement be adequate to develop factored flexural resistance ($M_r = \phi M_n$) greater than or equal to the lesser of 1.33 times the factored moment M_u and the cracking moment M_{cr}. The calculation of M_{cr} using AASHTO LRFD Eq. 5.6.3.3-1 includes factors that take into account the variability in the flexural cracking strength of concrete, the variability of prestress, and the ratio of nominal yield stress to ultimate stress for the flexural reinforcement. So in this situation, members must be designed to provide a significantly increased nominal capacity so the probability of a nonductile (brittle) failure is very low.

Conclusion

This article is the first of a series of articles on redundancy and ductility. It has provided background on the basic design approach in the AASHTO LRFD specifications and a further discussion of the concept of structural redundancy. Subsequent articles will continue discussions of different types of redundancies for concrete bridges to provide background information that we can use in adopting new materials and technologies as bridge design continues to evolve.

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PROJECT

Interstate 10/Ina Road Traffic Interchange Reconstruction

The Interstate 10/Ina Road interchange provides a gateway to the town of Marana, Ariz. The project included two bridges, the Interstate 10 interchange bridge is to the left and the Union Pacific Railroad bridge is to the right with the ramp supported by mechanically stabilized earth retaining walls between the bridges. Photo: Sundt/Kiewit Joint Venture.

by David Leistiko, Kimley-Horn and Associates Inc.

Interstate 10 (I-10) traverses the eastern and northeastern portions of Pima County, Ariz., and is part of the CANAMEX Corridor. Designated as such by federal legislation, the CANAMEX Corridor is a trade route extending from Canada to Mexico. The Arizona Department of Transportation (ADOT), in conjunction with the Federal Highway Administration, identified the need to reconstruct portions of I-10 to increase the roadway capacity and to improve operational efficiency. The Regional Transportation Authority in Pima County also identified the need to improve the capacity and safety of the crossroads through the elimination of at-grade crossings with the Union Pacific Railroad (UPRR) mainline tracks on the east side of the interstate.

ADOT selected a design team to design improvements to the I-10/Ina Road traffic interchange and widen 1.5 miles of I-10 to accommodate a future 10-lane roadway section, new auxiliary lanes, and entrance and exit ramps. Due to the project's complexity, unique challenges, and schedule constraints, ADOT proactively used the construction manager at risk (CMAR) delivery method. The design team worked closely with the selected CMAR contractor and through successful collaboration and partnering efforts, identified and implemented value engineering opportunities that saved $18 million in construction and right-of-way costs. The CMAR contractor provided preconstruction- and construction-phase services to complete the reconstruction of the Ina Road traffic interchange.

Concrete bridges are by far the predominant bridge type used in Arizona because of their relatively low cost, low maintenance, and the ready availability of the raw materials needed to produce concrete. Prestressed concrete I-girder and post-tensioned box-girder bridges are the most common bridge types constructed on Arizona's highways and local roads.

The major structural components of the I-10/Ina Road traffic interchange project included two concrete girder bridges and mechanically stabilized earth (MSE) retaining walls with precast concrete fascia panels. This article will focus on the two bridges over I-10 and the UPRR tracks.

I-10/Ina Road Traffic Interchange Underpass Preliminary Design

The preliminary design for the I-10/Ina Road traffic interchange underpass was a two-span, 292-ft 8¾-in.-long, 78-in.-deep precast concrete AASHTO Type VI-S girder bridge on a 42-degree skew with an out-to-out width of 121 ft 9 in. The preliminary design was included in the stage II (30%) plans that were prepared by ADOT and provided to the design team at the beginning of the project. The substructure consisted of full-height abutments and a multicolumn pier supported on drilled-shaft foundations.

Ina Road/UPRR Overpass Preliminary Design

The preliminary design for the Ina Road/UPRR overpass consisted of a three-span, 318-ft 9-in.-long hybrid cast-in-place post-tensioned box-girder section for spans 1 and 3 with short cantilevered sections of the box girder over the piers. Span 2 consisted of a drop-in span using 72-in.-deep precast concrete AASHTO Type VI girders. The bridge was designed with a 42-degree skew and an out-to-out width that varied from 143 ft 8 in. to

profile

INTERSTATE 10/INA ROAD TRAFFIC INTERCHANGE UNDERPASS AND INA ROAD/UNION PACIFIC RAILROAD OVERPASS / PIMA COUNTY, ARIZONA

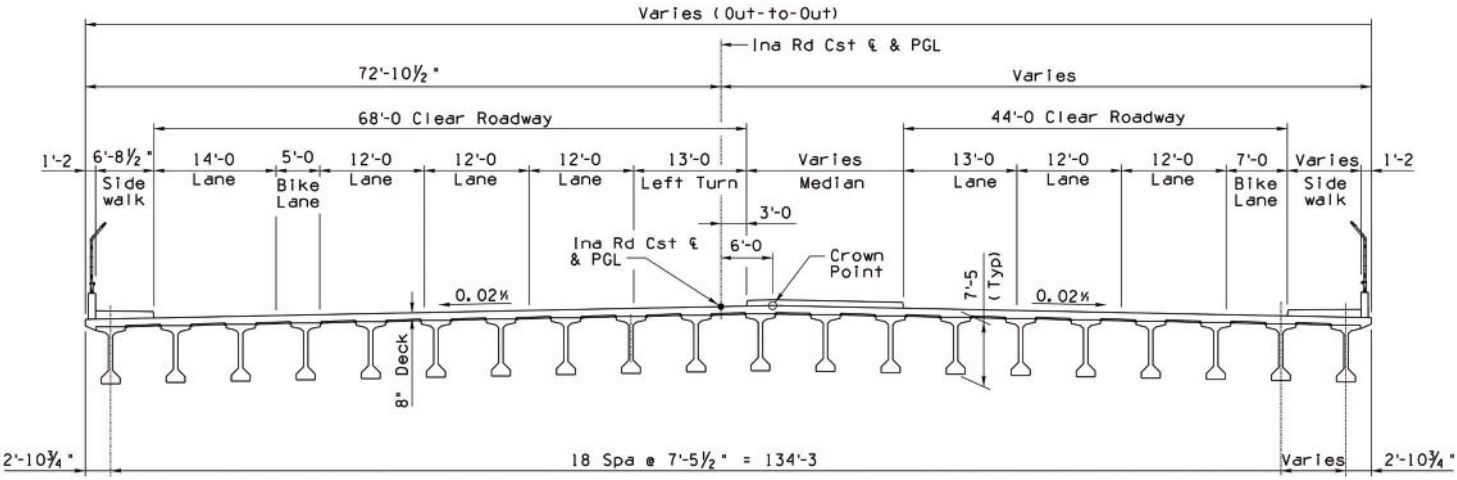
PRIME CONSULTANT: Psomas, Tucson, Ariz.

BRIDGE DESIGN ENGINEER: Kimley-Horn and Associates Inc., Phoenix, Ariz.

PRIME CONTRACTOR: Sundt/Kiewit Joint Venture, Tempe, Ariz.

PRECASTER: Stinger Bridge & Iron, Coolidge, Ariz.—a PCI-certified producer

PROJECT DESCRIPTION: Full reconstruction of the Interstate 10/Ina Road traffic interchange in Marana, Ariz. The project converted an existing interstate overpass and at-grade railroad crossing to an interstate underpass with bridges over Interstate 10 and the Union Pacific Railroad. The project included two precast concrete girder bridges and mechanically stabilized earth retaining walls with precast concrete fascia panels.



Typical section for Ina Road span 2 over the Union Pacific Railroad. (UPRR). As a result of value engineering, the final design used 78-in.-deep precast concrete AASHTO Type VI-S girders, which eliminated formwork towers within the UPRR right-of-way as well as railroad flaggers during construction. All Figures: Arizona Department of Transportation.

147 ft 6 in. The substructure consisted of full-height abutments and multicolumn piers supported on drilled-shaft foundations. Span 1 at the southwest corner of the bridge included a flared section to accommodate a right-turn bay on Ina Road. The cast-in-place post-tensioned box section would easily allow this flared portion of deck to be constructed.

Refining Preliminary Designs

Starting with these preliminary designs, the I-10/Ina Road structural design team began making refinements while working toward the stage III (60%) design submittal. The most notable change to both bridges was to replace the full-height abutments on two rows of drilled shafts with pier-style abutments on a single row of drilled shafts with MSE retaining walls located behind the piers. In addition, the main span for the Ina Road/UPRR overpass was refined to use a 63-in.-deep AASHTO Type V girder, reducing the superstructure depth by 9 in. The change to pier-style abutments allowed the designers to reduce the number of 60-in.-diameter drilled shafts at the abutments from a combined 104 shafts for both bridges to only 26 shafts. The shorter girder height at the UPRR overpass allowed the profile of Ina Road to be lowered, thus reducing the overall quantity of fill and MSE retaining wall. This change also eliminated the need to relocate two long-haul utilities at abutment 1 of the UPRR bridge. Together these changes resulted in a total project savings of close to $2 million.

Value Engineering Workshop

After the stage III (60%) submittal, ADOT held a weeklong value engineering study with the design team and the contractor. Through open dialogue among all parties and a willingness to look at alternative design options, the team further refined the design of the Ina Road/UPRR overpass bridge. The contractor immediately pointed out that the current design would require falsework towers within a UPRR right-of-way and railroad flaggers throughout construction, which would add costs and increase the construction duration of the cast-in-place construction. In addition, the contractor asked if it would be possible to use precast concrete girders for all three spans, pointing out that although deeper precast concrete girder sections would raise the profile and increase both fill and the MSE retaining wall quantities, the savings in the bridge would far outweigh these costs.

The structural design team agreed to evaluate different precast concrete girder sections for the bridge. Ultimately, the design team presented a revised superstructure design consisting of 54-in.-deep precast concrete AASHTO Type IV girders for spans 1 and 3, and 78-in.-deep precast concrete AASHTO Type VI-S girders for span 2. These design changes resulted in a three-span, 316-ft 8¾-in.-long bridge.

ARIZONA DEPARTMENT OF TRANSPORTATION, OWNER

STRUCTURAL COMPONENTS FOR BOTH BRIDGES: 2528 ft of 54-in.-deep precast concrete AASHTO Type IV girders, 7350 ft of 78-in.-deep precast concrete AASHTO Type VI-S girders, 800 ft^2 of bridge deck supported by 112 ft of cast-in-place concrete T-girders, 126,420 ft^2 of mechanically stabilized earth retaining walls with precast concrete fascia panels, cast-in-place concrete composite deck slabs, semi-integral pier-style abutments, cast-in-place concrete pier caps with multicolumn piers founded on drilled shafts

PROJECT COST: Interstate 10/Ina Road traffic interchange reconstruction cost: $126 million (budget), $136 million (initial CMAR estimate), $118 million (actual)

AWARDS: 2019 Arizona Transportation Partnering Excellence Award, 2020 Arizona Association of General Contractors Build Arizona Award, 2020 American Public Works Association (APWA) Southern Arizona Project of the Year (Transportation over $75M Category), 2020 APWA Arizona Statewide Project of the Year (Transportation over $75M Category), 2020 *Engineering News-Record* Southwest Region Best Projects Award (Highway/Bridge Category), 2020 American Council of Engineering Companies Arizona Grand Award

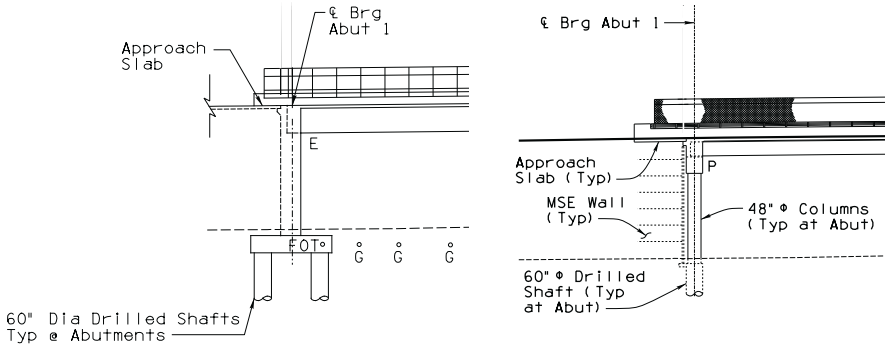
Final Design Challenge

To finalize the value engineering design changes to the Ina Road/UPRR overpass, the structural design team had to figure out how to handle the framing of the flared section of span 1 on the southwest corner of the bridge. The lane configurations for the traffic interchange coupled with the skew of the bridge required a flared (triangular) section of bridge deck for the turning movement from the I-10 ramps to eastbound Ina Road. With the change to precast concrete girders, a solution was needed to construct the required bridge deck that avoided constructing more bridge than needed. After evaluating the use of steel sections, precast concrete sections, and ledge beams, and overbuilding span 1, the final solution drew from the original design that used cast-in-place concrete. Using cast-in-place concrete T-girders for the flared section of span 1 provided a simple, cost-effective, and clean solution to this perplexing geometry.

Conclusion

This project was truly a team effort by ADOT, the design team, and the CMAR contractor. The total project savings of $18 million was only possible through this collaboration. The use of precast concrete girders accelerated construction over the UPRR, reducing construction costs and duration. Through the interaction and brainstorming with the contractor during the value engineering process, the structural design team gained valuable insight into construction preferences, efficiencies, and constructability.

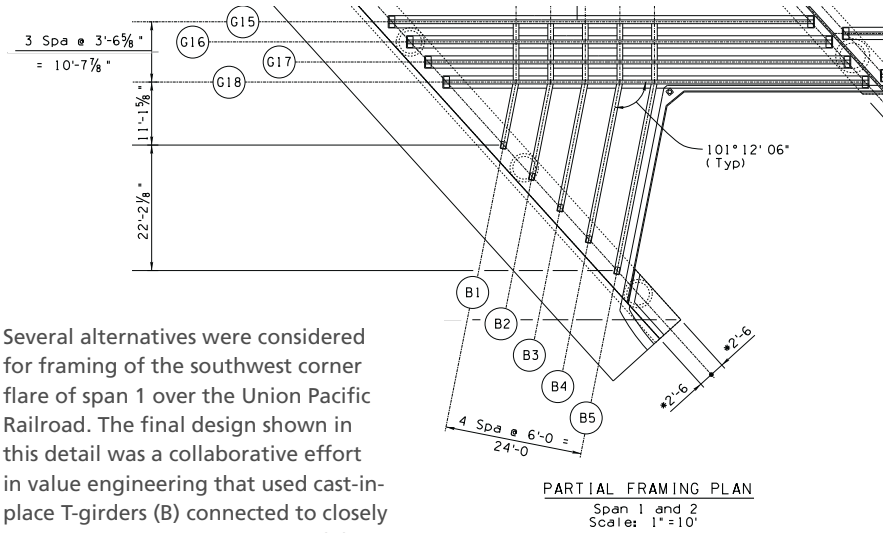
David Leistiko is a senior project manager and senior structural design engineer with Kimley-Horn and Associates Inc. in Phoenix, Ariz.



Full-height abutment

Pier-style abutment

The change from full-height abutments (left) to pier-style abutments (right) allowed the designers to reduce the number of 60-in.-diameter drilled shafts at the abutments from a combined 104 shafts for both bridges to only 26 shafts.



Several alternatives were considered for framing of the southwest corner flare of span 1 over the Union Pacific Railroad. The final design shown in this detail was a collaborative effort in value engineering that used cast-in-place T-girders (B) connected to closely spaced AASHTO Type IV girders (G).

Construction of the three-span Ina Road overpass at the Union Pacific Railroad (UPRR) in the foreground, with ramp construction and the Interstate 10 bridge beyond. The preliminary bridge design called for cast-in-place box girders, which would require falsework towers within the UPRR right-of-way as well as railroad flaggers. Closely spaced girders that will support the flared deck and T-girders are on the left edge of the span near the ramp. Although using precast concrete girders raised the road profile and increased fill, the savings far outweighed the costs. Photo: Sundt/Kiewit Joint Venture.

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Level 1&2 Multistrand & Grouted PT Inspector

PROJECT

U.S. Route 54 over the Canadian River

New Mexico's First Cast-in-Place Concrete Segmental Box-Girder Bridge

by Nyssa Beach, Jacobs Engineering, and
Jeff Mehle, McNary Bergeron & Associates

The new alignment of U.S. Route 54, with the nearly completed concrete segmental box-girder bridge, is offset just east of the existing alignment with its aging deck truss bridge. Photo: Malcolm International.

Amid the expanse of rural desert in northeastern New Mexico is the small, agriculture-centered village of Logan. In Logan, the Canadian River, which cuts deeply through the desert landscape, is crossed by a steel deck truss bridge carrying U.S. Route 54 (U.S. 54). This bridge is steeped in the rich history that brought commerce trails, rails, and roads to the area. Currently, the U.S. 54 corridor is the main trucking route from Chicago, Ill., to El Paso, Tex., with over 50% truck traffic. Additionally, it provides access to Ute Lake State Park—Ute Lake is the second largest lake in New Mexico and is popular with water and fishing enthusiasts.

The new U.S. 54 Canadian River Bridge is an exciting first for the state of New Mexico and a new chapter for the area's transportation future: a cast-in-place segmental box-girder bridge. This bridge, built with the balanced-cantilever method, was designed to replace a 1954 steel truss structure while also minimizing impacts to the Canadian River, the river's protected inhabitants, and the surrounding wetlands, as well as to adjacent historic and prehistoric archaeological sites. The new structure addresses the deficiencies and poor condition of the load-restricted steel deck truss bridge, thus improving safety and ensuring the future viability of the U.S. 54 corridor.

