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THE CONCRETE BRIDGE MAGAZINE

SUMMER 2024

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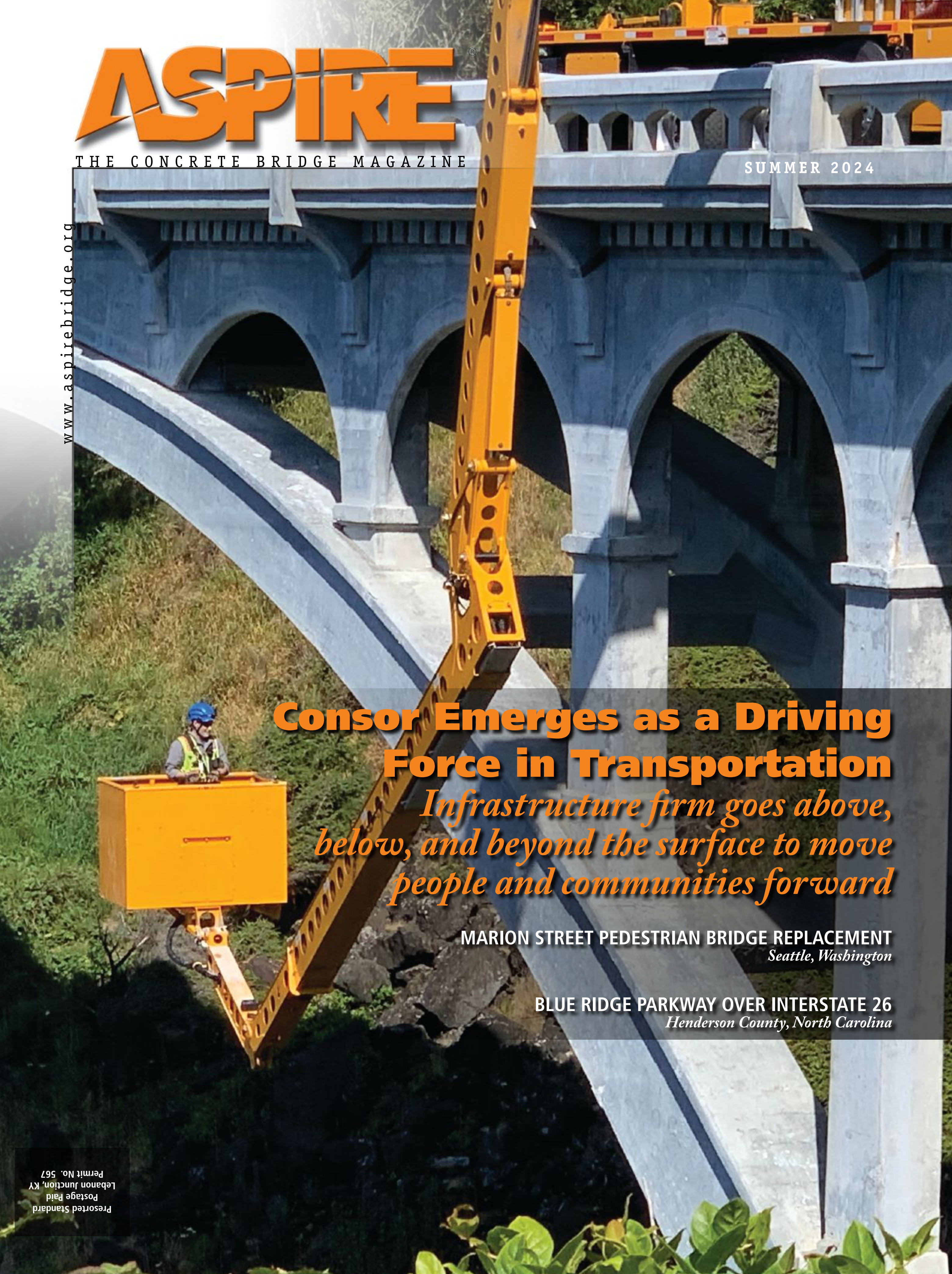

Consor Emerges as a Driving Force in Transportation

Infrastructure firm goes above, below, and beyond the surface to move people and communities forward

MARION STREET PEDESTRIAN BRIDGE REPLACEMENT
Seattle, Washington

BLUE RIDGE PARKWAY OVER INTERSTATE 26
Henderson County, North Carolina

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Departments

Photo: Alamo NEX Construction.

Photo: HDR Inc.

Photo: Federal Highway Administration.

Photo: Consor.

Advertisers' Index

EDITORIAL

Photo: PCI.

Embracing a Culture of Safety and Quality

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

Recently, I was part of a group that toured several precast concrete plants with the goal of sharing technology, efficiencies, and best practices. I paid careful attention to the safety practices, and I was encouraged by what I saw. Every facility followed the standard safety practices— inspirational posters located at each water cooler, yellow lines on the floor delineating walkways, and chains/rails with bollards indicating hazards. I was glad to observe that employees at all plants regularly take part in safety talks.

But one plant stood out from the rest. At that plant, every employee addressed and described each unit's function and processes in terms of what "*my* colleagues do" and what "*our* company" accomplishes. For example, one employee told us, "My colleagues in the department retrained after we got this new CNC [computer numerical control] machine, and now our company is saving materials, avoiding repairs/misfits [misalignments], and producing more with fewer people." I could tell from the unselfconscious tone in which this message was delivered that these employees see themselves as part of a team committed to common goals. This shared vision is critical to achieving a CULTURE of SAFETY and QUALITY.

The culture of safety and quality is not a new topic for the concrete bridge community. For example, we discussed it a few years ago in an editorial titled "Responsibility, Authority, and Accountability" (see the Winter 2017 issue of *ASPIRE*®). As we noted then, responsibilities, authority, and accountability (RAA) are clearly delineated and fully executed in a culture of safety and quality.

But how do we shift people's mindsets to RAA in the first place? Motivational speaker and safety culture expert Garrison Wynn, Certified Safety Professional, offers some good ideas. In a clip from one of his motivational presentations posted on YouTube,[1] he shares, "The reason people don't want to change is mostly because nobody wants to be a senior beginner." He elaborates, "They don't want to wake up and realize that what they know is no longer valid, and so they cling to their old ways."

In these situations, Wynn urges us to show individuals how their knowledge and experience are going to help carry out the new way. "Once they see their experience is still going to be valuable, then you can show them how different might be better."

Wynn also explains that younger people want to know why there are rules and expectations. "When younger people don't know why," he explains, "They feel less safe."

Of course, when we feel pressured for time, it may seem counterproductive to pause and delve into the "why." But this pause is extremely important. If you are in a leadership or management position, resist the impulse to just cite company policy. Instead, take the time to look into the topic and facilitate a real discussion in which the team fully explores how a rule or best practice enhances safety and quality.

For example, if you are visiting a plant, consider engaging employees in a review of the use of dunnage and related storage practices. Remember to explain why this topic matters:

- Proper dunnage and yard storage are needed to ensure the quality of precast concrete components.
- Failure to store products properly can lead to damage, which in turn can lead to waste or the need for field repairs that delay erection.
- Proper storage plans and dunnage methods are necessary to help maintain component quality and keep personnel safe.