The New Mexico Department of Transportation (NMDOT) and the design team met the challenges of this unique location by conducting a comprehensive alignment and structure selection study while engaging public input. The result is a three-span segmental structure with a sweeping alignment that is offset just east of the existing river crossing.

Designing a Bridge for a Unique Location

The design of the Canadian River Bridge balances both the structural and functional requirements of the project. The new 43-ft-wide cross section includes two 12-ft-wide lanes, 8-ft-wide shoulders on each side, and barriers. The pier locations were carefully considered during design to minimize environmental impacts and avoid the river's floodplain. The three-span configuration was optimally set at 200, 325, and 210 ft. The box-girder depth varies from 18 ft at the piers to 8 ft at midspan and at the abutments. This segmental box-girder structure is on a 3000-ft-radius horizontal curve with a constant 4% cross slope.

During the balanced-cantilever construction, the superstructure was consistently built a half segment out of balance on either side. Each pier table was built out to be half a segment longer on one side such that when the 15-ft-long segments were cast on either side of the cantilever, the superstructure was only ever half a segment out of balance at any time. The design of the twin-wall columns efficiently handled the large out-of-balance moment during construction and carried it as a couple to the foundation. The piers and abutments are founded on deep concrete drilled shafts.

Project Challenges

Constructing a bridge in rural northeastern New Mexico presented several unique challenges to the U.S. 54 Canadian River Bridge project team.

profile

U.S. ROUTE 54 CANADIAN RIVER BRIDGE / LOGAN, NEW MEXICO

BRIDGE DESIGN ENGINEER AND OWNER'S ENGINEER: Jacobs Engineering, Denver Colo.

PRIME CONTRACTOR: Fisher Sand & Gravel New Mexico Inc., Placitas, N.Mex.

SEGMENTAL CONTRACTOR: Malcolm International, Rancho Cordova, Calif.

CONSTRUCTION ENGINEERING: McNary Bergeron & Associates, Broomfield, Colo.

POST-TENSIONING SYSTEM AND FORM TRAVELER SUPPLIER: Schwager Davis Inc., San Jose, Calif.

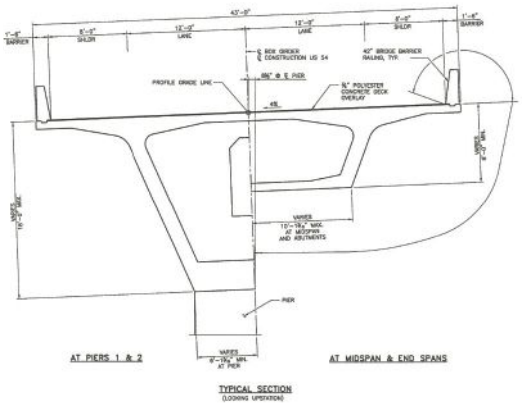
With the bridge about half complete, multiple construction operations were underway concurrently, including the nearly completed balanced-cantilever construction of cantilever 1, construction of pier table 2, and construction of both end spans on falsework. Photo: Malcolm International.

The new U.S. 54 alignment is on a sweeping 3000-ft-radius horizontal curve. Photo: New Mexico Department of Transportation.

Remote Location

Logan is approximately 200 miles east of both Albuquerque, N.Mex., and Santa Fe, N.Mex., and 100 miles west of Amarillo, Tex. It has a population of just over 1000 and is home to ranching and agriculture, as well as recreation at Ute Lake State Park.

The availability of workers with bridge construction experience was limited in Logan. Most of the workers were from out of town or out of state. Scheduling of major construction tasks needed to take workers' holidays and trips home into account. The remote site also led to a high turnover of laborers, which extended the learning curve associated with constructing a segmental bridge. To minimize the impact of the high turnover rate, a core group of key field staff members was relocated to New Mexico and remained consistent throughout the project. This helped keep consistency in key roles despite turnover challenges with construction staff in the remote project location. The segmental contractor also used three-dimensional sketchup models for workforce training to efficiently bring on new workers, as needed, and get them up to speed on the segmental construction and, in particular, the procedures, quality control, and terminology associated with each part of segmental construction. Scheduling, training, and coordination between tasks and operations was imperative for meeting the construction schedule.



Typical bridge section showing the box-girder depths, which vary from 18 ft at the piers to 8 ft at midspan and at the abutments. Figure: New Mexico Department of Transportation.

Brine Aquifer

During the final design phase, a specific challenge for the design team was the presence of a brine aquifer deep below the project site. If the aquifer were disturbed during construction, the high chloride concentration from the aquifer would contaminate the water supply downstream. Due to the aquifer's varying depth amid thin mudstone or siltstone layers, the greatest risk for penetration was around pier 2. The design team worked closely with the Canadian River Municipal Water Authority to weigh the risks and determine the best path forward. The final design mitigated risk by increasing the size of the pier footing and drilled shafts, which in turn allowed the shafts' lengths to be shortened.

The project's special provisions required the contractor to have a containment plan and system in place in the unlikely event that the drilling operations penetrated the aquifer. This containment plan included testing water samples from the Canadian River before drilling for a baseline and again after installing the drilled shafts. Provisions for capture and disposal of brine water were identified in case the testing showed elevated levels of saline. Although this phase of construction had uncertainty and potential risk, the drilling and shaft installation did not ultimately impact the brine aquifer and no additional mitigation measures during construction were required.

NEW MEXICO DEPARTMENT OF TRANSPORTATION, OWNER

OTHER MATERIAL SUPPLIERS: Reinforcement: CMC Rebar, Albuquerque, N.Mex.; bearings: D.S. Brown Company, North Baltimore, Ohio; expansion joints: Watson Bowman Acme, Buffalo, N.Y.; prepackaged grout: US SPEC, Denver, Colo.

BRIDGE DESCRIPTION: Three-span, 735-ft-long, post-tensioned, cast-in-place concrete segmental box-girder bridge

STRUCTURAL COMPONENTS: 38 variable-depth cast-in-place concrete segmental box-girder segments and three closure segments, end spans cast on falsework, monolithic twin-wall columns, pier footings, and drilled-shaft foundations

BRIDGE CONSTRUCTION COST: $22 million

Form travelers at either end of cantilever 1. Geometry control and monitoring during segment casting ensured accurate vertical and horizontal alignments. Photo: Malcolm International.

Concrete

For segmental construction, a high-performance concrete is required. This bridge's design specifically required concrete with a 28-day compressive strength of 6000 psi, and a 3500-psi minimum strength was required prior to tensioning the post-tensioned tendons. To meet the fast-paced construction schedule of the desired seven-day casting cycle, a high-early-strength concrete mixture was ideal.

To achieve a concrete mixture that met these requirements, more than 20 trial batches using local materials and admixtures were tested. A concrete mixture with an 8 to 9 in. slump to facilitate placement among congested reinforcement and a strength of 3500 psi within 12 hours was achieved.

The project's concrete supplier was located in Tucumcari, N.Mex., 25 miles south of the project site. To address the challenges of producing a special concrete mixture and transporting it over 30 minutes by truck, the contractor opted to set up a temporary batch plant at the project site. The on-site facility enabled easier communication and immediate response times, and provided concrete production that met the project specifications and demands.

Weather

Logan is located at an elevation of approximately 3800 ft above sea level, with a relatively dry climate. However, this region also sees a wide range of temperatures—with high temperatures above 100°F during the summer, and lows below freezing with periodic snow in the winter—and experiences extreme weather swings throughout the year. These drastic weather swings can occur within the same week or even within a 24-hour period.

Another weather obstacle for construction operations and schedule are high, gusty winds, which are common in this area. In March 2019, wind gusts of over 60 miles per hour derailed two dozen train cars on a rail bridge downstream from the project.

Design Support During Construction

Throughout the segmental construction, the project team—NMDOT, the design team, and the contractor—worked together to collaborate on the successful execution of vital construction operations.

Comprehensive Concrete Repair Plan

The design engineer and NMDOT coordinated with the segmental contractor to develop a comprehensive concrete repair plan that could be used as necessary throughout the construction to address repairs efficiently and effectively. In the early development of the optimal concrete mixture, concerns arose about slump, the effect of the dry desert climate on the mixture, and consolidation issues. Nondestructive testing, including the impact-echo method, was conducted in areas of significant repair to provide assurance and confidence regarding the quality of the final product.

Post-Tensioning and Grouting

During segmental post-tensioning operations, the design team and the contractor's engineer worked to fine-tune the design parameters based on field-verified values. The theoretical design prestressing parameters for

AESTHETICS **COMMENTARY**

by Frederick Gottemoeller

The challenge of inserting a new bridge into a spectacular natural scene is to design the bridge so that it complements, not clutters, the landscape. A new bridge will unavoidably become the center of attention, but it shouldn't fight for that attention. It should look like it has always been there. One way of accomplishing this is to fit the features of the bridge into the physical features of the site in an obvious and natural way. Here, the piers of the U.S. Route 54 bridge rest on the valley's slopes, out of the floodplain. Plus, the blocky, unadorned pier shafts give the piers an appearance similar to the blocky boulders of the nearby bluffs.

Simplicity itself usually helps fit a new bridge into a spectacular scene. That characteristic should extend to the basic geometry. Here the geometry consists of three horizontal curves: two at each end of the project to create the departure from U.S. 54, and a single, long curve in the middle. Its overall length matches the bridge to the scale of the desert landscape. Imagine how different the bridge would have looked if the designers had decided, for reasons of construction simplicity, to make the bridge straight. There would have been two short curves at each end of the bridge with a straight section in the middle, creating the "broken back curve" dreaded by highway engineers. The bridge would have had a choppy, cut-up appearance, completely out of place in this sweeping landscape.

Sloping the webs of the box girder is another direct, natural way of increasing the sculptural interest of the bridge. Simply by their geometric interplay with the haunches of the girders, the sloped sides create curved edges and varying widths for the girder soffits. Finally, deepening the haunches over the piers visually demonstrates the natural distribution of forces in the bridge, something I imagine the school kids in Logan grasped intuitively when they were given a chance to learn about the structure.

Closure beams were used to align and grade the cantilever tips and end spans for casting the closure sections. Photo: Malcolm International.

Formwork being set in preparation for casting the next box-girder segment. Photo: Malcolm International.

friction and wobble were compared with the actual observed coefficients measured in the field, with field friction tests and elongation compared with force results for each length of tendon. Additionally, post-tensioned tendon tensioning results were tracked to identify elongation trends during the progression of the cantilever segments. The cantilever tendon elongations during construction were compared to an allowable ±7% tolerance on the elongation difference between actual and theoretical values. This was valuable in recognizing trends to anticipate and understand the results and quickly address any anomalies.

The 38 top slab cantilever-construction tendons at each pier were composed of fifteen 0.6-in.-diameter strands. After the closure segments were cast, bottom slab continuity tendons—10 tendons with fifteen 0.6-in.-diameter strands in the end spans and 14 tendons with eighteen 0.6-in.-diameter strands in the center span—were tensioned. All continuity tendon ducts were sized for 19 strands as a provisional allowance. Six high-strength 1⅜-in.-diameter post-tensioning bars were also used in each end span.

In December 2019, the U.S. Route 54 project team gave presentations on the cast-in-place segmental concrete box-girder bridge to students of the Logan Municipal Schools. The students also participated in hands-on activities related to bridge construction. Photo: Logan Municipal Schools.

Before post-tensioned tendon grouting operations began, the U.S. 54 project team worked together with the American Segmental Bridge Institute (ASBI) to host ASBI Grouting Certification Training on site. This training provided valuable support and guidance ahead of the grouting operations, which were crucial to the long-term protection and durability of the structure.

Geometry Control

An essential aspect of cast-in-place segmental construction is geometry control. As box-girder segments are cast at either end of the cantilever, the geometry of the bridge is monitored with the ever-changing conditions, and adjustments are made with each casting. The geometry is constantly checked for agreement with the theoretical geometry, while also taking into account and correcting for the as-built conditions.

On the U.S. 54 project, geometry control was led by the contractor's construction engineer and survey team. Geometry-control quality assurance was performed independently by the design team and verified in survey checks. At the closure segments, the tips of the segments were aligned to within ½ in. of each other in both the vertical and horizontal geometry. This accuracy demonstrated the success of the geometry-control coordination and minimized cast-in correction forces.

"This Is *Your* Bridge"

During segmental construction, representatives from NMDOT, the design engineer, and the segmental contractor came together to give two interactive presentations for students in the Logan Municipal School District. At the time of the presentations, the residents of Logan had been observing the progress of the segmental structure for over a year. The students and school staff were therefore brimming with questions and eager to learn about the project. These educational presentations provided an opportunity to share project information and technical design facts, demonstrate career paths in engineering and skilled trades, and help the students develop a sense of ownership of their community's exciting new bridge. After the presentations, the students participated in hands-on activities to learn more about different bridge construction elements, including a tendon anchorage assembly, a reinforcement cage, concrete cylinders, personal protective equipment, and design drawings.

Conclusion

The U.S. 54 Canadian River Bridge project team worked in harmony with the rural and sensitive environment at the site to develop a bridge that is well suited to its beautiful and unique location. Segmental construction was completed in July 2020. Following the completion of a polymer concrete overlay placement, approach roadway, and tie-in to the new river crossing, traffic will be moved to the new structure and the existing steel deck truss bridge will be removed.

Nyssa Beach is a structural engineer and project manager with Jacobs Engineering in Denver, Colo. Jeff Mehle is a partner and principal engineer for McNary Bergeron & Associates in Broomfield, Colo.

Georgia DOT's Roadside Decked Beam ABC Project

by Jim Aitken and Kevin Kahle, CHA Consulting Inc.

Henry County, Ga., 30 miles south of Atlanta, is a rapidly growing county of over 230,000 residents. Many Henry County residents commute to the Atlanta metropolitan area, so the network of arterial and collector roads providing north-south connectivity is especially important. Blackhall Road is a two-lane arterial offering convenient access to Interstate 75 for the subdivisions in Lake Spivey and is also a heavily used school bus route for 30 buses on typical school days.

Blackhall Road crosses Rum Creek just downstream from, and generally parallel to, the dam impounding 600-acre Lake Spivey. The existing two-lane bridge, constructed in 1961, was designed for an H15 truck. The Georgia Department of Transportation (GDOT) decided to replace the bridge and approach roadway because of substandard geometrics and posted weight restrictions. A contract for design of the replacement bridge was awarded in 2014.

Aerial view of the completed bridge open to traffic. The dam and spillways for Lake Spivey, which limited the options for construction, are to the right of the photograph. Photo: C.W. Matthews Contracting Co. Inc.

The proposed new bridge was 195 ft long with a three-span configuration (60-93-42 ft). The bridge and roadway section consisted of two 11-ft-wide lanes with 8-ft-wide shoulders and concrete barriers on each side of the bridge. The bridge substructure consisted of concrete hammerhead piers on pile-supported foundations and pile-supported abutments. The roadway profile grade was raised 7 ft to improve stopping-sight distance in the vertical sag curve, and a 93-ft-long main span over the creek channel was required to satisfy GDOT hydraulic requirements.

Public Outreach

The concept design was completed in summer 2016, and the designers and GDOT reviewed plans with stakeholders, including schools, homeowners associations, elected officials, and the Lake Spivey Civic Association, which owns and maintains the Lake Spivey Dam. The public reaction was mostly positive, but stakeholders expressed major concerns about closing Blackhall Road and detouring traffic for up to 12 months. In response, GDOT agreed to

profile

BLACKHALL ROAD AT RUM CREEK / HENRY COUNTY, GEORGIA

BRIDGE DESIGN ENGINEER: CHA Consulting Inc., Atlanta, Ga.

PRIME CONTRACTOR: C.W. Matthews Contracting Co. Inc., Marietta, Ga.

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

Completed decked beam units in the staging area. The bracing can be seen in the background. Photo: CHA Consulting.

Aerial view of the on-site staging area showing completed decked beam units for spans 2 and 3. Photo: C.W. Matthews Contracting Co. Inc.

revise the design to shorten the duration of the road closure.

Traffic Maintenance Challenges

Maintaining traffic during construction proved to be a major challenge due to site constraints. The proximity of the dam and spillways eliminated the possibility of a temporary or permanent alignment shift to the west, and impacts on residential properties, including displacements and loss of public support, made the construction of a temporary or permanent alignment to the east impractical. The bridge would need to be replaced on the existing alignment with a temporary detour. GDOT and the designers determined that a six-month road closure would provide enough time for construction using traditional methods. However, the public still had concerns about the impacts of a six-month closure of Blackhall Road.

Accelerated Bridge Construction Solution

To further reduce the duration of the construction schedule, GDOT implemented an accelerated bridge construction (ABC) solution. Nearby residences and the dam spillway eliminated the possibility of using lateral slide-in ABC techniques, where the new bridge would be constructed adjacent to the existing bridge and rolled into place when finished. Instead, the decision was made to construct the bridge superstructure in on-site laydown yards and move the bridge superstructure to its permanent location during the closure of Blackhall Road. GDOT and the Federal Highway Administration determined this ABC approach would reduce Blackhall Road's closure to just 60 days.

The designers evaluated ABC solutions based on GDOT's preferences for a durable prestressed concrete superstructure that minimizes future maintenance and eliminates the need for painting. Several alternatives, including Northeast Extreme Tee (NEXT) precast concrete beams and decked bulb-tee beams, were considered. The

An 800-ton crane placing a 93-ft-long decked beam unit for the main span. Photo: C.W. Matthews Contracting Co. Inc.

GEORGIA DEPARTMENT OF TRANSPORTATION, OWNER

BRIDGE DESCRIPTION: Three-span, 195-ft-long bridge constructed using prestressed concrete decked bulb-tee units and accelerated bridge construction techniques. The three-span configuration has 60-, 93-, and 42-ft-long spans.

STRUCTURAL COMPONENTS: Fifteen prestressed concrete 54-in.-deep decked bulb-tee beams with a composite 8¼-in.-thick precast concrete deck, ultra-high-performance concrete closure pours, two cast-in-place concrete semi-integral end bents, and two cast-in-place concrete hammerhead intermediate bents founded on cast-in-place concrete pile caps supported by steel H-piles

BRIDGE CONSTRUCTION COST: $2.04 million

Decked beam unit being erected into final location at end span. Photo: CHA Consulting.

shipping weight for NEXT beams for the 93-ft-long center span was a major factor in the design of the bridge and would have required permits and route planning from GDOT. Decked bulb-tee beams had a weight advantage over the NEXT beams, as a bulb-tee section could be cast at a precast concrete plant and transported to the site without the composite deck. Once the bulb-tee beams were delivered, the deck would be cast atop the beams in the on-site laydown yards. The decked beams would then be erected into their permanent locations and concrete would be placed in the joints between the decked beams.

On-Site Staging Areas

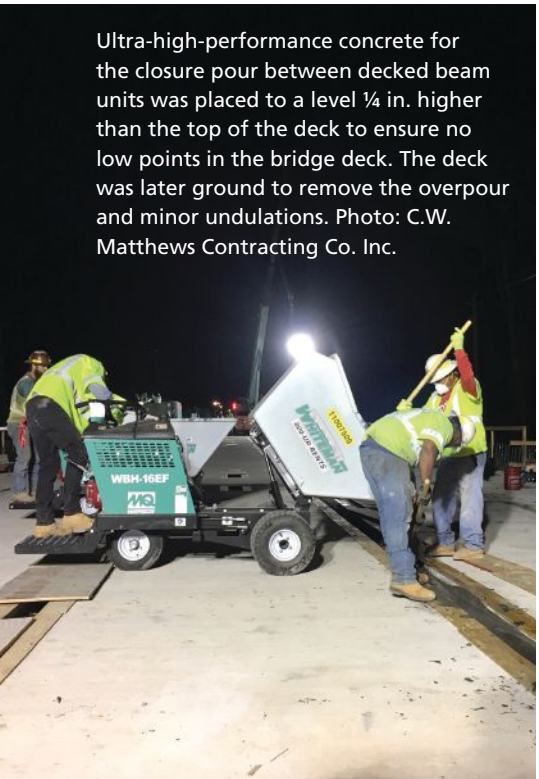
Staging areas were located on each side of Rum Creek, within 500 ft of the proposed bridge. The staging areas were large enough to construct up to two bridge spans, including the 93-ft-long center span. False bents that matched the vertical alignment and cross slope of the proposed bridge superstructure were constructed in the staging areas. The 54-in.-deep bulb-tee (BT-54) beams were then erected on the false bents. Next, the decked beam units were created by placing an 8¼-in.-thick concrete bridge deck on each beam with 9-in.-wide longitudinal gaps for closure pours between the decked beam units. Each unit consisted of a BT-54 beam with either a 7-ft 9-in.-wide (interior) or a 7-ft 6-in.-wide (exterior) composite bridge deck. This method allowed most of the bridge superstructure to be constructed before closing Blackhall Road, significantly reducing the road closure duration. Casting the deck on the girders while in the staging area also eliminated the common problem of profile management created by differential cambers in decked bulb tees.

The two staging areas were incorporated into the plans as temporary easements. Span 1 was constructed on the south side of Rum Creek; spans 2 and 3 were constructed in a larger staging area on the north side. Coordination with utility owners was required to eliminate overhead conflicts at the staging areas and along Blackhall Road between the staging areas and the proposed bridge. Overhead power lines were temporarily buried to allow the operation of cranes in the vicinity of the staging yards. With the staging areas cleared and utilities relocated, false bents consisting of driven steel H-piles and steel caps were constructed within the staging areas while Blackhall Road remained open. The longitudinal grade and deck cross slope were replicated through variable heights of timber dunnage placed atop the steel false bents. The permanent elastomeric bearings and beveled steel shim plates were placed on the timber dunnage to support the bulb-tee beams.

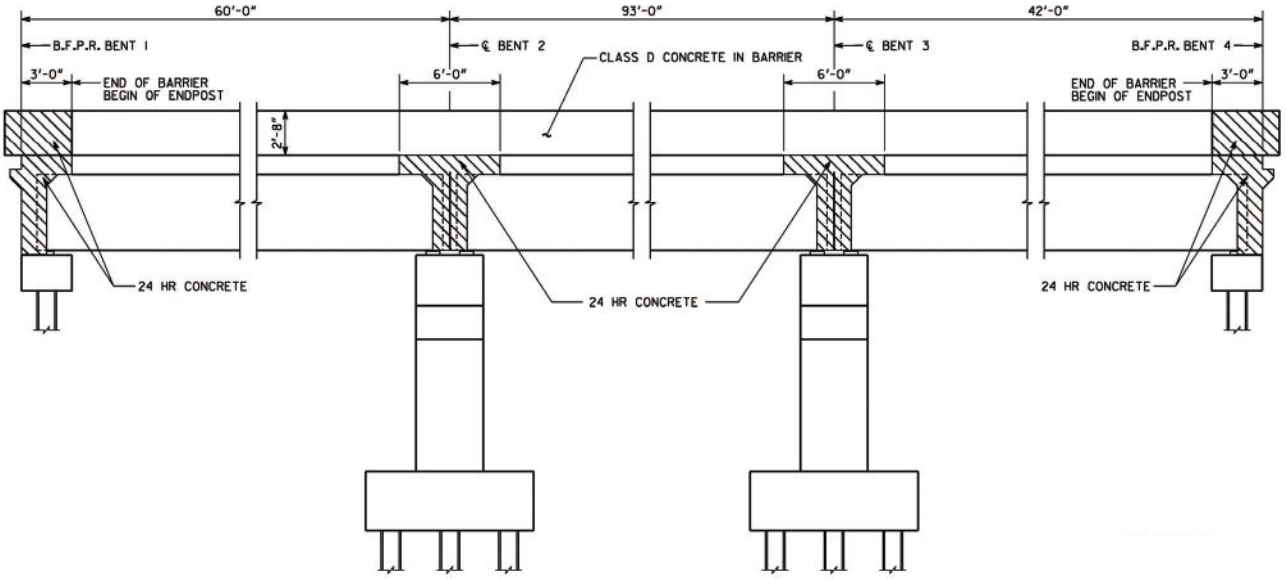
Decked Beam Unit Construction

BT-54 beams were used for all three spans for consistency and ease of construction. Once the BT-54 beams were

Ultra-high-performance concrete for the closure pour between decked beam units was placed to a level ¼ in. higher than the top of the deck to ensure no low points in the bridge deck. The deck was later ground to remove the overpour and minor undulations. Photo: C.W. Matthews Contracting Co. Inc.



The completed bridge open to traffic. Only a 60-day road closure was necessary. The ultra-high-performance concrete closure pours and pour backs at lifting loops are visible on the surface of the deck. Photo: CHA Consulting.



Elevation view of final structure. Field-placed concrete for end walls at abutments and edge beams at piers is hatched and denoted as "24 hr concrete." Figure: CHA Consulting.

delivered to the respective staging areas and erected and secured atop the false bents, intermediate steel diaphragms were installed at midspan to serve as bracing during casting of the deck. Steel diaphragms were selected because they could be used both as temporary bracing for the beams before and during deck placement in the staging yard as well as in the permanent condition with the structure in its final location.

The 8¼-in.-thick deck was designed using 4000-psi concrete. Polystyrene foam was used to create blockouts for the 9-in.-wide longitudinal closure strips between the decked beam units. The tops of the foam blockouts were recessed ½ in. below the finished deck to allow a skim pour of concrete to be cast over the blockouts. The skim pour achieved deck cross-slope continuity without requiring additional grinding of the decked beam units.

The bridge screed was set to match the deck profile at the permanent bridge location. The contractor was required to use a vibratory screed to finish the deck due to the presence of the closure pour blockouts and the lifting loops, which were needed to move the decked beam units into the final position. The bridge deck was placed in a single, continuous placement for each of the three spans. When the deck was cured, the skim pour material above the blockouts was removed by making shallow saw cuts along the length of the pour strips and chipping out the layer of concrete above the foam. The deck and overhang forms, including the foam blockouts, were then removed.

Erecting a Bridge in 60 Days

Once the decked beam units were substantially completed, Blackhall Road was closed to traffic. The existing bridge was demolished, and the new permanent bridge foundations and substructure units were constructed.

The decked beam units were erected over a three-day period. The steel diaphragms and elastomeric bearings were reinstalled at the permanent bridge location. The weight of the decked beam units was by far the most significant challenge in erecting the bridge. Each decked beam for the center span weighed over 180,000 lb. Two small cranes were considered to lift the decked beam units, but there was not sufficient room to construct crane pads for them. Instead, the contractor used one 800-ton crane and a temporary bulkhead built up to the edge of bent 3 to provide maximum lifting capacity. With the decked beam units erected, edge beams and end walls were promptly formed and cast with accelerated-strength concrete that achieved its 4000-psi compressive strength in 24 hours. The 9-in.-wide closure pours were filled with ultra-high-performance concrete (UHPC) supplied by LafargeHolcim. The challenges of working with the UHPC included mixing on site and properly sealing the formwork. This proprietary material was significantly more expensive than standard ready-mixed concrete and required additional lead time to procure. Topside forms were used to allow the top of the closure pour strips to be ¼ in. higher than the top of the deck to ensure no low points in the bridge deck. The deck was ground to remove the ¼ in. overpour and minor undulations. After deck grooving and completion of the roadway approaches, the road was reopened on schedule after a closure of only 60 days.

Conclusion

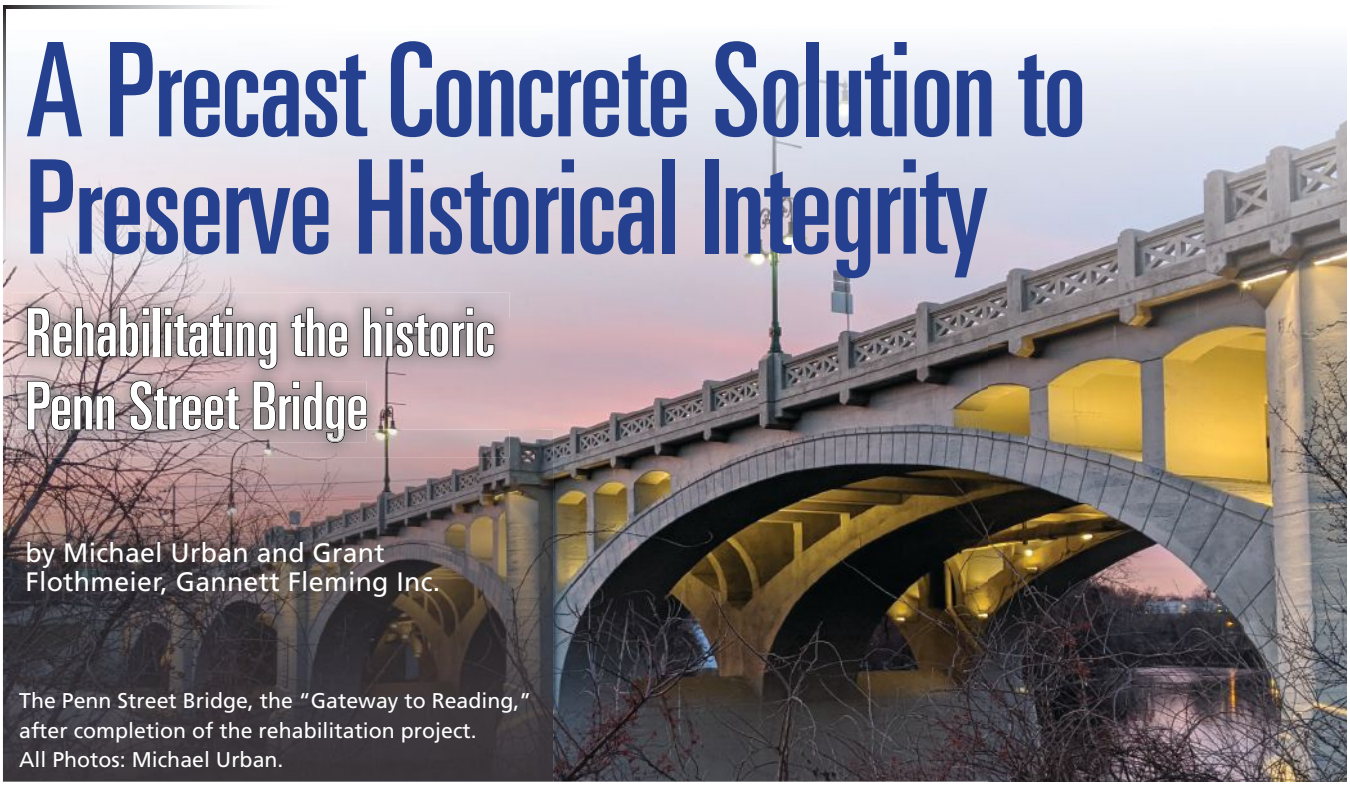
This project is an excellent example of an ABC application using readily available precast concrete beams. The method helps contractors complete bridge construction more efficiently by working away from traffic, and provides a schedule advantage over traditional deck construction. Typical prestressed concrete decked beams require special detailing because the camber of the beams is not usually consistent with the roadway profile. Deck cross slopes and differential camber can also be problematic. To address these issues, variable top flange thickness or variable thickness overlays are sometimes required. The decked bulb-tee units in this project were built with a standard beam haunch, allowing the profile and cross slope to be cast into the deck, significantly improving geometry control for the finished product. This concept can be applied to a wide range of ABC projects constructed with prestressed concrete bridge elements.

Jim Aitken and Kevin Kahle are senior project managers with CHA Consulting Inc. in Atlanta, Ga.

A Precast Concrete Solution to Preserve Historical Integrity

Rehabilitating the historic Penn Street Bridge

by Michael Urban and Grant Flothmeier, Gannett Fleming Inc.



The Penn Street Bridge, the "Gateway to Reading," after completion of the rehabilitation project. All Photos: Michael Urban.

Located on the Schuylkill River, the city of Reading, Pa., with a population of more than 88,000, is bustling with industry, art, and culture. The city's residents are very proud of its deep history and love its five large, open-spandrel concrete arch bridges that cross the river. The jewel of these bridges is the century-old Penn Street Bridge, considered the "Gateway to Reading."

The 1337-ft-long concrete viaduct, built in 1913, spans the Schuylkill River, Norfolk Southern railroad, Reading Area Community College, Front Street, a local trail, and a utility access road. The bridge is adjacent to a limited-access freeway, State Route 422, which is locally known as the West Shore Bypass. The bridge serves as the transition from the West Shore Bypass to the city via five open-spandrel arch spans, nine closed-spandrel arch spans, a concrete pile-supported slab structure, and a two-span concrete T-beam ramp attached to the main bridge.

Deterioration of the historic structure was noted in the bridge inspections and led to the evaluation of rehabilitation options. At the onset of the project, it was understood that the estimated $42.5 million bridge rehabilitation would require many carefully coordinated details to respect the historical integrity of the structure while extending its useful life. Priorities included incorporating as many of the original details of the bridge into the rehabilitation as possible while upgrading to current safety standards. Originally, the bridge carried both trolley tracks and a vehicular lane in each direction along with wide sidewalks. This configuration provided enough width to update the roadway cross section to today's standards without widening the structure. Character features such as detailing on the arch ribs, jack arches, and floor beam dimensions were relatively easy to replicate. However, today's standards for crashworthy barriers require a greater resistance to impacts, leaving the detailing of the existing barriers obsolete and not suitable for current requirements. By providing a new crashworthy barrier at the sidewalk curb, the original reticulated balustrade barrier could be recreated as part of the rehabilitation.

Preserving and Restoring Historic Details

The bridge is eligible for the National Register of Historic Places, so the rehabilitation had to balance preserving the bridge's character with bringing it up to current standards, as well as not precluding future work planned around the bridge. Due to the historic nature of the bridge, a goal of the project was to salvage as much of the original structure as possible. However, preliminary evaluation and

profile

PENN STREET BRIDGE OVER THE SCHUYLKILL RIVER / READING, PENNSYLVANIA

BRIDGE DESIGN ENGINEER: Gannett Fleming Inc., Audubon, Pa.

OTHER CONSULTANTS: Susquehanna Civil Inc., York, Pa.; Specialty Engineering Inc., Bristol, Pa.; CHRS Inc., Lansdale, Pa.; Liberty Environmental, Reading, Pa.; SoftDig, West Chester, Pa.; TRC Companies Inc., Export, Pa.

PRIME CONTRACTOR: J.D. Eckman Inc., Atglen, Pa.