Also, share the most common issues that crews might encounter, including improperly placed dunnage that could result in component deformity; unlevel or soft ground for storage; uneven or misaligned stacking that could lead to a tilted stack that damages components or is unstable and dangerous; and so on.

Finally, point out a few key best practices: level bearings on flat ground; the use of two dunnage points, unless more are required by design; timely checks for compliance by a quality-systems team.

As a second example, when at a jobsite, help your team comply with Occupational Safety and Health Administration rules and general crane safety. Start by reviewing the four key exclusion zone areas:

- The load crush zone (the area between a

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A worker from Consor performs inspection work for the Oregon Department of Transportation from a bucket on a snooper truck underneath the Ben Jones Bridge over Rocky Creek. Photo: Consor.

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suspended load and a fixed object)

- The counterweight crush zone (the area between the moving counterweight of the crane and a fixed object)
- The dropped-object zone (the area where individuals are at risk for being struck in the event of a load drop)
- The bystander exclusion zone (the outermost zone, which should be clearly marked and have physical barriers or security checks to prevent unauthorized entry)

Next, look at the site together to identify these zones on this specific project. Reassure yourself and the crew that there are proper deterrents in place to stop unwarranted entry and other dangerous practices—and if there are problems, act promptly to remedy them.

During these teaching moments, be sure to let the employees know that company leadership always wants to know if they ever have safety or quality concerns. Remind them of how to report these kinds of issues. In a culture of safety and quality, every teammate feels empowered to act knowledgeably to keep themselves and others safe.

Let's all do our part to make safety and quality part of our concrete bridge community's culture. We can start by helping everyone overcome resistance to change, taking the time to explain "why," and listening when others express concerns.

Remember, people who feel valued make fewer mistakes, and that positively impacts quality and safety.

Reference

1. Wynn, G. 2018. "Safety Speaker Garrison Wynn: Influence and Engagement." YouTube. https://www.youtube.com/watch?v=nUKkwVzyekM.

EDITOR'S NOTE

The PCI eLearning Center, a tool dedicated specifically to precast concrete and prestressed concrete but available to the general public, has a wide range of educational offerings, including several free, on-demand webinars on the topic of safety and quality culture. To access these webinars, visit https://oasis.pci.org/Public/Catalog/Home.aspx, click "Onboarding" from the Browse menu, and then select the "Mental Health and Safety" or "PCI Production Workshop Series" options.

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CONCRETE CALENDAR FOR SUMMER 2024

The events, dates, and locations listed were accurate at the time of publication. Please check the website of the sponsoring organization.

July 16, 2024
2025 PCI Design Awards
Submission deadline

July 22–25, 2024
Bridge Engineering Institute Conference
Tropicana Las Vegas
Las Vegas, Nev.

August 4–8, 2024
AASHTO Committee on Materials and Pavements
Sheraton Madison
Madison, Wisc.

August 11–14, 2024
4th International Bridge Seismic Workshop
Carleton University
Ottawa, ON, Canada

September 9–13, 2024
PTI Certification Week
Embassy Suites by Hilton Miami Airport
Miami, Fla.

September 15–18, 2024
AREMA Annual Conference and Expo
Kentucky International Convention Center
Louisville, Ky.

September 23–27, 2024
PCI Committee Days Conference
Renaissance Nashville
Nashville, Tenn.

October 1–4, 2024
PTI Committee Days
Kempinski Hotel Cancun
Cancun, Mexico

October 20–23, 2024
ASBI Annual Convention and Committee Meetings
Loews Atlanta Hotel
Atlanta, Ga.

October 21–24, 2024
PTI Certification Week
Doubletree by Hilton Denver
Denver, Colo.

November 3–6, 2024
ACI Concrete Convention
Philadelphia Marriott Downtown
Philadelphia, Pa.

November 10–13, 2024
CRSI Fall Business and Technical Meeting
Drake Hotel
Chicago, Ill.

November 18–24, 2024
PTI Certification Week
Hilton Austin Airport
Austin, Tex.

January 5–9, 2025
Transportation Research Board Annual Meeting
Walter E. Washington Convention Center
Washington, D.C.

February 3–7, 2025
PCI Convention at The Precast Show
Indianapolis Marriott Downtown
Indianapolis, Ind.

Consor Emerges as a Driving Force in the Transportation Industry

This coast-to-coast infrastructure firm focuses on going above, below, and beyond the surface to move people and communities forward.

by Monica Schultes

In 2018, four regional consultancies merged to create Consor, a civil infrastructure consulting firm with sufficient breadth and depth to compete on the national stage, win high-profile projects, and provide excellent service.

With approximately 1700 employees based in 95 offices across North America, Consor focuses on public infrastructure for transportation and water. While the Consor name and brand are technically young, the firm's roots date back to the 1980s. Their services include planning, design, strategic planning and communications, structural assessment, program management, and construction.

Meeting Client Expectations

Transportation and bridge design are at the core of Consor's offerings. The firm's engineers and other professionals are recognized as industry leaders in roadway and bridge engineering, and their decades of experience have included new bridge designs, bridge-widening projects, bridge repairs and rehabilitations, and pedestrian bridges, as well as bridge inspections and load ratings. Consor also provides design services for a wide range of ancillary highway structures, including traffic signal and sign structure supports, noise walls, retaining walls, and culverts.

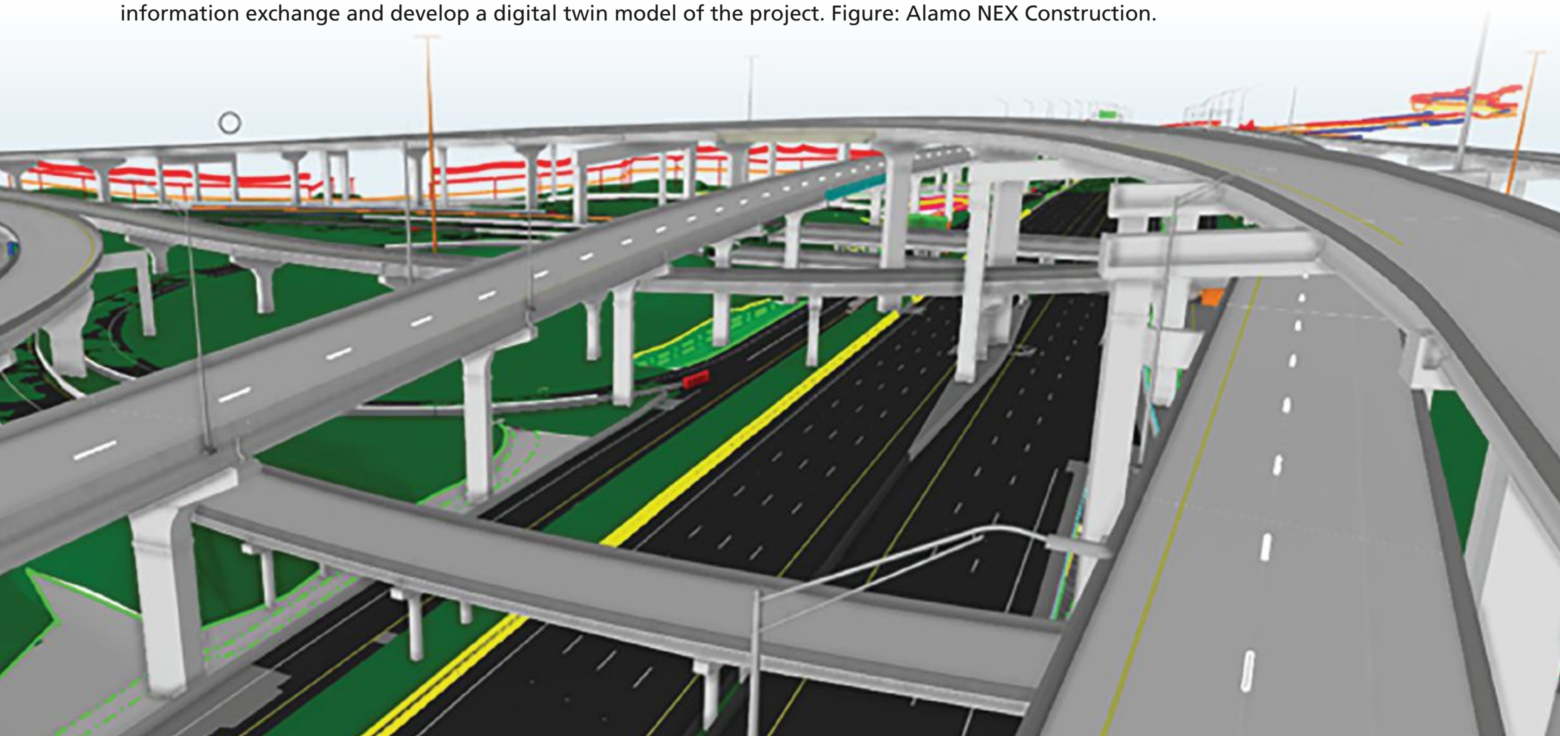
"Consor takes a regional approach to designing bridges across the country to make sure the design considers local construction practices, materials, and techniques to meet client expectations," says Sandeep Patil, chief engineering officer at Consor. "Having local offices in multiple states allows us to share best practices across the country to better serve our client partners. Our nationwide structural-assessment practice provides insight into the types and causes of deterioration in bridges, which we draw upon to design resilient structures."

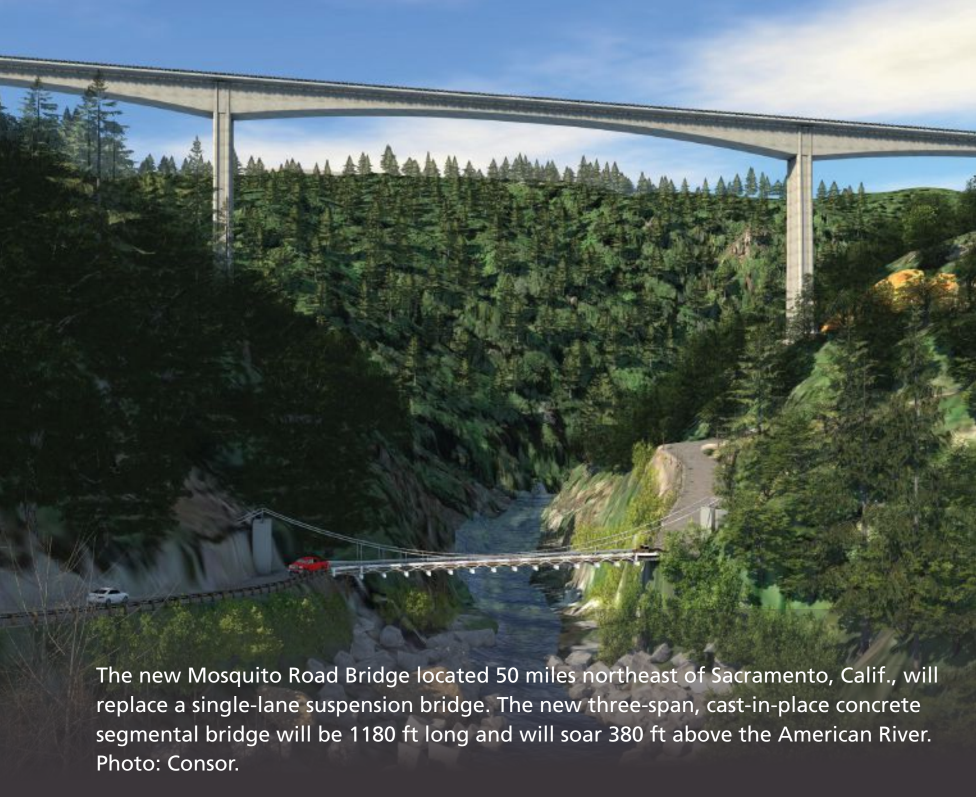
"Our nationwide structural-assessment practice provides insights into the types and causes of deterioration in bridges, which we draw upon to design resilient structures."

Mosquito Road Bridge

Consor is the prime consultant responsible for the design of the Mosquito Road Bridge project, which is currently under construction

A rendering of part of the $1.5 billion Northeast Expansion Central Project in the San Antonio, Tex., area. The project is part of the Texas Clear Lanes Initiative. Consor, in coordination with Alamo NEX Construction, used a cutting-edge software platform to improve information exchange and develop a digital twin model of the project. Figure: Alamo NEX Construction.





The new Mosquito Road Bridge located 50 miles northeast of Sacramento, Calif., will replace a single-lane suspension bridge. The new three-span, cast-in-place concrete segmental bridge will be 1180 ft long and will soar 380 ft above the American River. Photo: Consor.

and scheduled to be completed in 2025. Located 50 miles northeast of Sacramento, Calif., the new structure spanning the American River Canyon is a three-span (322-536-322 ft), cast-in-place concrete segmental bridge. The bridge deck soars 380 ft above the American River on columns that are nearly 200 ft tall. Given the significant structure height, a balanced-cantilever construction method was used to eliminate the need for falsework.

The Mosquito Road Bridge will improve access for El Dorado County's emergency vehicles. The project is replacing a single-lane suspension bridge and eliminating more than 1 mile of roadway switchbacks, which are subject to landslides. Consor is the engineer of record for the bridge foundation design as well as several combination ground-anchor/soil-nail retaining walls, which are approximately 60 ft tall. Consor also performed an independent peer review of the superstructure and column design.

22nd Street Bridge

Consor is serving as the engineer of record for the 22nd Street Bridge project in Tucson, Ariz. The bridge consists of a 1433-ft-long, three-span, cast-in-place

History of Consor

Consor was established in 2018 when four regional consultant leaders—AIA Engineers, Infrastructure Engineers, Target Engineering Group, and Johnson-Adams & Associates—joined forces.

Following the merger, Consor has built a coast-to-coast presence and expanded to add complementary services to their portfolio. This growth has included the addition of significant industry players: Structural Grace, TKW Consulting Engineers, U.S. Underwater, Apex Design, Murraysmith + Quincy Engineering, Civic Engineering & Information Technology, CPM Associates, Project Engineering Consultants, and, most recently, American Consulting Professionals.

Going forward, Consor is focused on continuing to deepen their technical services while cultivating an industry-leading employee experience.

Consor designed the 22nd Street Bridge project in Tucson, Ariz., as a three-span, cast-in-place concrete segmental structure over Union Pacific Railroad tracks and city streets. Construction to build the new bridge, which will replace a functionally obsolete structure, is scheduled to begin in 2025. Rendering: Consor.