Reusable formwork for a precast concrete railing panel. Slots in end and top forms are for extended J-bars. The panels were cast on site under a span that could be closed to the public. Tarps were hung to control the environment.

Two panel forms with epoxy-coated reinforcement installed, including J-bars extending from the top and sides of the panels to tie into the cast-in-place posts and rails. Inserts embedded in sides of the panel feet (see arrows) are for installing longitudinal reinforcement to tie panels to the sidewalk reinforcement.

Precast concrete panels ready for erection. J-bars extending from top and sides of panels tie into cast-in-place posts and rails. Pieces of the reusable two-part blockout forms are sitting on the panels.

hands-on inspection of the bridge found that some of the original elements, including the existing deck, sidewalks, floor beams, and reticulated balustrades, were deteriorated beyond repair and required replacement. The original arch ribs, closed-spandrel arches, and piers were in good condition considering their age and could be rehabilitated with only concrete repairs. The rehabilitation included a two-part, breathable, polymer-modified, cementitious base coat applied to the entire structure below the new deck, which provided texture to the new smooth concrete while filling in the deeper profiles of the weathered existing concrete. A breathable acrylic topcoat provided long-lasting protection and achieved a uniform color finish while adding corrosion protection.

Early on, the character-defining and aesthetic features of the bridge were identified: namely, its reticulated balustrades and outlooks. Originally, at each pier of the open-spandrel spans, an outlook extended from the sidewalk, allowing pedestrians to step away and enjoy the river scenery. However, in the 1950s, a major rehabilitation of the bridge included closing off these outlooks due to deterioration of the

"Panel feet" were incorporated into the design of the precast concrete panels so that the feet could be placed directly on the sidewalk and the panels required only temporary lateral support before casting the posts, curb, and top rail.

Typical precast concrete panel with cast-in-place posts, curb, and top rail. A conduit and a junction box for bridge lighting are visible.

PENNSYLVANIA DEPARTMENT OF TRANSPORTATION, ENGINEERING DISTRICT 5-0, OWNER

BRIDGE DESCRIPTION: 1337-ft-long, 14-span reinforced concrete open- and closed-spandrel arch bridge

STRUCTURAL COMPONENTS: New cast-in-place reinforced concrete deck, sidewalks, and floor beams; precast concrete barrier panels with cast-in-place posts, curbs, and rails to replace existing reticulated balustrades; rehabilitated original arch ribs, closed-spandrel arches, and piers

CONSTRUCTION COST: $43.2 million

AWARD: 2020 American Society of Highway Engineers East Penn Project of the Year

piers. Likewise, renovations in the 1970s replaced the original obelisks, which initially supported gas lamps, with light poles and electric lighting.

Based on input from stakeholders, it was determined that these original features would need to be incorporated into the design of the rehabilitation. Although current standards for street and pedestrian lighting required a luminaire height that prevented the use of obelisks in the original locations, adding two obelisks to the eastern end of the bridge met design requirements while preserving the obelisk's original character and enhancing the entrance to the city. In addition, with the rehabilitation and strengthening of the piers, the outlooks could be incorporated into the design.

Precast Concrete Barrier Panels

One of the major character-defining features of the Penn Street Bridge is the reticulated balustrade barrier. Each panel of the barrier consists of three X-shaped balustrades typically separated by a 1-ft 6-in.-wide post as well as a 4-ft-wide post at the midspan of the arches and at the piers. Some of these 4-ft-wide posts also support the new light poles. On the exterior face, post details extend below the soffit of the deck, breaking up the horizontal line at the bottom of the soffit. Conduits and junction boxes for the above- and below-bridge lighting were required to pass through the barrier curb along the length of the bridge.

In addition to the detailed requirements for the barriers, construction duration was also a key component of the rehabilitation. Penn Street serves as the entrance to downtown Reading, and construction staging already required reducing the four-lane bridge to three lanes during construction. It was critical that a tight schedule be maintained to avoid delaying any of the six construction stages.

Outlooks at the river piers, which were closed off during rehabilitation in the 1950s, were reconstructed as a part of this project.

The ornate detailing of the barriers required a construction technique that would satisfy the time constraints and provide a quality product worthwhile of the historical rehabilitation. Due to the X-shape within the panels, using typical cast-in-place construction could potentially lead to poorly consolidated concrete, resulting in unnecessary patching or the need for horizontal construction joints.

To eliminate these potential construction issues, the new barriers were designed using a combination precast concrete and cast-in-place system, with precast concrete panels for the X-shaped balustrades designed specifically to minimize construction time and provide a quality finish to the barrier. The panels varied in length from 10 ft 1 in. to 9 ft 3 in. along the length of the bridge, with smaller panels used around the outlooks. Each panel was approximately 3 ft high and 6 in. thick. The posts, curbs, and rails of the barrier were cast in place around the precast concrete balustrade panels to give the barrier its final appearance. This system enabled a high quality of construction, while providing for the conduits and junction boxes and creating a strong connection between the deck and barrier.

Because a crashworthy barrier is provided at the sidewalk curb, the reticulated balustrade barriers did not have to be designed to resist crash loads and were therefore designed to resist pedestrian and wind loads. No modifications to the original geometry of the reticulated balustrade barriers were required to resist these design loads. With a minimum height of 45 in., the original reticulated balustrade barriers also met current height requirements for pedestrian railings. To allow for any movement, vertical expansion/deflection joints were provided in the barrier, where they were placed inconspicuously adjacent to the posts.

Construction

The need to anchor the precast concrete panels to the deck while also limiting the required support when casting the posts, curbs, and rails led to the addition of "panel feet." The feet were placed directly on the sidewalk such that the panels required only temporary lateral support before the posts, curb, and top rail were cast. Inserts were embedded in the sides of the feet to receive longitudinal reinforcement that tied to hooked reinforcing bars projecting from the sidewalk between the panel feet. The height of the foot was based on the dimensions of the junction box that had to be positioned between the panel feet at certain locations. Epoxy-coated reinforcing J-bars extended from the top and sides of the panels to tie into the cast-in-place posts and rails. The J-bar provided adequate development for the reinforcement to secure the panels in place.

During construction, the contractor opted to cast the panels on site, using the space below one of the closed-spandrel spans that could be closed to the public. Tarps were hung to provide the contractor with a controlled environment and enough space to cast several panels at a time. Once casting was completed, the same area was used to store the panels until they were ready for installation.

As required by the tight schedule, the efficiency of having the panels on site and ready to erect omitted the time-consuming steps of forming, placing, and curing the concrete for the intricate geometry of the panels in place. In addition, defective castings would not slow the construction process.

An Impressive Gateway

The rehabilitation of the Penn Street Bridge was completed in 36 months, which was four weeks ahead of schedule, and within 2% of the original bid, a credit to all those who worked together on every detail. From beginning to end, the project team focused on the details so that the "Gateway to Reading" could be restored to its former glory and reestablish itself as the jewel of Reading's Schuylkill River bridges.

Michael Urban is a senior engineer and Grant Flothmeier is a project engineer, both with Gannett Fleming Inc. in Valley Forge, Pa.

Portland-Limestone Cement for More-Sustainable Concrete

by Jamie Farny, Portland Cement Association

Like most industries today, the construction industry is concerned about atmospheric carbon dioxide (CO_2). Responding to the need for more-sustainable building materials, cement manufacturers in the United States produce several formulations of lower-carbon, blended cements called portland-limestone cements (PLCs). Because these materials may be less familiar to some segments of the construction industry, the Portland Cement Association has created a campaign to raise awareness of PLCs and the benefits they offer.

The centerpiece of the campaign is a new website: www.greenercement.com. It provides architects, specifiers, and others with practical information on how to easily implement PLCs to create durable, resilient concrete.

PLCs are engineered with a higher limestone content (5% to 15%) than portland cement (maximum 5% limestone) to reduce the carbon footprint of concrete by about 10%. Cement producers optimize PLCs to make them easy to use, and test them to ensure they meet the requirements of ASTM C595.[1] PLCs perform just like ordinary portland cement and can be substituted for it in a concrete mixture at a 1:1 replacement level, using the same specifications, the same mixture proportions, and the same delivery and installation chain.

Because so much concrete is placed each year, even small changes to its formulation can have dramatic effects on the construction industry's annual carbon footprint. Modifying a concrete mixture to replace materials that have a higher carbon footprint with lower carbon footprint ingredients is an effective strategy. PLCs offer an easy way to accomplish this, much like fly ash and slag cement have been used for decades. And concrete mixtures designed with PLCs are compatible with all supplementary cementitious materials, so substituting PLC for ordinary portland cement does not preclude the use of any other carbon-footprint-reduction strategy a designer chooses to employ.

PLCs have been subjected to extensive industry testing, both in the United States and elsewhere. The body of research points toward positive results: Concrete mixtures with PLCs require little to no adjustment to achieve similar placing and handling characteristics and durability test results as traditional concrete mixtures. And experience demonstrates that these materials are suitable for transportation infrastructure; several states have been placing PLC concrete pavements for more than a decade. PLC concretes are appropriate for structural members of any type or size, so they are good for every aspect of bridge construction—from the deck down to the foundation—even including geotechnical work that might be needed to improve soil conditions.

The cement industry has made great strides with other agencies toward the acceptance of PLCs, making it easy to transition to environmentally friendlier concrete. Just like portland cement and other blended cements, PLCs are recognized by the American Concrete Institute's *Building Code Requirements for Structural Concrete* (ACI 318-19) *and Commentary* (ACI 318R-19)[2] and *Specifications for Concrete Construction* (ACI 301-20).[3] And beyond ACI standards, PLCs are recognized by the International Code Council, the Federal Aviation Authority, and the American Institute of Architects' MasterSpec.

Since the 1970s, improvements to cement manufacturing in the United States have resulted in a more than 40% decrease in production energy while also reducing CO_2 emissions. As climate issues and carbon footprint come to the forefront in material selection criteria, PLCs and other blended cements give designers an easy way to lower the carbon footprint of concrete by about 10%, providing sustainable and resilient construction while helping to address global climate challenges.

References

1. ASTM International. 2020. *Standard Specification for Blended Hydraulic Cements*. ASTM C595-20. West Conshohocken, PA: ASTM International.
2. American Concrete Institute (ACI) Committee 318. 2019. *Building Code Requirements for Structural Concrete* (ACI 318-19) *and Commentary* (ACI 318R-19). Farmington Hills, MI: ACI.
3. ACI Committee 301. 2020. *Specifications for Concrete Construction* (ACI 301-20). Farmington Hills, MI: ACI.

Jamie Farny is the director, building marketing, for the Portland Cement Association in Skokie, Ill.

Transferring the Body of Knowledge Through an Institute Publication

Inside the ASBI *Construction Practices Handbook*

by Tim Barry, RS&H Inc.

In 2019, the American Segmental Bridge Institute (ASBI) published an update to one of its most valued publications, *Construction Practices Handbook for Concrete Segmental and Cable-Supported Bridges.*[1] This third edition of the handbook is a "how-to" guide for the industry and a comprehensive summary of the important aspects of concrete segmental bridge construction. It examines the different types and methods of concrete segmental construction along with important details used in this unique construction technology. Because the industry has evolved since the previous edition was published in 2008, the handbook required a complete overhaul to update information regarding new specifications and include new techniques, new procedures, up-to-date photographs and diagrams, and current industry resources.

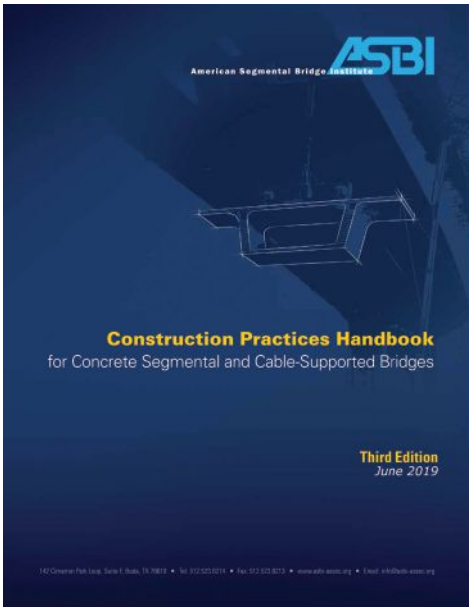
The handbook is a comprehensive examination of all aspects of concrete segmental bridge construction. It includes valuable information on the unique characteristics of concrete segmental bridge technology. There are chapters on cable-stayed, span-by-span, balanced-cantilever, and incremental-launch erection methods including both precast and cast-in-place concrete segments. The manual dives deeper into the details of concrete segmental bridge construction with chapters on segment casting yard setup and operation, specialty erection equipment, bridge bearings and expansion joints, geometry control, post-tensioning and grouting details, and overall project management and inspection. Included throughout the manual is guidance on best practices and, of course, lessons learned.

Industry Expertise

The Construction Subcommittee responsible for producing the handbook includes segmental subject matter experts representing academia, the contracting community, owners, designers, construction managers, and suppliers. The goal of the subcommittee was to ensure that the handbook was developed by the people who are immersed in this industry every day. With 22 members currently, the subcommittee is constantly seeking new members who are willing to join their colleagues in advancing the success of the segmental bridge industry. This collection of talent ensures that the information is up to date and relevant in a constantly changing construction world. It is these professionals who are responsible for developing and reviewing the content in the handbook. The time and resources invested and the information collected are what make this an essential document for anyone involved in the construction, and even the design, of a concrete segmental bridge.

The Process

The handbook with its 17 chapters provides a comprehensive overview of segmental bridge construction and touches on all important details and techniques. It defines, identifies, and explains different techniques; explains the details of how segmental bridges are built; and identifies the proper terminology unique to the industry. For each chapter of the handbook, the subcommittee assigned one or more industry experts to review the content and highlight the most important elements that readers need to comprehend.



Once each chapter was fully reviewed and edited, it was then peer reviewed by an independent group of industry experts. Finally, the ASBI Executive Board reviewed the content before final publication. At each stage of review, individuals representing the best this industry has to offer provided their insight and expertise. It is the goal of the subcommittee and ASBI to provide the most up-to-date, relevant, and useful information available.

A Useful Tool

The handbook is a complete overview of segmental bridge construction. The target audience is as diverse as those who are responsible for its content. The handbook can be used by newcomers with zero industry experience as a "beginner's guide" to segmental bridges, or it can be used by seasoned professionals as a useful reference. With this current edition,

Precast concrete box-girder segments being erected for the Lesner Bridge project in Virginia Beach, Va. All Photos: RS&H Inc.

ASBI decided to make the digital version of the handbook free to download for the first time. This was a logical decision considering the reasons for which the handbook was produced, which are, according to ASBI's leadership, "to educate and inform the industry. We want this information out there and we want people to use it." ASBI hopes that this handbook achieves that goal.

Construction Practices Seminars

In addition to producing the handbook, ASBI holds a yearly Construction Practices Seminar that presents information from the handbook in a formal group setting. It is held in a different city each year and has been attended by hundreds of professionals. Billed as "Segmental Bridges 101," the seminar addresses both the basics and the details that make this technology unique. Many of the same experts who contributed the information contained in the handbook also present at the seminar. This provides an invaluable opportunity for attendees to further expand their knowledge by hearing industry experts explain current construction practices in greater depth. In addition to covering the content of the handbook, presenters are eager to share valuable lessons learned from years of project experience with the next generation of industry professionals. Another bonus for attending the seminar is that all attendees receive a printed copy of the handbook to take home. It is the desire of ASBI that all segmental bridge professionals have a copy of this handbook sitting on their desks.

Construction of the main span cantilevers for the first of three cast-in-place segmental bridges for the Wekiva Parkway project in Sorrento, Fla. (see the Project article in the Fall 2020 issue of *ASPIRE*®).

Along with everything else in our lives, COVID-19 has disrupted the Construction Practices Seminar. In the interest of continuing to educate the industry, ASBI held its first virtual seminar this past year as a three-week series of online presentations. Presenters still gave an overview of the handbook and provided their expertise and lessons learned, but in an online setting. According to ASBI executive director Gregg Freeby, "We felt it was important to continue the seminars in any way we could." Because ASBI values personal interactions and information sharing, in-person seminars will resume when conditions allow.

Improving the Industry

The mission of ASBI has remained steadfast since the institute's creation in 1989:

> To advance, promote, and innovate segmental bridging technology; share the knowledge; educate stakeholders; build professional relationships; and increase the value of our infrastructure by providing sustainable solutions.

The development and continual improvement of the *Construction Practices Handbook* and its accompanying yearly seminar is just one more way that ASBI continues to achieve that mission.

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Tim Barry is a vice president with RS&H Inc., in Virginia Beach, Va., and the chair of the ASBI workgroup that that produces the **Construction Practices Handbook.**

The Popular CRSI Rebar Reference Mobile App Continues to Evolve

by Dave Mounce, Concrete Reinforcing Steel Institute

The Concrete Reinforcing Steel Institute (CRSI) has entered the mobile app arena with a practical application designed to benefit the architectural, engineering, and construction communities. Introduced in 2019 at the World of Concrete tradeshow, the app quickly gained popularity among the contractor audience. The CRSI Rebar Reference mobile app was developed to augment the vast array of industry publications and resources that CRSI produces. Available for both Android and iOS mobile devices, the app serves as a ready reference guide for general information pertaining to steel reinforcement (rebar). Developed initially to be used in the field, its evolution has enabled it to cross over to the office environment.