On the Interstate 35 Northeast Expansion Central Project in the San Antonio, Tex., area, Consor was responsible for the design of 7.6 miles of elevated structures, which included more than 350 prestressed concrete hammerhead bent caps on top of cast-in-place columns. The left photo shows prestressed concrete girders on prestressed concrete hammerhead bent caps, and the right photo shows the architectural details of the cast-in-place columns and the protruding reinforcement for the connection to the precast concrete bent caps. Photos: Alamo NEX Construction.

concrete segmental structure combined with four AASHTO girder back spans over a series of Union Pacific Railroad tracks and city streets. The bridge will increase lane capacity from two to three, provide pedestrian and bicycle access, and increase horizontal and vertical clearances.

According to Mike Keller, technical practice manager of complex bridges at Consor, the existing structure is "functionally obsolete." Keller adds that the balanced-cantilever construction method will reduce falsework and move construction above and away from the railroad operations. The project is minimizing impacts to traffic and the environment, which in turn benefits the community. With the project being developed under a construction-manager-at-risk contract, Consor has worked with the City of Tucson, Union Pacific Railroad, the contractor, and the local community to meet the needs of all project stakeholders. Construction is scheduled to begin in early 2025.

Wildlife Bridge Crossings

Consor collaborated with the Florida Department of Transportation (FDOT) on unique wildlife crossing structures near Interstate 75 (I-75) in southern Florida, commonly known as "Alligator Alley." What began as a resurfacing project was expanded to include wildlife crossings over canals that run parallel to I-75. The crossings were needed to prevent wildlife-vehicle collisions after fencing was placed along a 9-mile stretch of I-75.

When the structures were added to the project, the team used low-cost materials in unique ways to stay within FDOT's budget. Prestressed concrete piles placed horizontally were used to span distances of up to 75 ft. Concrete end blocks were used to keep concrete beam members from washing downstream when water is anticipated to overtop the bridges during rainy seasons or storm events.

The new structures enable wildlife such as black bears and Florida panthers to access existing culverts to cross under the interstate.

Digital Advancements for the Interstate 35 Northeast Expansion Central Project

The Texas Department of Transportation launched a $1.5 billion project to construct a viaduct that will add one high-occupancy-vehicle lane and two general-purpose lanes in each direction along Interstate 35 (I-35) in northeast San Antonio as well as the neighboring municipalities of Live Oak, Selma, and Schertz. Alamo NEX Construction (ANC), a joint venture between Ferrovial

Bridge Inspection Standards

In 2022, after more than a decade of review, the Federal Highway Administration (FHWA) issued an update to the highway bridge inspection standards. Their recommendations include a risk-based, data-driven approach to inspection intervals and rigorous professional licensing and training requirements for bridge inspectors. Consor played an active role in the development of the new provisions, offering input on the mandatory qualifications for inspection program team leaders as well as the requirements for training bridge inspectors.

Consor has also been involved in the training of inspectors. Since 2007, the FHWA has tasked them with teaching bridge inspection courses across the country. According to Jeff Rowe, executive vice president of structural assessment at Consor, his firm has trained individuals on underwater bridge inspection in partnership with the FHWA's National Highway Institute. Rowe, himself an engineer diver, has performed thousands of inspections and trained hundreds of inspectors. Consor is also helping to update FHWA's *Underwater Bridge Inspection Manual*.

In southern Florida, wildlife crossings allow black bears, cougars, and other animals to traverse the canals running along Alligator Alley (Interstate 75) and safely cross under the interstate at existing culverts. For some of the crossings, Consor used a cost-efficient design with prestressed concrete piles placed horizontally in a simple-span configuration. Photo: Consor.

Construction US Corp. and Webber LLC, was selected for the design, construction, and maintenance of the I-35 Northeast Expansion Central Project and brought Consor onto the team for design services. To overcome the challenges of working in a limited space and integrating the new structure with existing infrastructure, the team turned to a collaborative three-dimensional (3-D) technology solution.

ANC used a cutting-edge software platform to improve information exchange and develop a digital twin model of the project. During the design phase, an automated clash-detection program identified more than 3500 potential conflicts between various components. Early identification helps prevent disruptions and enhances overall efficiency.

Consor was responsible for the design of 7.6 miles of elevated structures, including the corresponding roadway, drainage, illumination, intelligent transportation systems, signing, and pavement marking designs. The bridge submittals were broken down by segment and bridge element, with separate deliverables for bridge superstructures, substructures, and foundations. There were 23 bridge units with a total of 298 spans. These were split into six segments with four bridge submittal packages each. This arrangement allowed crews to begin construction on the highest-priority locations while the design elsewhere was being finalized. All of the Consor bridge elements were modeled in 3-D as part of the design, which included over 2.2 million ft^2 of deck, nearly 1700 girders (156 Tx82, 1279 Tx70, and 251 Tx60) totaling over 230,000 ft in length, and 21,000 ft of concrete drilled shafts. In addition, Consor designed more than 350 prestressed concrete hammerhead bent caps to optimize the design for fabrication and installation. The prestressed concrete bent caps were typically 40 or 52 ft long, with depths ranging from 7 to 9.5 ft, and were supported on cast-in-place concrete columns.

Structural Assessment

Consor is a national leader in full-service structural inspections, and its team of bridge inspectors has delivered more than 60,000 bridge inspections in 49 states. Every Consor inspector is certified in specific areas of expertise, from climbing inspection to underwater inspection. Visual inspection of bridges is the first step of federally mandated bridge inspection. The firm's engineer divers and technicians perform commercial diving and underwater structural assessments—specialty areas that are vitally important but often overlooked.

According to Jeff Rowe, executive vice president of structural assessment at Consor, the firm's inspection services

A drone captures an aerial view of the new viaduct under construction northeast of San Antonio, Tex. Consor optimized the design with the use of precast concrete Tx82 girders on all but two spans. Photo: Alamo NEX Construction.

often lead to additional work, such as rehabilitation projects. Rowe believes that the combination of bridge inspection and complementary construction inspection and design services gives Consor a unique perspective on each project. "We don't work in a vacuum. Any deterioration we witness firsthand informs our design," he says.

"We don't work in a vacuum. Any deterioration we witness firsthand informs our design."

Keller agrees, explaining that "we get the opportunity to see what works well in design, during construction, and in service. Then we use that knowledge to avoid problematic details or field issues. Not everyone has that trifecta in the consulting world."

Technology

Unmanned aircraft systems (UAS), or drones, have become ubiquitous in bridge inspections. "Drones supplement the visual inspection and are among the many tools available to assess a structure," says Rowe. "We look at a drone as a tool. It is not intended to replace an inspector; however, it can provide information efficiently and gain access more easily. We have nearly 30 UAS for our teams to use," he adds.

Drones can record heat signatures emanating from concrete bridges to locate areas of delamination, and they can be used in underwater systems to measure scour and record images of piers. A promising area of use is to document cracks in concrete structures. Rowe predicts that artificial intelligence (AI) may perform this time-consuming task in the future. The AI software identifies cracks, measures and monitors their sizes, and informs the inspectors of areas to investigate.