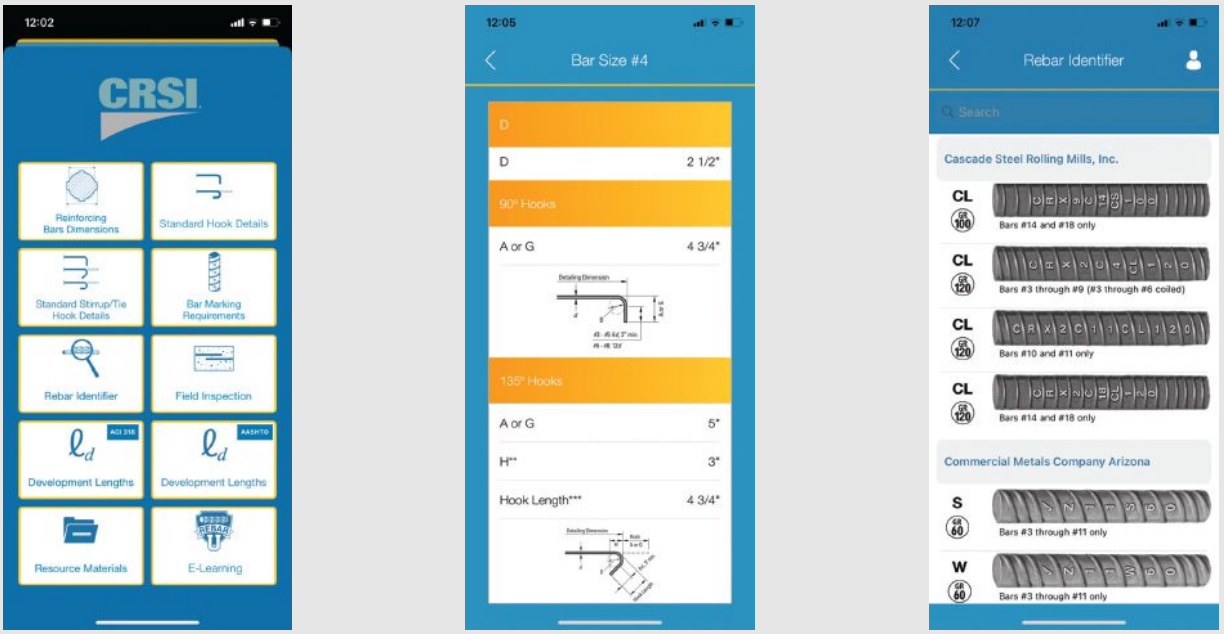
The free portion of the app includes a variety of commonly referenced reinforcing steel data such as

- ASTM reinforcing bar specifications on dimensions and deformations,
- an illustrated explanation of bar marks for reinforcing steel grades 40, 50, 60, 75, 80, 100, and 120,
- standard hook (90- and 180-degree) details, and
- standard hook (90-, 135-, and 180-degree) details for stirrups and ties.

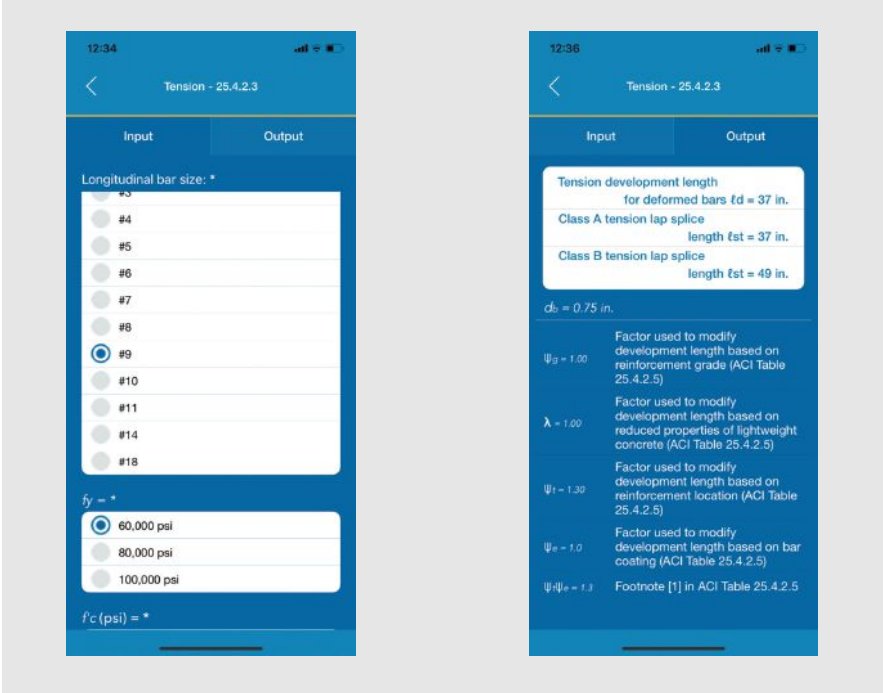
This information is based on CRSI's popular "bar cards," which are available for purchase from CRSI.

Since the app's release, CRSI has supported it with updates and new features. Additional free content added late last year included a Field Inspection module. The module provides information for cast-in-place concrete construction, including required cover depth, an explanation of tie wire along with diagrams of the types of ties used at steel reinforcing bar intersections, reinforcing bar placing tolerances and details, and lap splice information with illustrations of the types of lap splices.

Users can further enhance and customize the app on their devices by opting to purchase access to additional modules. The dynamic Rebar Identifier module is based on Appendix A of CRSI's industry-trusted *Manual of Standard Practice.*[1] The Rebar Identifier displays representations of reinforcing bars from all manufacturers



The CRSI Rebar Reference mobile app allows users to access free modules on reinforcing bar data and resource links, as well as the Rebar Identifier and Development Lengths calculators, which are additional modules that can be purchased. CRSI is focused on keeping the app relevant by updating the data regularly. All Figures: Concrete Reinforcing Steel Institute.



The Development Lengths calculators walk users through a simple process to calculate the development lengths and lap splice lengths for reinforcing bars.

of concrete reinforcing bars in the United States. Users can input or filter results by entering specifics such as bar size, grade, and/or type. Manufacturer information is also a swipe away for each bar image, making this tool handy both on site and during field inspections.

The most recent app features that can be purchased as an add-on are the Development Lengths calculators. These modules are available in versions for the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[2] and the American Concrete Institute's (ACI's) *Building Code Requirements for Structural Concrete* (ACI 318-14) *and Commentary* (ACI 318-14R).[3] In these modules, users are taken through a simplified process to calculate the development lengths and lap splice lengths for steel reinforcing bars of any grade, including deformed bars in tension (AASHTO LRFD 5.10.8.2.1, ACI 318 25.4.2.3, and ACI 318 25.4.2.4), standard hooks in tension (AASHTO LRFD 5.10.8.2.4 and ACI 318 25.4.3), deformed bars in compression (AASHTO LRFD 5.10.8.2.2 and ACI 318 25.4.9), and headed deformed bars in tension (ACI 318 25.4.4). The Development Lengths calculators are especially useful for determining the development and lap splice lengths when away from the office.

"The Rebar Reference app provides the Institute with a vehicle to engage our industry in a new and dynamic way," says Danielle Kleinhans, president and CEO of CRSI. "Along with the general data provided, we can also push important information out, including things like best practices that help standardize steel reinforcement issues within the industry. It's all about making our extensive technical knowledge more accessible and useful."

The app includes a link to CRSI's online Resource Materials portal where users can find free downloadable technical documents, design aids, and reference materials, as well as design guides and field publications for purchase. Also featured is Rebar-U, CRSI's eLearning portal for continuing education and professional development, which offers a variety of eLearning courses and webinar presentations on design and construction topics.

As part of its member committee structure, CRSI has a mobile app task group that focuses on improving and furthering the development of the Rebar Reference app. "The app allows CRSI to stay nimble and utilitarian with information that needs to be communicated to the AEC [architecture, engineering, and construction] community. For example, there has been a recent change to the way fabricated reinforcing steel is measured," Kleinhans states. "That kind of information needs to be readily available on the spot, especially when it comes to field inspection."

CRSI is excited to keep up the momentum around the app, including the addition of a Measuring Fabricated Bars module, which is due to be released in early 2021. CRSI is contemplating a desktop version to make the app—and the Development Lengths module specifically—more easily accessible to designers and detailers while at their workstations during the design and development phase. Additional content and modules will be developed based on input that CRSI's region managers acquire from industry contacts, along with recommendations from technical committees. With multiple new features identified, in development, or already implemented, CRSI is striving to ensure the app remains beneficial and relevant to its users into the future.

The Concrete Reinforcing Steel Institute (CRSI)

Founded in 1924, CRSI is one of the oldest trade associations in the United States. It is a technical institute and an ANSI-accredited Standards Developing Organization that stands as the authoritative resource for information related to steel-reinforced concrete construction. CRSI offers many industry-trusted technical publications, standards documents, design aids, reference materials, and educational opportunities to advance and standardize the reinforced concrete construction industry.

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Dave Mounce is the director of communications for the Concrete Reinforcing Steel Institute in Schaumburg, Ill.

Prescriptive versus Performance-Based Design

by Dr. Timothy Mays, The Citadel

One of the most common complaints I hear from practicing engineers is, "Why are codes and standards thicker every time they are updated?" This question is usually followed with some suggestion that engineers these days are doing too much engineering and too much computer modeling in their design offices. My counterintuitive response to this question is that we are not doing too much engineering, but that we are actually doing less engineering, and the unfortunate impact this has on our graduates may be long lasting. The more voluminous codes and standards we see each update cycle are more associated with a paradigm shift away from designing structures using engineering theory to prescriptively detailing structures to meet assumed performance objectives developed by code committees.

Probably the most significant negative impact of prescriptive codes is that they can be misused by a litigious society. It is much easier for attorneys to prove that the engineer missed one prescriptive code provision than it is to show that the engineer did not appropriately address an "engineer shall consider" requirement explicitly. I can't speak for every engineer out there, but I can promise that I have reviewed my fair share of structural drawings developed by others over the last 20 years, and I can't recall any set of drawings without what some could label a "missed prescriptive requirement" of some sort. So, is performance-based design a step into the future or a revisiting of the past? A few examples may help answer this question.

The seismic design of piles that are part of flat slab bridge bents is a classic example. Flat slab bridges are one of the most common bridge types built in my home state of South Carolina, as well as in many other parts of the country. The bridge superstructure is supported on pile bents that resist lateral forces like a moment frame in the transverse direction and like a series of cantilevered columns in the longitudinal direction. Force-based methods presented in the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[1] allow the engineer to design the piles for forces that are reduced from their maximum expected value by a response modification factor (*R* factor). The *R* factor is based on the expected seismic performance of the piles. Spiral reinforcement quantity, spiral placement, and other pile details are then prescriptively determined based on the seismic zone designated for the structure by the AASHTO LRFD specifications. In other words, moment demand in the piles is calculated based on a fictitious reduction in seismic demand, and the spiral reinforcement quantity and layout are entirely prescriptive and are tied to confinement levels that the spiral is assumed to provide to the core concrete inside the spiral. This type of design and analysis is very easy to perform, but there is no guarantee regarding the performance of the structure. It is not true to say that this type of design is conservative because there is a real possibility that some of the required prescriptive details and analysis requirements could diminish, rather than improve, the seismic performance of the structure.

States such as California and South Carolina have developed their own performance-based design criteria for flat slab bridge structures. In these states, the engineer is required to more

Performance-based design and advanced modeling of bridges requires true engineering to determine structural details. This often leads to more economical designs that may perform even better than designs using a prescriptive approach.

accurately assess the performance of the piles using nonlinear pushover models to determine the maximum force that can be delivered to the system, the actual locations of seismic damage, and a more realistic estimate of overstrength requirements for the rest of the structure. The benefit to this advanced modeling is that true engineering is used to determine the pile details, including spiral reinforcement quantity and placement. When flat slab bridges are designed using both prescriptive design and performance-based design and the resulting structures are compared, the structure designed using performance-based design not only performs better but also may be more economical.

Pile cap design is another example. Many engineers do not realize that everything needed to check pile caps for shear in accordance with the AASHTO LRFD specifications is, well, not included in the AASHTO LRFD specifications. Many pile configurations used for bridge columns create shear demands for which the provisions of the AASHTO LRFD specifications are inappropriate. Is deep beam analysis appropriate for these cases? Maybe. Should finite element models be used? Possibly. Can the Concrete Reinforcing Steel Institute's *AASHTO Design Guide for Pile Caps*[2] (see the Fall 2018 issue of *ASPIRE*® for details of the contents of this guide) address all limit states? Hopefully. Fortunately, or unfortunately, to design pile caps, engineers need to engineer a solution. Engineers designing pile caps need to think through the limit states and provide additional conservatism where they feel uncomfortable with the load path or specific details. This is what some engineers have told me engineering was like "in the good old days."

After more than 20 years of teaching the design of reinforced concrete beams and columns at The Citadel, I can say that I spend more time helping students learn prescriptive detailing requirements at low Bloom's taxonomy assessment levels than I do teaching the higher-level applications of material modeling necessary to perform performance-based design of these elements. This means students are spending too much time memorizing rules and less time understanding the principles behind the topic, which would be much more valuable when they are required to think outside the box. Unfortunately, we have no choice but to teach this way because engineers are expected to adhere to all prescriptive requirements in the codes. If there is some good news, it might be that prescriptive requirements are most difficult to accept when they are expensive to build and when it can be shown that they are not really applicable to the subject structure. Fortunately, prescriptive requirements are much more of an issue for building codes than bridge codes and standards. Many bridge design specifications and state-specific design guidelines are already moving toward performance-based design.


Pile cap foundation construction.

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Can Chemistry Solve a Serious Distress Problem That Has Plagued the Concrete Industry for 80 Years?

by Terry Arnold, Federal Highway Administration

The formation of alkali-silica reaction (ASR) gels in concrete structures (**Fig. 1**) is a sometimes serious, and often expensive to remedy, distress mechanism that has plagued the concrete industry since it was first discovered in France more than 80 years ago and in the United States in 1940.[1] The gels are formed by the reaction of alkalis in the cement with silica in certain aggregates. The gels can expand as they absorb moisture and exert sufficient force to crack the concrete. This often results in the costly replacement of concrete structures.

Many aggregates do not form ASR gels. For those that do, some aggregates form gels rapidly, with the gels appearing within the first few years of service life. Other aggregates react much more slowly, and it might take many years for the distress to manifest itself. In either case where the gels form, the result may be® the very expensive replacement of the structure (for more information about ASR, see the Perspective in the Summer 2018 issue and the Safety & Serviceability article in the Spring 2019 issue of *ASPIRE*®).

The chemistry laboratory at the Turner-Fairbank Highway Research Center (TFHRC) has been studying the chemistry of ASR gels for more than 10 years. The research has shown that the chemical structure of ASR gels varies and has a marked influence on the expansion rate.[2–4] This research has also shown, for the first time, that aluminum plays an important role in the expansion of these gels.

Figure 2 shows a scanning electron microscope image of a concrete section with an aggregate particle cracked by ASR. The ASR gel that has developed inside the aggregate is high in alkali content and is expansive. As the gel expands, it forces the crack in the aggregate to widen; ultimately this will crack the concrete. As the gel moves from the aggregate into the cement paste, it absorbs calcium and changes to a calcium-rich gel, which is not expansive.

A reliable method to accurately determine the likelihood that an aggregate will form expansive ASR gels has eluded the industry since the first test methods were put forward in 1947. Almost all the test methods—and there have been many variations over the years—rely on measuring the physical expansion of prepared mortar or concrete specimens immersed in sodium hydroxide solution at 85°C. The most commonly used tests are ASTM C1260,[5] which is rapid (taking only 16 days), and the more accurate ASTM C1293[6] method (**Fig. 3**), which takes one year. A newer test, AASHTO T 380,[7] is under evaluation by the industry. It too measures physical expansion of specimens.

The Turner-Fairbank ASR Susceptibility Test, T-FAST, is a test method being developed at the TFHRC chemistry laboratory that does not require the preparation of mortar or concrete samples, unlike the existing ASTM tests.[8] All that is needed is 5 grams of crushed aggregate (**Fig. 4**), which is 1000 times less than the aggregate needed for ASTM C1293. The test is carried out in a 50-mL test tube made of polytetrafluoroethylene, which is placed in an oven at 55°C for 21 days (**Fig. 5**). A typical laboratory oven can contain up to 300 of these test tubes.

At the end of the test period, the contents of the tube are filtered and the filtrate analyzed for silicon, calcium, and aluminum. The analysis is used to calculate the reactivity index (RI), which is the concentration of silicon divided by the

Figure 1. Cracking in a concrete bridge pier caused by alkali-silica reaction. Photo: FHWA.

Figure 2. Scanning electron microscope image of concrete containing alkali-silica reaction gel. Photo: FHWA.

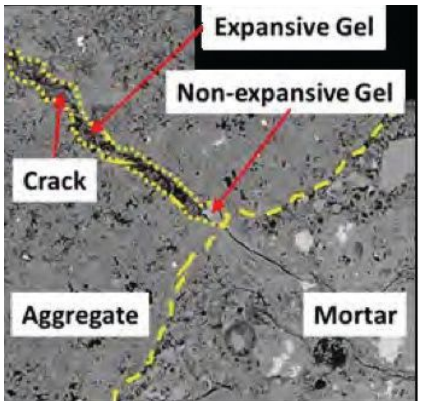

Figure 3. A 3 × 3 × 11.25-in. concrete specimen for an ASTM C1293 test. Photo: FHWA.