Inspired Employees

Consor's complete services, from inspection through design and construction, allow the firm to meet the needs of clients at any point in the life cycle of structures. The diveristy of service offerings also helps Consor recruit and retain qualified employees. Whether designing a complex structure or inspecting a signature bridge, "it goes back to having opportunities for people to pursue their passion," says Rowe. "We try to have opportunities for everyone at Consor to fulfill their passion and develop as professionals and as people." The firm's ideal project allows them to inspire the people involved, which is effective in attracting and keeping their talented workforce.

A Consor bridge engineer diver enters the water to inspect the piers supporting Interstate 526 over the Wando River in Charleston, S.C. Underwater structural assessment is one of the firm's unusual areas of expertise. Photo: Consor.

As part of Consor's statewide inspection work for the South Carolina Department of Transportation, inspectors certified by the Society of Professional Rope Access Technicians climb the cables anchored to the diamond-shaped concrete towers supporting the Arthur Ravenel Jr. Bridge over the Cooper River in Charleston, S.C. Photo: Consor.

PCI's Workforce Development Committee

Prioritizing recruitment, retention, and wellness to address key challenges facing the industry

by Erin Clark, Standard Concrete Products Inc.

While networking at the PCI Student Job Fair during the 2024 PCI Convention, students and professionals build bridges in the precast concrete industry. All Photos: PCI.

Workforce development is an industry-wide issue that we are all facing. One of Matt DeVoss's goals as PCI's chair in 2022 was to "lead PCI in establishing new workforce recruitment and development plans to attract talent to the precast concrete industry at all levels." Over the last year and a half, the Workforce Development (WFD) Committee has been working diligently to help accomplish that goal. The committee began by creating a strategic plan focusing on recruitment and retention. The demographics prioritized for recruitment are women, veterans, second-chance workers (that is, individuals with criminal records), immigrants, and traditional workers. The priorities for retention are onboarding, employee experience, and professional development.

Since our first meeting during PCI Committee Days in September 2022, we have been gathering and promoting resources on the PCI website that align with our strategic plan.[1] We have expanded our efforts by adding the Workplace Wellness Task Group. Its vision is to create a construction industry where mental health is prioritized, stigma is eradicated, and every worker feels supported and valued. The task group's goals are to raise awareness, promote supportive culture, and provide resources, as well as to educate and train. Since the formation of the Workplace Wellness Task Group, we have operated a booth at the PCI Convention to promote our efforts and spread the word.

Additionally, the WFD Committee hosted a webinar by Cal Byer called "Shining Light on Mental Health and Suicide Prevention" and created stickers and posters with a QR code linking directly to our resources page. We are slated to host four webinars this year. Potential webinars include a presentation by a speaker from the Center of Employment Opportunities (a second-chance program), a mentorship program case study, and a presentation on the SkillBridge veterans' program. Furthermore, we are developing a bimonthly email to PCI members to highlight some of the resources that are available on the WFD web page.

The WFD Committee collaborates with the PCI regions in their local efforts to introduce people to the career opportunities in the precast concrete industry. Mike Johnsrud and Margaret Mills at PCI Midwest developed a variety of educational videos on the wide array of careers that are available in our industry. Dan Eckenrode at PCI Gulf South developed a letter (available on our resources page) that he mails out to vocational schools within a 50-mile radius of his producers' facilities to build relationships and educate them about our industry. Lenny Salvo of Coreslab

Scan the QR code to access all PCI workforce development resources in recruitment, retention, and wellness.

Students and other participants convene at the Women in Precast event during the 2024 PCI Convention to engage in the networking and discussions around professional growth within the precast concrete industry.

Structures (Orlando) serves on the the executive committee of the Florida Prestressed Concrete Association (a chapter of PCI) and on a local county school career and technical education advisory council that recently started distributing a guide for eighth graders describing local career-path options, including careers in the precast concrete industry. The local outreach in all of these regions is incredible. Our committee wants to help share success stories and ideas among PCI regions to strengthen the nationwide labor pool.

The Recruitment Task Group is currently developing a three-dimensional animation video for PCI members to show students or present at job fairs. This video will show examples of precast concrete structures and how they are constructed to create some excitement and promote the precast concrete industry. The group is also working on developing relationships with organizations such as the Center for Employment Opportunities, which helps incorporate second-chance workers back into the workforce. Another goal is to continue to promote and support programs such as Leadership PCI, the Productivity Tour, the Architectural Seminar, and other workforce-related efforts from PCI committees. A collection of interviews from women, veterans, second-chance workers, and foreign workers will be part of a video series to promote the industry and gain access to these demographics. The Retention Task Group is working with the Safety Committee to develop new training videos in English and Spanish that will be made available to PCI members.

Employing foreign workers has been a challenge. Bob Risser, PCI president and CEO, has supported our efforts by meeting in Washington, D.C., with politicians who want to help our industry gain access to foreign workers. In a recent meeting with U.S. Rep. Lloyd Smucker (PA-11), Bob learned about H.R. 3734, which proposes to "amend the Immigration and Nationality Act to provide for an H-2C nonimmigrant classification, and for other purposes."[2] This bill would help alleviate the worker shortage in the construction industry by establishing a market-driven visa system to help employers find more laborers. Employers would be required to prove they were unable to find U.S. workers for vacant positions, pay their foreign employees fair wages based on local wage data, and use the E-Verify program to ensure that only individuals with H-2C nonimmigrant status are hired. The sponsors of H.R. 3734 have asked our membership to reach out to their local representatives to support this bill.

Members gather during the 2024 PCI Convention at the PCI Resource Center, which highlights workforce, wellness, and PCI resources.

We are extremely grateful to have active, motivated committee members who are working to make a difference in the precast concrete industry. The success of our efforts relies on the time and dedication given by all of our members.

The WFD Committee has had an outpouring of support and input from PCI members, which we greatly appreciate! Our mission is to host and facilitate national and local resources to engage and develop the precast concrete workforce, and we are only successful if we are providing the necessary resources. Please visit our website, pci.org/workforce, for more information and to see how we can be of assistance.

References

1. Precast/Prestressed Concrete Institute. n.d. "Workforce Development." Accessed April 19, 2024. https://www.pci.org/workforce.
2. Essential Workers for Economic Advancement Act, H.R. 3734, 118th Cong. (2023–2024). https://www.congress.gov/bill/118th-congress/house-bill/3734.

Erin Clark is with Standard Concrete Products Inc. in Columbus, Ga. where her roles include sales and marketing, contracts management, and special projects. She is also chair of the PCI Workforce Development Committee.

PROJECT

Marion Street Pedestrian Bridge Replacement: A New Icon for the Revitalized Seattle Waterfront

by Don Nguyen and Bethy Clark, HDR Inc., and Stephen Wilson, Seattle Department of Transportation

The project to replace one of the busiest pedestrian bridges on the West Coast, the Marion Street Pedestrian Bridge servicing the Colman Dock Ferry Terminal, has been a collaboration between the City of Seattle, Wash., the Washington State Department of Transportation (WSDOT), and bridge design engineers. The new bridge is located in a high-seismic region, in the dense urban area of Seattle's Central Waterfront adjacent to the Elliott Bay seawall. The project, which is part of the Waterfront Seattle Program's overall transformation of Seattle's downtown waterfront, includes a main-span bridge over a major arterial, an approach bridge above a sidewalk, and a transition to an existing pedestrian bridge that remains in place.

Project Background

In 2019, the State Route 99 (Alaskan Way) double-decker viaduct freeway structure, which posed a barrier between Seattle's downtown corridor and the waterfront, was replaced with a tunnel as part of the waterfront improvement efforts. The city is constructing a park promenade along the water and building a new surface street where the viaduct once stood. The Colman Dock Ferry Terminal has been reconstructed, and the adjacent attached building is slated for demolition and replacement.

Along with those changes, the city needed to replace the Marion Street Pedestrian Bridge to provide improved waterfront access. The existing bridge was a combination of bridge types using various materials, with some segments dating back to the 1930s. It included steel through-plate girders as well as an aluminum truss and concrete and

Photo from 2019 showing the previous pedestrian structures, which included a section that traveled beneath the State Route 99 (Alaskan Way) viaduct. The double-deck viaduct has since been demolished and replaced with a tunnel.

profile

MARION STREET PEDESTRIAN BRIDGE / SEATTLE, WASHINGTON

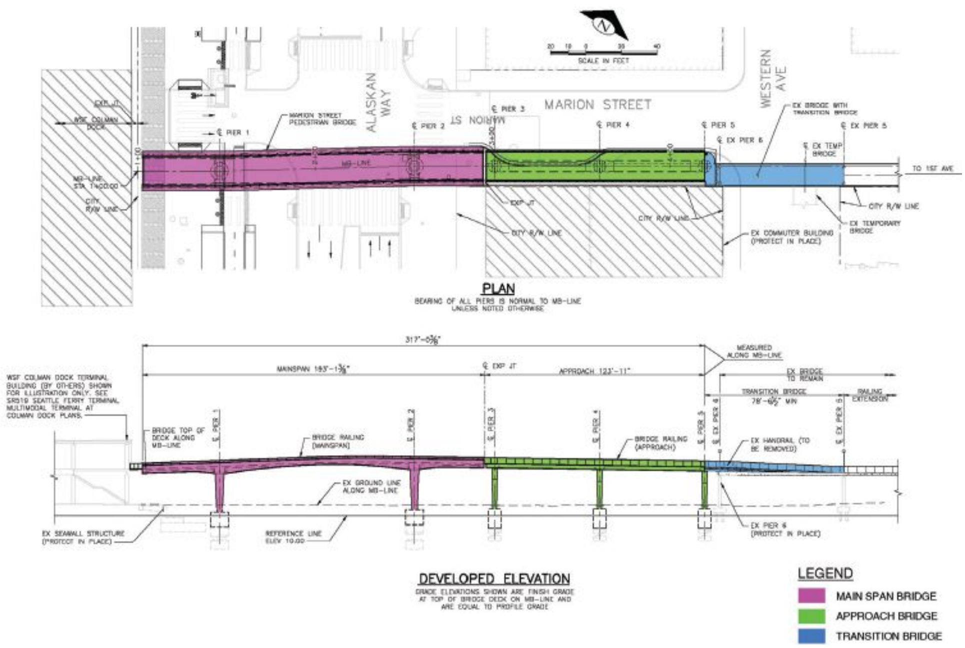
BRIDGE DESIGN ENGINEER: HDR Inc., Bellevue, Wash.

OTHER CONSULTANTS: Architecture: Rosales + Partners, Boston, Mass.; electrical engineering: Reyes Engineering, Portland, Ore.; construction engineering: Ott-Sakai & Associates, Mountlake Terrace, Wash.; geotechnical consultant: Shannon & Wilson, Seattle, Wash.

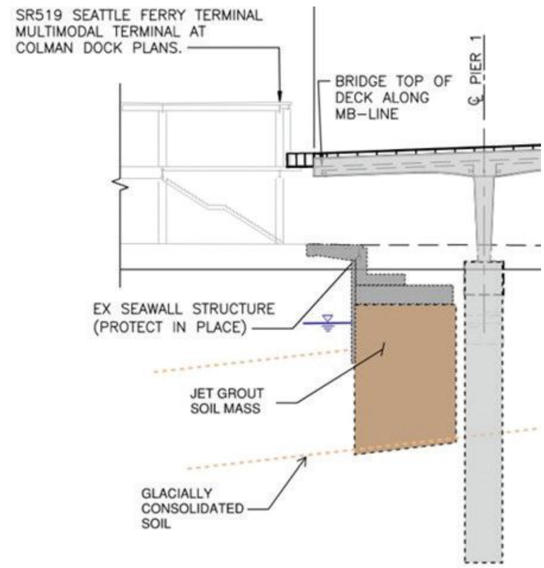
PRIME CONTRACTORS: Superstructure and columns: Flatiron, Renton, Wash.; approach foundations: Walsh, Seattle, Wash.; main-span bridge foundations: Gary Merlino Construction Co., Seattle, Wash.

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

CONCRETE SUPPLIERS: Superstructure and columns: CalPortland, Seattle, Wash.; foundations: Cadman, Seattle, Wash.



The Marion Street Pedestrian Bridge project is part of the Waterfront Seattle Program's overall transformation of Seattle's downtown waterfront. It includes a main-span bridge over a major arterial, an approach bridge above a sidewalk, and a transition to the existing pedestrian bridge. All Photos and Figures: HDR Inc.



An elevation view of the west cantilever of the Marion Street Pedestrian Bridge, its connection to the ferry terminal, and the proximity of the bridge's foundation to the seawall structure and its foundation.

steel superstructure that hung off the sides of neighboring buildings in some places and shared foundations with adjacent structures in others. One particularly uninviting section of the bridge snaked under the viaduct. During peak commuting times, the bridge would become very congested, and its steep grades did not meet the accessibility requirements of the Americans with Disabilities Act (ADA). Goals for the bridge project included ADA compliance, a walkable width to meet projected levels of use, enhanced user experience, and an aesthetic design suitable for the structure's prominence and context within the new Seattle Waterfront.

Early Design Considerations

The first consideration for the new bridge was to determine possible foundation locations that would avoid existing utilities and structures. Notably, the bridge had to cantilever over the Elliott Bay seawall and its improved soil mass to connect to the Colman Dock Ferry Terminal.

Once an optimal span configuration was set, three superstructure alternatives—cast-in-place concrete, extradosed concrete, and steel Fink truss—were analyzed from visual, structural, constructability, cost, and long-term maintenance perspectives. Ultimately, the cast-in-place concrete option was recommended because it best fulfilled the project goals by combinging cost, aesthetic, and constructability objectives into a balanced, cohesive, and clear structural system.

The geotechnical consultant provided recommendations for the design of the drilled-shaft foundations and designed the Elliott Bay seawall's improved soil mass. The placement of the foundations, the foundation type, the construction tolerance and risk, and the unique soil-structure interaction led to the use of drilled shafts and a long span over the arterial with cantilevers on either side. The geotechnical analysis, originally completed for the seawall design, was used to determine lateral spreading and liquefaction design parameters for the drilled-shaft foundations. The models determined that liquefaction occurred after peak ground motions and provided a peak ground acceleration concurrent with liquefaction. That finding reduced the overall demand on the foundations, and a total of 19 loading combinations were analyzed to account for the various strength, service, and seismic cases.