Figure 4. Five grams of crushed aggregate for T-FAST. Photo: FHWA.

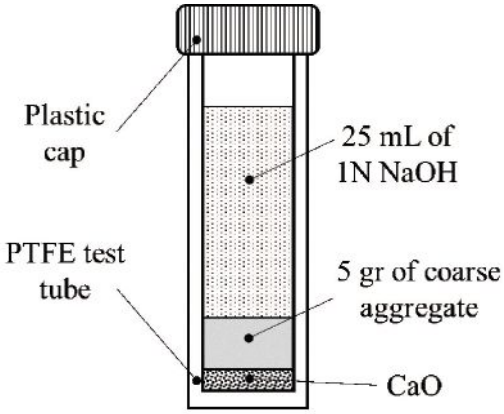

Figure 5. A 50-mL polytetrafluoroethylene (PTFE) test tube is placed in an oven at 55°C for 21 days to determine the reactivity index of the test aggregate in the T-FAST test. Figure: FHWA.

Table 1. Comparison of Test Data with Block Farm Data

Test	Agreement with Block Farm Data
ASTM C1260	68%
ASTM C1293	71%
AASHTO T 380	75%
T-FAST	88%
T-FAST (adjusted for effect of fine aggregate)	100%

Sources: Table created by FHWA with data from Ideker, J. H., A. F. Bentivegna, K. J. Folliard, and M. C. Juenger. 2012. "Do Current Laboratory Test Methods Accurately Predict Alkali-Silica Reactivity?" *ACI Materials Journal* 109 (4): 395–402, and Thomas, M. D., K. J. Folliard, B. Fournier, T. Drimalas, and S. I. Garber. 2013. *Methods for Preventing ASR in New Construction: Results of Field Exposure Sites.* FHWA-HIF-14-004. Washington, DC: Federal Highway Administration.

concentration of calcium plus aluminum in the filtrate. If the RI is greater than 0.45, then the aggregate is likely to form a gel.

The most reliable way of determining the alkali-silica reactivity of an aggregate is either by field experience, or by constructing farms of concrete blocks that are exposed to weather for a period of years (**Fig. 6**). The physical dimensions of the blocks are periodically measured to determine expansions.

Table 1 shows the correlation of current test methods with block farm results. T-FAST shows 88% agreement with block farm data. In addition to showing the importance of aluminum, the research at TFHRC shows the effect of fine aggregate on the formation of ASR gels. The fine aggregate used in the current tests is very often chosen because it is known to be nonreactive. However, it was found that the fine aggregate, while being nonreactive, can sometimes leach alkalis into the system, distorting the test results. If the effect of the fine aggregate is compensated for in T-FAST, there is 100% agreement with block farm data.

This research on ASR is still a work in progress, so references have not yet been published.[8] The next step is mitigation. If only reactive aggregates are available, how can they be used? Finally, the icing on the cake will be a method to evaluate not just the aggregate but the job-specific concrete mixture, so that everything that will be in the concrete is accounted for in the test.

Figure 6. Block farm of concrete specimens. The physical dimensions of the blocks are periodically measured to determine expansions. Photo: Massachusetts Department of Transportation.

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Baby It's Cold Outside (for Your Concrete Bridge)

by Dr. Kyle A. Riding,
University of Florida

Figure 1. Concrete surface with severe salt scaling caused by use of deicers. All Figures and Photos: Kyle A. Riding.

You have likely seen the signs that say "Bridge ices before road" near bridges. These signs serve to warn drivers of the potential danger ahead caused by black ice that is difficult to see. Clear pavements can lull drivers into thinking the roadway is free of ice before they drive over the bridge and its hidden dangers. To help mitigate this unseen danger, bridges can be heavily salted with rock salt or, in recent years, with concentrated brines that keep the roads clear of ice.

Just as it does for drivers, cold, wet weather poses hidden dangers for bridge structures and their owners. Most civil engineers are aware of the danger of damage to concrete bridges caused by freezing and thawing, and know that air entrainment is used in concrete as a mitigation measure for this hazard. It is less well known, however, that deicing salts can exacerbate damage from freezing and thawing and can cause two other types of deterioration, namely, salt scaling and calcium oxychloride formation.

Salt Scaling

Deicing salt scaling causes progressive flaking of the concrete surface, leading to a rough ride. Mild salt scaling can resemble an exposed aggregate finish, whereas more severe scaling such as that shown in **Fig. 1** can be more problematic for bridge durability. Saltwater can pond in the scaled areas, leading to further scaling damage and increased chloride penetration. Deicing salt scaling can occur on bridge decks, vertical surfaces of parapet walls or columns where water with deicing chemicals is splashed, or on bent caps below leaky joints. Like sodium chloride and calcium chloride deicers, magnesium chloride deicers can contribute to deicing salt scaling. Magnesium chloride salts also react chemically with the glue holding the concrete together, the calcium silicate hydrate, resulting in rapid deterioration.

Preventing Salt Scaling

Salt scaling occurs in bridges where the concrete near the surface has low strength or a bad air-void system. Air entrainment should be used and measured during construction. Low water–cementitious material ratios (*w/cm*) below 0.45, curing with either curing compound or extended wet curing, and good finishing practices have been shown to improve salt scaling resistance by improving the concrete surface strength.[1–4] Retempering concrete by adding water on the jobsite has been associated with some cases of salt scaling damage and should be avoided.[5] There has been a concern that high dosages of supplementary cementitious materials (SCMs) increase salt scaling damage; however, field studies have shown that concrete containing SCMs can do well in the field if it is well cured and not overworked during finishing, and not finished while there is still bleed water on the surface.[3,4] Machine-finished concrete tends to do better than hand finished concrete because more-aggressive finishing with multiple passes with a float, which often occurs with hand finishing, tends to coalesce air bubbles at the surface, causing a bad air-void system.[4]

Calcium Oxychloride Formation

Calcium oxychloride formation causes joint deterioration in concrete pavements in cold regions of the United States; see **Fig. 2** for an example of a joint in the early stages of deterioration. Calcium oxychloride forms when chlorides from deicing salts react chemically with calcium hydroxide and water, which can result in large expansion, cracking, and deterioration.[6] Calcium oxychloride formation increases with salt concentration and as the temperature drops below 40°F.[7] Calcium chloride deicing salts are the most destructive, followed by magnesium chloride salts, with sodium chloride showing only a mild risk.[6] Many salt suppliers use impure salts or add calcium chloride or magnesium chloride to salt to lower the ice melting temperature below that of sodium chloride.[8]

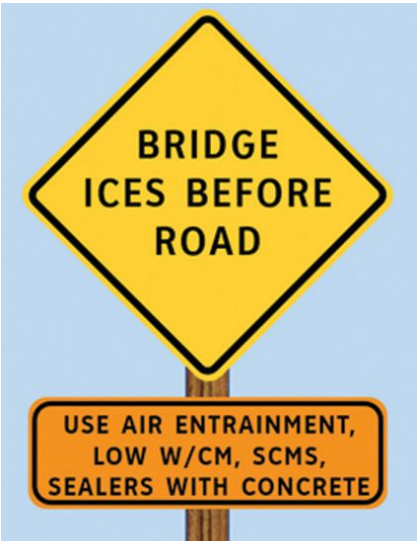
Figure 2. A concrete joint in the early stages of joint deterioration. A straight edge is used to show the expansion in the concrete near the joint caused by calcium oxychloride formation.

Instances of joint deterioration have significantly increased in the past 15 years because agencies have started to apply concentrated liquid salt brines before storms as an anti-icing treatment, which allows the dry concrete to absorb the saltwater. Joints, areas near drains, and the cold joint between the parapet wall and the deck are especially vulnerable to problems because saltwater can pond in these locations and concentrate when the water evaporates. Salts that penetrate the concrete are also hygroscopic (that is, they tend to absorb moisture from their surroundings), which causes the joints to dry at a much slower rate. Because concrete cold-weather related deterioration involves moisture, these locations that stay wet longer experience more deterioration than areas that dry faster.

Reducing Calcium Oxychloride Damage

If temperatures are not too far below freezing, the best way to reduce joint deterioration on bridges is to not use deicing salts that contain calcium chloride or magnesium chloride. Calcium oxychloride formation requires the presence of a chloride-containing salt and calcium hydroxide. Good drainage, joint sealant maintenance, use of concrete sealers such as soy methyl ester polystyrene blends, low *w/cm*, and SCMs help keep salts out of the concrete.[9] SCMs reduce the amount of calcium hydroxide available to form calcium oxychloride through dilution of the portland cement and the pozzolanic reaction. Use of Class F fly ash (ASTM C618[10]), slag (ASTM C989[11]), or metakaolin (ASTM C618[10]) as cementitious materials with portland cement replacement rates of 35%, 35%, and 12%, respectively, is needed to minimize joint deterioration damage.[12] For hand-finished concrete where there is a concern about using more than 25% fly ash, a ternary blend could be used.[1]

Figure 3. Proposed signage **during** *concrete bridge construction. When proper materials are used along with proper construction practices, damage caused by freezing and thawing, salt scaling, and calcium oxychloride is mitigated.*



Summary

Concrete bridges in cold climates can be very resistant to damage caused by freezing and thawing, salt scaling, and calcium oxychloride formation when proper materials are used along with proper construction and maintenance practices. Recommended construction practices include avoiding retempering or overfinishing the concrete, as well as using air entrainment, a *w/cm* below 0.45, SCMs to reduce permeability and calcium hydroxide content in concrete, and sealers in areas of concern like joints and around drains. To remind everyone involved in bridge construction about material needs and practices to prevent damage, I think we should consider installing signs that state "Bridge ices before road," when construction projects begin, instead of at completion, and we should add the new sign proposed in **Fig. 3**.

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CONCRETE BRIDGE PRESERVATION

CBP

Externally Applied Fiber-Reinforced-Polymer Composites

by Dr. Abdeldjelil Belarbi and
Dr. Lara Zerbe, University of Houston

The use of fiber-reinforced-polymer (FRP) composites as construction materials is increasing steadily. The application of externally applied FRP composites for the strengthening and rehabilitation of existing structures is an advanced technology that has gained considerable attention over the past three decades. Among design professionals, FRP composites are a preferred solution over traditional materials for applications such as rehabilitation and repair of existing structures because the composites provide significant advantages such as their light weight and ease of application. FRP composites also contribute to more durable, and hence more sustainable, construction than many traditional materials. Using FRP composites for external reinforcement is a complex process that requires a full understanding of the materials and the existing structural conditions, as well as accurate design and detailing practices. This article provides an overview of different FRP systems and their applications, limitations, and design considerations, and presents international design guidelines that have been driven by the state of the practice.

FRP Materials

FRP composites are advanced materials that consist of continuous fibers—such as carbon, glass, aramid, basalt, or natural materials—embedded in a polymer matrix. The fibers are the primary load-carrying components, whereas the polymer resin acts as a binder that transfers forces among the fibers and protects the fibers. FRP composites are available in various forms, including thin strips and bars that are both typically made by pultrusion, and flexible, dry-fiber sheets, which are typically saturated with resin on site during installation.

FRP composites are orthotropic materials with properties that can vary in different directions. Both unidirectional and multidirectional FRPs are available; unidirectional FRPs are the most common for structural applications. The overall properties of FRP materials depend on the mechanical properties of the fibers, mechanical properties of the matrix, the interaction between the fibers and matrix, and the method of manufacturing. Moreover, because the fibers are the main load-carrying components, the type of fibers, fiber-volume fraction, and orientation of fibers are key factors that dictate the stiffness and tensile strength of the material.

FRP materials exhibit a linear-elastic stress-strain relationship up to failure when loaded in direct tension, with no yielding or plastic behavior. This behavior is contrary to that of elastoplastic steel; therefore, standard methods used for

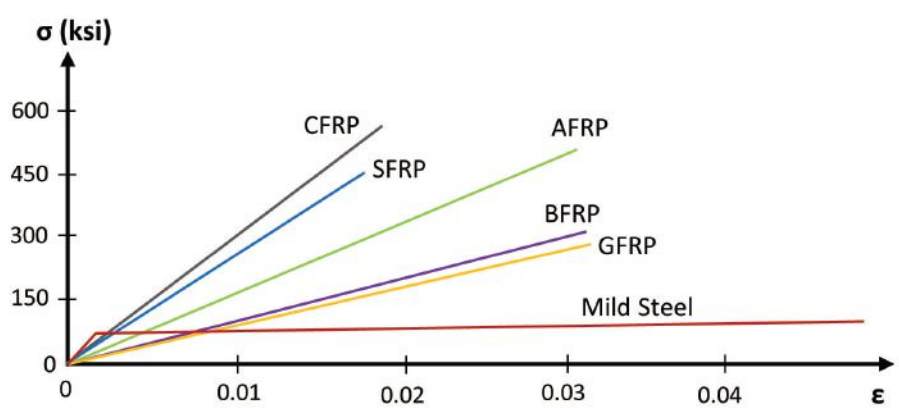

Figure 1. Typical stress-strain relationships of mild steel and different types of fiber-reinforced polymers. Note: AFRP = aramid-fiber-reinforced polymer; BFRP = basalt-fiber-reinforced polymer; CFRP = carbon-fiber-reinforced polymer; GFRP = glass-fiber-reinforced polymer; SFRP = steel-fiber-reinforced polymer. Source: Adapted from reference 2.

design to determine the amount of steel reinforcement do not apply to FRP reinforcement, where more complex procedures and design methodologies are involved. In general, FRP composites have a higher ultimate strength than steel but a lower modulus of elasticity and strain at failure. **Figure 1** shows typical stress-strain diagrams for different types of unidirectional FRP composites compared with mild steel reinforcement. The different types of FRP depend on the type of fibers and are noted on the figure; for example, carbon-fiber-based FRP composites are referred to as CFRP and glass-fiber-based FRP composites are referred to as GFRP. The modulus of elasticity can vary for the same type of FRP because the modulus is a function of the fiber modulus, matrix modulus, and fiber-volume ratio. The most common FRP systems for structural applications are CFRP composites because of their superior mechanical properties, including higher tensile strength, stiffness, and durability, compared with GFRP and other types of FRP composites.

FRP Strengthening Systems and Applications

The increased use of FRP materials in structural engineering for strengthening and rehabilitation of existing structures is mainly due to the advantages of FRP composites compared with traditional materials and techniques. These advantages include corrosion and chemical resistance, high tensile strength, design versatility, and, most importantly, their light weight and the variety of available forms (different sizes, geometry, and dimensions), which allow easy and rapid application of the system while minimizing labor costs and disruption of use, as well as enabling use in areas with limited access.

FRP systems have been successfully used for different strengthening applications such as flexural strengthening, shear strengthening, column confinement, and seismic retrofitting (**Fig. 2**). The most common types of FRP strengthening systems include surface-bonded FRP reinforcement (precured or cured on site) and near-surface-mounted FRP reinforcement. Each of these systems requires a specific installation and material selection process. Therefore, it is critical that engineers follow reliable guidelines when selecting suitable FRP systems for specific strengthening projects. For example, prefabricated (or precured) strips are typically stiff and cannot be bent; therefore, they are best suited for straight surfaces such as the top or bottom surfaces of slabs and beams. In contrast, flexible fabrics cured on site can be tailored to fit any geometry and can be adhered to straight surfaces or wrapped around almost any profile (**Fig. 3**). Near-surface-mounted FRP strengthening systems are suitable for applications where the FRP should be protected or if improved bond conditions between the concrete and FRP are necessary.[1]

Limitations and Design Considerations

Although FRP composites offer many advantages, these systems do have certain disadvantages and limitations

Figure 2. Strengthening of concrete bridge elements using carbon-fiber-reinforced polymer systems bonded to the concrete, which appear as black bands or wraps in the photo. Photo: Simpson Strong-Tie.

Figure 3. Flexible carbon-fiber-reinforced polymer sheets can be wrapped around and bonded to almost any concrete element profile. Photo: SURFPREP.

that should not be neglected by engineers. Examples of such limitations are the linear-elastic behavior of FRP composites, which leads to reduced ductility of the material; different thermal expansion coefficients for FRP materials compared with concrete; and their low maximum temperature threshold, specifically related to the epoxy resins, which could cause premature degradation in case of fire. Additionally, careful consideration should be given during design to determine reasonable strengthening limits for the FRP composite system. The design guides and codes described in the following section outline the required minimum capacity of the unstrengthened structural member and other guidelines regarding the condition of the structure. For example, the American Concrete Institute's (ACI's) *Guide for the Design and Construction of Externally*

Bonded FRP Systems for Strengthening Concrete Structures (ACI 440.2R)[1] recommends that the existing strength of the unstrengthened structure should be sufficient to resist a level of load as described by the following equation:

$$(\phi R_n)_{existing} \geq (1.1S_{DL} + 0.75S_{LL})_{\text{new}}$$

where

R_n = nominal strength of a member

ϕ = strength reduction factor

S_{DL} = dead load effects

S_{LL} = live load effects

Moreover, the cost of FRP materials is high compared with conventional materials such as steel. However, due to their high strength-to-weight ratio, the cost of using FRP systems can be favorable if the structure is properly designed, because they can be more cost effective over the life of the structure.