Main-Span Bridge Design

The three-span, cast-in-place concrete superstructure of the main-span bridge includes a gracefully arched center span with cantilevered end spans and angular V-shaped columns. To design this signature feature of the revitalized downtown waterfront, the bridge architect also incorporated suggestions from the Seattle Design Commission and considered the planned aesthetics of the

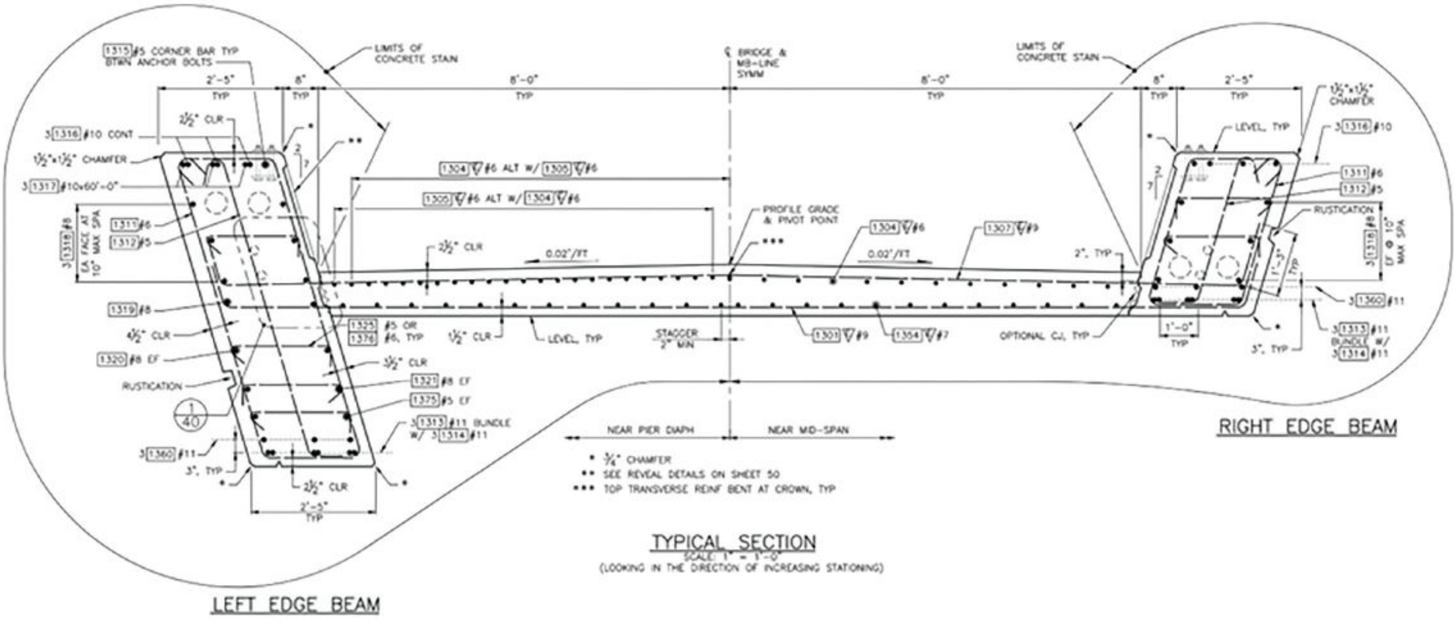
SEATTLE DEPARTMENT OF TRANSPORTATION, OWNER

OTHER MATERIAL SUPPLIERS: Rebar International, Edgewood, Wash.; Rainier Welding, Redmond, Wash.

BRIDGE DESCRIPTION: The main-span bridge is a cast-in-place, post-tensioned concrete structure with two edge beams and a concrete deck slab. It is 192 ft 6 in. long including a 43 ft 7 in. cantilever, a 110 ft center span, and a 38 ft 10 in. cantilever. The approach bridge is a cast-in-place, post-tensioned concrete slab bridge. It is 123 ft 3 in. long with two 59 ft 3 in. spans.

STRUCTURAL COMPONENTS: Concrete edge beams and deck slab for the main-span bridge; concrete slab superstructure for the approach bridge; cast-in-place concrete columns supported on drilled shafts for both bridges. The project used 450,000 lb of structural reinforcement and 800 yd^3 of concrete.

BRIDGE CONSTRUCTION COST: $9.4 million.



Cross section of the Marion Street Bridge showing the variable-depth, post-tensioned edge beams connected by a reinforced concrete deck slab.

new ferry terminal building. The length of the main-span bridge is approximately 192 ft 6 in. from the cantilevered west end at Colman Dock to the cantilevered east end at the approach bridge, not including the expansion joints. The span 1 (west) cantilever is 43 ft 7 in. long. The center span's length between piers 1 and 2 is approximately 110 ft. The span 3 (east) cantilever is 38 ft 10 in. long.

The cast-in-place, post-tensioned concrete structure includes two post-tensioned concrete edge beams connected by a reinforced concrete deck slab. Diaphragms connect the deck, edge beams, and columns at the piers. The edge beams vary in depth, from the maximum depth of 6 ft 3½ in. for the edge beams at pier 1 to the minimum depth of 4 ft 6 in. at the cantilevered ends. There are two tendons in each edge beam, and each tendon has twenty-seven 0.6-in.-diameter strands. The concrete for the edge beams was placed, then the deck was placed, and the edge beams were later post-tensioned. The concrete design strength was 5000 psi. Epoxy-coated reinforcement and increased concrete covers were used to minimize corrosion risk due to the proximity of the bridge to seawater and the use of deicing salts during cold weather. The substructure consists of integral V-shaped piers supported by 9-ft 10-in-diameter drilled shafts. The shapes of the piers as well as the superstructures contribute to the bridge's aesthetics. The piers taper in two directions, which made detailing for the reinforcement as well as construction a challenge because the stirrups also taper in two directions. Given the two arms of the V-shape, the longitudinal reinforcement had to

The cast-in-place concrete edge girders of the main-span bridge were placed first and then used as work platforms during the deck placement. Epoxy-coated reinforcement and large concrete covers were used to minimize corrosion risk.

With 5 million users passing through the Colman Dock Ferry Terminal each year, the new Marion Street Pedestrian Bridge enables pedestrians safe passage over multiple streets to access downtown Seattle.

The two-span approach bridge, a cast-in-place, post-tensioned concrete slab structure, connects to the east cantilever of the main-span bridge.