According to the *fib* (International Federation for Structural Concrete) Bulletin 90, *Externally Applied FRP Reinforcement for Concrete Structures*,[2] FRP materials for strengthening and repair applications should not be blindly used as a replacement for conventional materials. Instead, proper evaluation of their advantages and drawbacks, accurate design processes, a good understanding of the existing structural conditions, and appropriate detailing should all be considered when making the final decision regarding their use.

Guides and Codes for Externally Applied FRP Reinforcement

Several guides that address the design and construction aspects of externally applied FRP reinforcement systems for concrete structures have been published. In the United States, ACI published the latest edition of the previously mentioned *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures* (ACI 440.2R) in 2017.[1] Later, ACI published a code for the repair of existing structures, *Code Requirements for Assessment, Repair, and Rehabilitation of Existing Concrete Structures* (ACI 562),[3] which permits the use of FRP materials and refers to several documents by ACI Committee 440. In 2012, the American Association of State Highway and Transportation Officials (AASHTO) published *Guide Specifications for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements*,[4] which was intended to supplement the *AASHTO LRFD Bridge Design Specifications*.[5]

In 2019, *fib* published the aforementioned *Externally Applied FRP Reinforcement for Concrete Structures,*[2] which provides supplementary provisions for FRP-strengthened concrete structures. The bulletin discusses the design of reinforced concrete members strengthened with externally applied composite materials, as well as currently available composite materials, durability, and proper installation techniques. It focuses on strengthening and seismic retrofitting with FRP in the form of externally bonded or near-surface-mounted reinforcement and provides all necessary information to ensure the overall structural safety of the strengthened member.

Conclusion

Externally applied FRP reinforcement systems have gained significant popularity and have been successfully used to strengthen concrete structures including bridges, buildings, and tanks. Their advanced properties and design versatility have caught the attention of both design professionals and researchers, who have carried out several studies to demonstrate the material's efficiency and have contributed to the development of design guides and specifications. The higher cost of FRP materials is counterbalanced by lower life-cycle costs and reduced costs of labor, installation time, and equipment; as a result, using FRP materials is often more cost effective than traditional strengthening techniques. Finally, understanding the properties and limitations of FRP systems is an important step in developing adequate design solutions and using the proper system for each application.

References

1. American Concrete Institute (ACI) Committee 440. 2017. *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures*. ACI 440.2R-17. Farmington Hills, MI: ACI.

2. *fib* (International Federation for Structural Concrete). 2019. *Externally Applied FRP Reinforcement for Concrete Structures*. Bulletin 90. Lausanne, Switzerland: *fib*.

3. ACI Committee 562. 2019. *Code Requirements for Assessment, Repair, and Rehabilitation of Existing Concrete Structures (ACI 562-19) and Commentary*. Farmington Hills, MI: ACI.

4. American Association of State Highway and Transportation Officials (AASHTO). 2012. *Guide Specifications for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements*. Washington, DC: AASHTO.

5. AASHTO. 2020. *AASHTO LRFD Bridge Design Specifications*. 9th ed. Washington, DC: AASHTO.

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

IN THIS ISSUE

http://aspirebridge.com/magazine/2019Fall
The Marc Basnight Bridge at North Carolina's Outer Banks is mentioned in the Focus article on specialty post-tensioning contractor Schwager Davis Inc. on page 6. The bridge is 14,800 ft long with 11 spans of post-tensioned segmental concrete box girders and 71 spans of prestressed concrete girders. It was featured in Project and Concrete Bridge Technology articles in the Fall 2019 issue of *ASPIRE*®. Both articles can be found at this link.

https://dot.ca.gov/-/media/dot-media/programs/engineering/documents/seismicdesigncriteria-sdc/sdc20april2019final.pdf
The latest version of *Seismic Design Criteria* published by the California Department of Transportation is available for download at this website. The history and development of this document are presented in the Perspective article on page 10.

https://www.structuremag.org/wp-content/uploads/2014/08/D-GA-Blume-Weingardt-Dec-081.pdf
John Blume, who is recognized as the "Father of Earthquake Engineering," is mentioned in the Perspective article on the evolution of seismic bridge design at the California Department of Transportation on page 10. This is a link to a 2008 *STRUCTURE* magazine article about his contributions to seismic analysis and design.

https://www.youtube.com/watch?v=T9FjrtDlUZc
The New Mexico Department of Transportation created this video of construction activities on the U.S. Route 54 Canadian River Bridge project. The three-span cast-in-place, post-tensioned segmental box-girder bridge is featured in the Project article on page 24.

https://pahistoricpreservation.com/section-106-success-readings-penn-street-bridge
This is a link to a blog post that focuses on the preservation and community involvement aspects of the Penn Street Arch Bridge Rehabilitation project that is featured in the Project article on page 32. It also contains links to two aerial drone interactive 360-panoramic images of the bridge during construction.

https://www.asbi-assoc.org/index.cfm/resources/construction_practices_handbook
The American Segmental Bridge Institute's *Construction Practices Handbook for Concrete Segmental and Cable-Supported Bridges*, third edition, is a "how-to" guide for the industry and is featured in the Concrete Bridge Technology article on page 36. The handbook is available as a free download from this webpage.

https://www.tsp2.org/2013/10/04/alkali-aggregate-reactivity-aar-facts-book/
The FHWA article on page 42 discusses available tests for alkali-silica reaction as well as a new, faster test being developed by the Chemistry Research Laboratory at the Federal Highway Administration (FHWA) Turner-Fairbank Highway Research Center. The FHWA's *Alkali-Aggregate Reactivity (AAR) Facts Book* (FHWA-HIF-13-019) is available for download from this webpage.

https://www.fib-international.org/publications/fib-bulletins/externally-applied-frp-reinforcement-for-concrete-structures-detail.html
The Concrete Bridge Preservation article on page 46 provides an overview of externally applied fiber-reinforced-polymer composites and their applications, limitations, and design considerations. The article also discusses international design guidelines, including *Externally Applied FRP Reinforcement for Concrete Structures*, Bulletin 90, published by *fib* (International Federation for Structural Concrete). This webpage provides a link to the detailed table of contents and information for purchasing the bulletin.

www.pci.org/AnchoringToConcreteImp
As mentioned in the LRFD article on page 50, Article 5.13 of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications* has incorporated by reference the provisions for anchoring to concrete from the American Concrete Institute's *Building Code Requirements for Structural Concrete* (ACI 318-14) *and Commentary* (ACI 318R-14). Under the sponsorship of the National Cooperative Highway Research Program, PCI developed a five-part webinar series for bridge engineers on the requirements for designing, detailing, and installing concrete anchors. This webpage provides a link to access to a Dropbox folder that contains the recorded webinar series, course handouts, and additional resources.

http://resources.crsi.org/resources/rebar-reference-mobile-app
The Concrete Reinforcing Steel Institute has a free mobile app, "Rebar Reference," which can be obtained at this link. The app, available for both Android and iOS mobile devices, serves as a ready reference guide and provides general information pertaining to steel reinforcement. The features of the mobile app are described in detail in the Concrete Bridge Technology article on page 38.

OTHER INFORMATION

https://store.transportation.org/Item/CollectionDetail?ID=214
AASHTO recently published the *Guide Specification for Service Life Design of Highway Bridges*. This link leads to a webpage with a description of the publication, a link to the table of contents, and purchasing information.

https://store.transportation.org/Item/CollectionDetail?ID=215
AASHTO recently published the *Historic Bridge Preservation Guide*. This link leads to a webpage with a description of the publication, a link to the table of contents, and purchasing information.

Anchors in Concrete: Drawing Details to Identify and Procure Concrete Anchors via Specification Special Provisions—Part 3 of a four-part series

by Dr. Donald F. Meinheit

This article is part 3 in a four-part series addressing concrete anchorage in the reorganized Section 5 of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*,[1] published in 2017. At that time, AASHTO added Article 5.13, Anchorage to Concrete, to the design specifications. Part 1 of this series of articles (see the Summer 2020 issue of *ASPIRE*®) outlined a PCI program sponsored by the Transportation Research Board to educate bridge engineers on the implementation of the new concrete anchorage provisions adopted from the American Concrete Institute's *Building Code Requirements for Structural Concrete* (ACI 318-14) *and Commentary* (ACI 318R-14).[2] Part 2 (see the Fall 2020 issue of *ASPIRE*) focused on the qualification procedures for post-installed concrete anchors. Any anchor that is designed according to the provisions of AASHTO LRFD specifications Article 5.13 must be qualified, and its behavior must be shown to be consistent with the design provisions in the code. This article reviews the mechanisms that are available to procure concrete anchors once they have been designed.

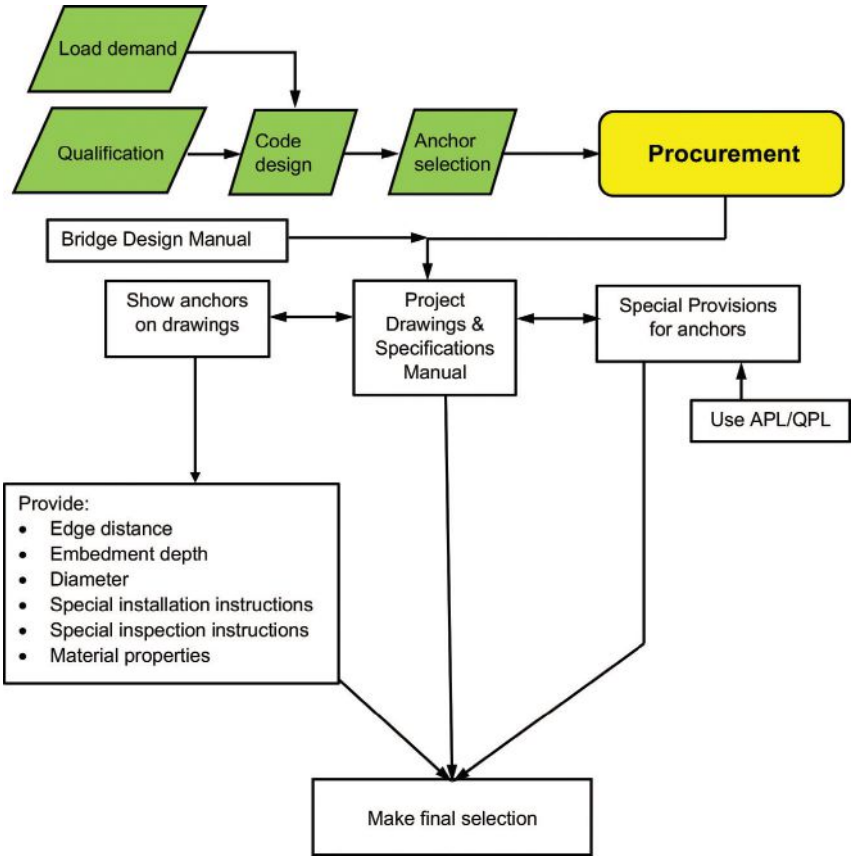

Figure 1. Flowchart depicting the procurement process for anchors.
Note: APL = approved product list; QPL = qualified product list. Figure: PCI.

Background

State highway authorities (SHAs) generally have standard procedures for selecting products that are used in the construction of bridges. The typical procedure for selecting an anchor is for an SHA to check its approved product list (APL) or its qualified product list (QPL) to find a product that the SHA is comfortable using in construction. Part 2 of this series of articles discussed how anchors are qualified by standards, in particular two ACI standards, ACI 355.2, *Qualification of Post-Installed Mechanical Anchors in Concrete and Commentary*,[3] and ACI 355.4, *Qualification of Post-Installed Adhesive Anchors in Concrete and Commentary*.[4] It seems appropriate that any anchor or anchor system that satisfies either of these two qualification standards should be considered acceptable for an SHA's APL or QPL.

How to Procure Anchors

The flowchart in **Fig. 1** provides a convenient overview of issues that must be considered in procuring the desired anchors for the designated project. The procurement process begins with the yellow block in the upper right-hand corner of the flowchart. Assuming the loads have been determined and the qualified anchor selected satisfies the design code, the question becomes, "How is that anchor procured for the project?" There are several separate steps to take, and all of these steps are necessary.

- Show the anchors on the drawings and provide the minimum requirements for edge distance, embedment depth, anchor diameter, material properties,

and any special installation or inspection instructions such as training or certification.

- Specify the anchors in general notes on the drawings, or include this information in either the project specifications or the state construction specifications.
- Because concrete anchorage is new to the AASHTO LRFD specifications, it may take some time for state construction specifications to be updated and incorporate anchors into their standard specifications. So, in the interim, project-specific special provisions may need to be written.

Even after standard specifications for a state are revised to include anchors, project-specific special provisions may be needed if unique characteristics of the project require more detailed instructions or special requirements.

Additional Information to Show on Drawings

Drawings are often the only legacy information that remains long after the project is completed. A prudent approach is to err on the side of putting too much information on the drawings. General notes are usually the best place on drawings to address cast-in-place, mechanical, and adhesive anchors.

The Concrete Reinforcing Steel Institute (CRSI) has published a technical note, CTN M-3-20,[5] which is available as a free download and provides excellent suggestions on what information to include on drawings. The suggested drawing notes are listed under four headings: materials, general installation guidelines, installation, and field quality control. The following sections briefly discuss the notes under each of these headings.

Materials

Notes 1 to 7 in CTN M-3-20 list the specified mechanical anchor products or adhesive anchor products. It is important to include the following items and to tie anchor properties back to the qualification standard: ACI 355.2[3] for mechanical anchors or ACI 355.4[4] for adhesive anchors.

- **List the assumptions.** For adhesive anchors, list the bond shear stress "tau" values for cracked (τ_{cr}) and uncracked (τ_{uncr}) concrete use.
- **List anchor product types.** For an adhesive anchor, all parts of the system should be listed. Have you included the all-thread rod? Is the anchor a manufactured product? Is it stainless steel, and if so, what type of stainless steel? What other hardware is part of the system?
- **Reinforcing bars.** In many cases, adhesively bonded reinforcing bars are used. Specify the grade of the reinforcing bar and the level of cleanliness for the bar surface, such as wire brushing or sandblasting to remove mill scale.
- **Payment.** How will the installed anchor be paid for?

General Installation Guidelines

Notes 8 to 12 describe conditions of the concrete at the time of installation.

- **Concrete strength.** Concrete strength greater than 2500 psi at installation should be required.
- **Concrete age.** The concrete's age at installation for adhesive anchors should be at least 21 days. This is not necessarily a strength requirement, but a moisture content issue. Most, but not all, of the available adhesives are moisture insensitive. The adhesive cartridge and manufacturer's printed installation instructions must be checked.
- **Concrete temperature.** A note should indicate the minimum temperatures of the substrate and the adhesive cartridge material.
- **Thread projection.** What is the minimum projection distance of the anchor?
- **Adhesive cartridge storage.** There are manufacturer's recommendations on the package or in the installation instructions for the minimum and maximum storage temperatures. In warm weather, the adhesive might be runny when ejected from the cartridge and may flash set. In cold weather, the adhesive will take a long time to gel and may freeze before the polymerization process is complete.

Installation

Notes 13 to 25 provide guidelines on key information required of the installer.

- **Certification.** What is the minimum level of training necessary, and do the installers need to be certified? ACI 318-14 currently requires certification of installers when an anchor is installed in a horizontal to upwardly inclined (overhead) orientation. ACI offers an installer certification program that has a certification life of five years.
- **Inspection.** Not all anchors on a job require certified installers. If there is a combination of mechanical and adhesive anchors, the specifications might indicate either a project-specific or state policy on how to identify anchors that require a certified installer. For example, similar to the way epoxy-coated reinforcing bars are often indicated with an "(E)," the designation "(CERT)" could be put on the drawings to indicate anchors requiring installation by a certified installer.

  What about the inspector? The 2019 edition of ACI 318[6] requires inspector certification, and ACI has that certification program available.
- **Installation technique.** The mixing nozzle that comes with the product must be used as-is, without modification. The anchor itself should be clean and free of mud or other contaminants. Without fail, the installation must follow the manufacturer's printed installation instructions. This is a golden rule! Once you put the anchor in the hole, do not disturb the installation.

  Typically, hole drilling is done with a rotary-impact hammer drill. Core holes are typically too smooth. The manufacturer's printed installation instructions are prepared by the anchor manufacturer and list the anchor manufacturer's recommendations regarding whether a cored hole can be used when installing the anchor.
- **Hole cleaning.** It cannot be emphasized enough that this is a very important step in producing a quality installation. Protect recently drilled and cleaned holes if the anchor is not installed immediately. There is no specific measure of the amount of delay permitted because the permissible delay depends on

the environmental conditions. The strong recommendation is that the anchor be installed immediately after the hole is drilled.

Field Quality Control

Notes 26 to 30 offer suggestions on inspection and compliance testing in the field.

- **Inspection.** This was discussed in the previous section but is repeated here because it is part of quality assurance.
- **Proof testing.** The specifications should include provisions for the number of anchors to be tested, the frequency of testing, the pass/fail criteria, and the next level of testing frequency in case of failure. More on this topic will be covered in the forthcoming part 4 article. Additionally, as noted below, PCI has written customizable special provisions for post-installed concrete anchors, and these also discuss field testing.

Special Provisions

The majority of SHAs have a state standard specification manual or a set of bridge special provisions. As part of National Cooperative Highway Research Program (NCHRP) Project 20-44(04), PCI wrote a customizable special provisions section, which is available as a free download as part of the five-part webinar series developed by PCI (see the Course Resource subfolder for webinar no. 4 at https://www.pci.org/AnchoringToConcreteImp). A Microsoft Word version is available so that agencies can tailor the text to a specific project. PCI intentionally tried to make the special provisions text comprehensive. Some information in the document might appear repetitive or may not apply to the project; simply delete what is not needed. The special provisions section originates from an appendix in NCHRP Report 757.[7]

Space does not permit a detailed discussion of the special provisions here, but the outline in the Topics sidebar gives an indication of what is covered.

Summary

This article discusses items that should be included on contract drawings and information that should be included in either the project specifications or in the state construction specifications for concrete anchors. Free downloads are available for resources such as the CRSI technical note.[5]

Under the sponsorship of the NCHRP, PCI developed a five-part webinar series for bridge engineers on the requirements for designing, detailing, and installing concrete anchors. Information discussed in this *ASPIRE* article was covered in webinar no. 4 presented on July 16, 2020. PCI has made available a Dropbox folder (https://www.pci.org/AnchoringToConcreteImp) where documentation from the five-webinar series, including course handouts and resources, can be found. A customizable draft of special provisions for procuring concrete anchors for a project is available for free download from the Course Resource subfolder for webinar no. 4.

Part 4 of this four-part series on concrete anchors will focus on issues related to inspection and compliance testing of concrete anchors in the field.

References

1. American Association of State Highway and Transportation Officials (AASHTO). 2017. *AASHTO LRFD Bridge Design Specifications*, 8th ed. Washington, DC: AASHTO.
2. American Concrete Institute (ACI) Committee 318. 2014. *Building Code Requirements for Structural Concrete* (ACI 318-14) *and Commentary* (ACI 318R-14). Farmington Hills, MI: ACI.
3. ACI Committee 355. 2019. *Qualification of Post-Installed Mechanical Anchors in Concrete* (ACI 355.2-19) *and Commentary*. Farmington Hills, MI: ACI.
4. ACI Committee 355. 2019. *Qualification of Post-Installed Adhesive Anchors in Concrete* (ACI 355.4-19) *and Commentary.* Farmington Hills, MI: ACI.
5. Concrete Reinforcing Steel Institute (CRSI). 2020. *Suggested General Drawing Notes for Post-installed Mechanical and Adhesive Anchors.* CTN-M-3-20. Schaumburg, IL: CRSI. http://resources.crsi.org/resources/notes-for-mechanical-and-adhesive-anchors.
6. ACI Committee 318. 2019. *Building Code Requirements for Structural Concrete* (ACI 318-19) *and Commentary* (ACI 318R-19). Farmington Hills, MI: ACI.
7. Cook, R. A., E. P. Douglas, T. M. Davis, and C. Liu. 2013. *Long-Term Performance of Epoxy Adhesive Anchor Systems*. National Cooperative Highway Research Program (NCHRP) Report 757. Washington, DC: Transportation Research Board.

Topics in the Customizable Special Provisions Developed by PCI

575.01 DESCRIPTION
- Ordering Information

575.02 DEFINITIONS

575.03 MATERIALS
- Reference Documents
- Quality Assurance
  - Installer Qualifications
  - Product Evaluation Reports and Listings
- Mechanical Anchors
- Adhesive Anchors
- Fasteners and Anchors
- Reinforcing Bar
- Equipment
  - Hammer Drill
  - Cleaning Equipment
  - Compressed Air
  - Vacuum

575.04 SUBMITTALS
- Adhesive Anchor Installer Certification

575.05 CONSTRUCTION
- Meeting
- Project Conditions
- Substrate Conditions
- Installation
- Inspection and Maintenance

575.06 FIELD QUALITY CONTROL TESTING
- Demonstration Tests
- Production Proof Tests

575.07 METHOD OF MEASUREMENT

575.08 BASIS OF PAYMENT

Structural Engineering Is More than Just Meeting the Building Code

by Dr. William D. Lawson, Murdough Center for Engineering Professionalism, Texas Tech University

EDITOR'S NOTE

This is the latest in an ongoing discussion in ASPIRE® *about what we can learn regarding professionalism and ethics from the collapse of the Florida International University (FIU) pedestrian bridge and the subsequent National Transportation Safety Board (NTSB) investigation. In the Summer 2020 issue of* ASPIRE*, we invited Dr. William D. Lawson to write an article on engineering responsibility, and in that article he included a lengthy quote from Dr. J. Greg Soules on structural engineering judgment. We were very impressed by Dr. Soules's comments and found that every sentence was significant and worthy of further discussion. So, as part of the continuing discussion in* ASPIRE *of the FIU tragedy and what we should learn from it, we invited Dr. Lawson to interview Dr. Soules to expand on the thoughts Dr. Soules had shared. Their conversation is presented here and reflects Dr. Soules's experience, which has been with buildings rather than bridges.*

About Dr. Soules

*J. G. (Greg) Soules, PhD, PE, SE, PEng, is a senior principal structural engineer and the technical authority for seismic and wind engineering for CB&I Storage Solutions, a division of McDermott International, in Houston, Tex. His responsibilities include the supervision of design engineers, management of engineering for large projects worldwide, and development of CB&I Storage Solutions engineering standards on aboveground storage tanks and on the application of wind loads, seismic loads, and building codes to tank and vessel designs. He is the past chair of the Energy Division of the American Society of Civil Engineers (ASCE), current vice chair of the ASCE/SEI 7 Main Committee (*Minimum Design Loads and Associated Criteria for Buildings and Other Structures*), and chair of the ASCE/ Structural Engineering Institute (SEI) National Technical Program Committee for the ASCE/SEI Structures Congress.*

J. G. (Greg) Soules, PhD, PE, SE, PEng

Dr. Soules earned BS, MS, and PhD degrees in civil engineering from Texas Tech University and an MBA from the University of Houston. He is a fellow of the American Society of Civil Engineers, a fellow of the Structural Engineering Institute, a licensed professional engineer in 23 jurisdictions, and a licensed structural engineer in eight jurisdictions. Dr. Soules was presented the Stephen D. Bechtel, Jr. Energy Award by ASCE in 2010. He joined CB&I Storage Tank Solutions in 1980.

Q This interview came about, in part, as a response to your comments quoted in my Summer 2020 Perspective article on the role of engineering judgment relative to the FIU pedestrian bridge collapse. I would like to further explore those themes with you. As we begin, please share some about your technical experience portfolio, types of projects, geographic scope, etc.

A I have been a structural engineer with CB&I Storage Solutions (originally Chicago Bridge and Iron Company) for more than 40 years. In that time, I have designed structures and foundations for the oil and gas, bulk material, water and wastewater, and aerospace industries worldwide. My technical expertise is primarily in the areas of plate and shell structure design and behavior, wind design, and seismic design.

In his "Board Member Statement" on the FIU pedestrian bridge collapse (the statement was reproduced following the Perspective article by Freeby and Nickas in the Spring 2020 issue of *ASPIRE*), NTSB vice chairman Bruce Landsberg states, "A bridge-building disaster should be incomprehensible in today's technical world" and "the science should be well sorted out by now." Does Mr. Landsberg's statement align with your experience? How so?

Unfortunately, Mr. Landsberg's statement does not align with my experience. Most structural failures occur during construction. The commentary (Section C1.3.1) to ASCE/SEI 37-14, *Design Loads on Structures during Construction*, states, "During erection of a structure, the permanent lateral load-resisting system is generally not complete. Also, other elements of the structural system that are essential to the overall performance of the struc-

ture may not be in place or may only be partially secured. As such, the structure may be vulnerable to severe and widespread damage should a single, local failure or mishap occur." The determination of construction loads is not always straightforward. Both the structural engineering profession and the construction industry promote innovation, so it is not uncommon for a new structural system or a new construction method to be employed. Because of innovation, the science may not always be well sorted out. In many of these cases, engineering and construction professionals rely on their judgment, which in turn is based on their professional experience. Professional judgment, while a key tool used by engineers and constructors, is not perfect. There is also the human factor to deal with. Humans make mistakes, whether in a design office or in the field. Human error cannot be completely eliminated. Please do not take my comments as excusing incompetence or gross negligence. I am simply pointing out that the individuals who engineer and build structures are not perfect.

Please tell *ASPIRE* readers about ASCE 7, and your role there. In what ways does the ASCE 7 Committee inform and improve structural engineering? To what extent has the FIU pedestrian bridge collapse influenced or otherwise affected the work of the committee?

ASCE/SEI 7, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, is the primary document used in the United States (and many international locations) to determine loads used in the design of new structures—live and dead loads, wind loads, seismic loads, etc. I currently serve as the vice chair of the ASCE 7 Main Committee for the 2022 cycle. I will serve as chair of ASCE 7 for the 2028 cycle. ASCE 7 is specified for use in the *International Building Code*, which is the basis for virtually every state and local building code in the United States. A few structures, such as bridges, are outside the scope of ASCE 7 because they are covered by other nationally recognized standards (AASHTO for bridges [the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*]).

[Many] structural engineers use ASCE 7 every day. It provides minimum prescriptive loads and requirements for a wide range of structures, but not all structures. Please note that I stated *minimum* loads and requirements. No building code can cover every possibility for every type of structure. We rely on the skill and judgment of professional engineers to supplement the loads and requirements given in ASCE 7 [and other design specifications]. It should also be noted that the structural engineering profession is moving in the direction of using performance-based design. Basically, in performance-based design, the building code will specify the level of performance required and the structural engineer will use his or her knowledge, judgment, and creativity to meet the specified performance goals. This is in stark contrast to our current prescriptive codes that spell out minimum requirements to meet these performance goals, which often stifle innovation and creativity. [See the Professor's Perspective in this issue of *ASPIRE* for additional insight regarding prescriptive and performance-based design.]

The FIU pedestrian bridge collapse, as with any failure, impacts the structural engineers who serve on the ASCE 7 committee. The collapse, however, did not affect the work of the committee. The collapse occurred during construction of a bridge. The scope of ASCE 7 does not include construction loading, nor does it include bridges. As mentioned in a previous response, ASCE 37, *Design Loads on Structures during Construction*, is the standard that addresses loads during construction. This document recognizes that "many elements of the completed structure that are relied upon implicitly to provide strength, stiffness, stability, or continuity are sometimes not present during construction." I chaired the environmental loads chapter (Chapter 6) of this document during the 2014 cycle. The members of this committee take any construction failure to heart and such events do impact the committee's deliberations.

Clearly you possess deep and broad expertise in the structural engineering discipline. From that perspective, please comment on the observation, "When a [structural] engineer 'follows the code,' engineering judgment is already handled and thus does not come conspicuously into play (very much)."

The "code" represents a set of minimum requirements. The code by itself is not sufficient to ensure a safe structure. It takes the skill and judgment of highly trained engineering and construction professionals to properly implement and supplement these minimum requirements. The code does often address areas where its committee members see deficiencies in professional practice. Sometimes, we end up stifling innovation and creativity to protect the public from a few bad actors. In any case, a "code-compliant" structure is not always suitable for its intended use, nor is it necessarily a guarantee against failure.

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You have asserted that "the codes are minimum requirements and that engineering judgment is very necessary in the design of safe structures." But is this an accurate way to characterize day-to-day structural engineering practice? Do not many, perhaps most, structural designs for buildings and other structures specifically (and only) follow the code? Is this not why we have design codes in the first place?

Day-to-day, structural engineering involves a lot more than simply complying with the requirements in the 402 pages of ASCE 7-16 and other associated documents of technical authority. Determining the best structural system that meets a client's needs, that is constructable, that is economical, and that is safe requires extensive use of the engineering skills learned in school and in practice and requires extensive use of

professional judgment. I personally would not want to work in a building that simply met only the requirements in the code.

As a professional business, structural engineering is not immune from market competition. Owners, for example, might seek an engineered design that is both technically sound and cost effective. In such a case, is it a stretch to believe owners might view the definition of "technically sound" as equivalent to "meeting code," even to the extent that if the engineer were to design beyond code, this would be seen as expensive overdesign?

This question is more about the professional relationship between the owner and his or her structural engineer than it is about "meeting code." In the best relationships, the structural engineer provides a structure that meets the owner's needs. Such a structure is always beyond code in some respects. There are, of course, owners and developers who want the least-expensive structure possible. Some engineers will provide a purely "code-compliant" structure. In many of these cases, the engineer may find himself or herself in court in a few years defending a "code-compliant" structure. Minimum code requirements are insufficient by themselves.

Under what circumstances is the structural engineer justified in going beyond code-mandated minimums? More specifically, what are some of the primary factors you recognize as indicating a project or application may require more than "strictly following the code"?

The structural engineer must understand the needs of his or her client. Serviceability of the structure over its life usually requires more than just meeting minimum code requirements. While the codes require that serviceability be considered, specific requirements are usually few and far between in the code. Serviceability is often specific to the use of the structure, its occupants, and its owner. An owner may decide that a structure is so important to his or her business and livelihood that the structure must be designed for higher environmental loads or more-restrictive drift limits or other requirements that minimize damage and downtime. The codes have life safety as their goal, not economic or functional recovery.

Knowing you face a unique design situation is one thing. Getting the owner or decision maker on board with paying the premium is something else. What experiences in your education and your career prepared you not only to recognize unique or special factors, but also to be able to persuasively (and successfully) advocate for autonomy in your design practice to address these factors—this in the face of ever-more competitive business and/or public opinion pressures?

It takes time to develop a relationship with a client. You have to learn the client's needs and be able to explain how the "added" cost of some feature of the structure meets those needs. An engineer will not come out of school with the skills or experience to persuade a client to do the "right thing." A young engineer will need to develop these skills working with more experienced engineers. For example, I have clients who have extensive internal standards to provide them with structures that meet their needs. My own company has extensive standards to help fill in the gap when a client thinks he or she only wants the "minimum." In these cases, the engineer must help educate the client in what the client's needs really are. These skills take time to develop, so the best outcomes occur with long-term, repeat clients.

You mention that you know a "growing number of younger [structural] engineers who believe they can analyze any problem (correctly) with today's software." Further, these same engineers "accept the software defaults for modeling as gospel—they basically substitute a programmer's judgment for their own when they do this." What is it about using software that prompted you to single out this aspect of the challenge of cultivating sound engineering judgment in structural engineering? Please elaborate.

I have seen a lot of young engineers misuse software that they did not fully understand. They wasted time analyzing and designing a member that they could have done using half of a page of calculation paper instead of the 100-page output from the analysis program. As a profession, structural engineers rely on very complex software to design a wide range of structures, from the average "box" structure to very unique structures such as the Burj Khalifa. The more complex and powerful a software package is, the easier it is to make mistakes. It is interesting to note that a favorite problem on the 16-hour structural engineering license exam is to give the candidate the output from a commercial structural analysis program and ask the candidate to find 10 things wrong with the model. I firmly believe that the initial design of a structure should be done with pencil and paper and then verified using a computer model. Many experienced structural engineers would say you should be able to rough out the initial design on the back of an envelope. To be able to rough out a design takes both engineering judgment and experience.

I firmly believe that the initial design of a structure should be done with pencil and paper and then verified using a computer model.

Turning our attention to the FIU bridge collapse, the NTSB report indicates all parties—the design-builder, the designer, the construction project administrator/inspector, the owner/construction manager, and the state transportation agency—showed poor engineering judgment and response to precollapse cracking. But unforeseen structural response is not unheard of in structural engineering practice. From your experience, how does an engineer handle complex situations where you see movement, cracking, deflection, etc., that is unexpected?

Unexpected behavior of a structure is always a bad sign and should be dealt with immediately. The engineer must determine the cause or, at the very least, treat the symptoms of the problem. There is never enough time to carry out a full-blown "research project" to determine every aspect of the unexpected behavior. Safety of the construction personnel always dictates a quick response. But there are also cost and schedule pressures that demand a quick response to the unexpected behavior. When unexpected behavior occurs, the structural engineer must step up and say "stop." This is never easy in the schedule-driven projects we all work on. It is even harder for a young engineer to do. The engineer of record (EOR) has a responsibility to intervene when such a problem comes up. The pressures can be significant on the EOR. The EOR probably works for a large company, so he or she must report to a boss. The client has a need to use the structure to make money, and the construction company may be under liquidated damages. Given these kinds of pressures, delays are rarely tolerated. The EOR, as I said, must step up and say "stop." He or she (in reality, a team) must rapidly investigate the unexpected behavior and determine a path forward (a plan to correct the unexpected behavior). If the EOR is young and relatively inexperienced, he or she should [seek to] bring in the engineers with gray hair. There is no shame in doing so. This may require delaying the project, reinforcing a deficient portion of the structure, correcting quality issues on the site, etc., etc., etc.

In closing, what lessons learned have you taken from the FIU pedestrian bridge tragedy? Is there anything you say, or do, differently now?

Inasmuch as I am not directly involved with the project or failure investigation, I cannot address specifics of the FIU pedestrian bridge collapse. More broadly, and as I've noted, most structural failures occur during construction. Unexpected behavior must be identified and reported immediately, and action must be taken immediately, even when the action is unpopular with the participants and adversely impacts the project. In my career, I have investigated failures, some with loss of life. Some of these failures were caused by engineering errors, fabrication errors, or construction errors. In all cases, solutions must be found quickly and a plan developed to implement the solutions. I cannot say this is "new knowledge," but the FIU pedestrian bridge collapse does serve as a strong reminder to the structural engineering profession about the importance of this type of response.

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