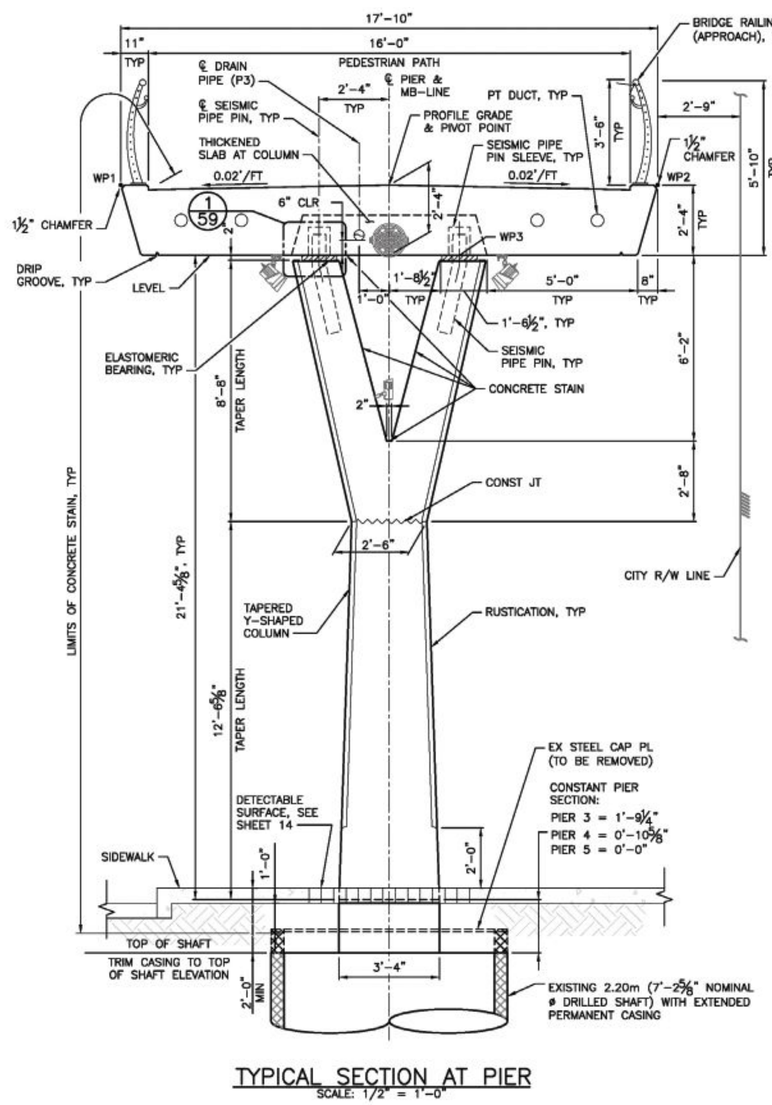
View from the top of the main-span bridge showing the stainless steel railing, pod lights, and a scenic view of the Seattle Waterfront.

weave from one face to the other to provide connectivity.

The site constraints and bridge geometry presented a few design and construction challenges to overcome. With the varying-depth edge beams tying into the deck at different elevations, the reinforcement as well as the post-tensioning duct placement had to be carefully coordinated with the contractor. The bridge was constructed on falsework, the design of which required special consideration to avoid impacts (such as unanticipated loading or settlement) to sensitive utilities and the seawall. In addition, the construction of the main-span bridge required concrete to be placed over a main overheight route through Seattle. Much of the work on the project had to be accomplished within the limited allowable closures for the main arterial and the exit lane for vehicles using the ferry terminal.

Seismic Movement Considerations

The expansion joint at the west end of the main-span bridge connects to the second story of the ferry terminal. The joint must accommodate very large movements while maintaining ADA accessibility. At this interface, there is a 3-ft gap between the end of the superstructure and the start of the building. Under the design earthquake event, the bridge moves only 18 in. longitudinally; however, total longitudinal movement exceeds 6 ft 6 in because the entry building movement includes the two-story



The approach bridge structure type is a cast-in-place, post-tensioned concrete slab. To maintain a relatively thin superstructure-to-column connection at the approach bridge, the design team took a unique approach using a seismic pipe-pin system, which would not transfer moment from the column into the superstructure.

The new Marion Street Pedestrian Bridge offers a width to meet projected levels of use, enhances the user experience, and showcases a bridge design suitable for the structure's prominence and context within the new Seattle Waterfront.

building as well as the movement from the lower trestle. There is also 5 ft of lateral movement and 1 ft 6 in. of vertical movement. Many expansion-joint options available today can either comply with ADA requirements or allow the level of movement needed here, but cannot do both. The solution was a 1-in.-thick stainless steel plate that spans the gap with a series of tensioned springs to clamp down the plate. The plate bears on an ultra-high-molecular-weight polyethylene bearing bar that slides on an embedded steel plate to accommodate the required range of motion. The plate thickness was governed by deflection limits rather than strength. At the ends of the plate, a hinge plate is attached to reduce the height drop to ¼ in. for ADA compliance. The bridge design team worked closely with the ferry terminal designers to design a safe, serviceable transition between the two structures.

Approach Bridge Design

The approach bridge length is approximately 123 ft 3 in., not including the expansion joints. The approach bridge consists of two spans, each measuring 59 ft 3 in. between piers. The structure type is a cast-in-place, post-tensioned concrete slab using 5000-psi concrete and four post-tensioning tendons with 14 strands per tendon. The depth of the concrete slab ranges from 2 ft 4 in., measured at the center of the crown at the pier locations, to 1 ft at the crown away from the piers.

The substructure consists of Y-shaped piers supported by 7-ft 2.5-in.-diameter drilled shafts. These piers are smaller than the V-shaped piers of the main-span bridge but use the same family of shapes to create a harmonious ensemble. The Y-shaped piers had similar detailing and construction challenges as the V-shaped piers. The columns are not integral with the superstructure. The design team took a unique approach using a seismic pipe-pin system to maintain a relatively thin superstructure-to-column connection at the approach bridge. The seismic pipe-pin connections were used for the approach bridge column-to-superstructure connection in lieu of integral connections to limit the moment that could be transferred from the columns into the superstructure during a seismic event. A pipe-pin detail similar to one used by the California Department of Transportation and WSDOT was used on this bridge. This detail allows service-level moments, but it does not transfer moment from the columns into the superstructure in a seismic event. During post-tensioning of the superstructure, setting the gaps for the pipe-pins was important to account for both long- and short-term movements.

Conclusion

Despite the many complex design requirements, this bridge will exude a clean, sleek appearance and be a landmark for the Seattle Waterfront. The structure helps unify the waterfront pedestrian routes and ties in with existing structures to the east. The remainder of the existing bridge has been modified to slightly increase the walkable width by about 1 ft 6 in.; this extra width was achieved by replacing the existing railing and concrete curb with a side-mounted, custom stainless steel railing to match the approach bridge railing. In future projects, Alaskan Way will be redeveloped to replace a surface road and include a bicycle path and a park promenade along the seawall. The new linear park will improve access to the docks and activities on the waterfront.

Visit waterfrontseattle.org for more information about the Marion Street Pedestrian Bridge and the planned changes for the Seattle Waterfront.

Don Nguyen is a senior bridge engineer and the bridges and structures sustainability leader and Bethy Clark is the Washington-area bridge and structures lead for HDR Inc. in Bellevue, Wash. Stephen Wilson is a supervisor in the Seattle Department of Transportation structural engineering services group.

Example Factor Calibration Calculations for Platoon-Permit Loads

by Dr. Jay Puckett and Dr. Joshua Steelman, University of Nebraska

In the Winter 2024 issue of *ASPIRE*®, we introduced a potential strategy based on live-load factor calibration and reliability principles to allow truck platoons to increase truck weights safely. The premise of this work is that trucks are and will become smarter and will achieve the ability to drive long distances autonomously. With such intelligence, these trucks will likely be able to report their axle weights and spacings and control their relative headways.

This article uses an example of a simple-span, concrete T-beam bridge with conventional reinforcment to review how computations might be used to support platoon-permit loads being larger than legal loads. A forthcoming article will expand these concepts to the more complex case of pretensioned concrete girder bridges, and the details related to structural reliability.

Example Bridge

Figure 1 shows a typical six-girder, conventionally reinforced concrete T-beam bridge and gives the cross-section properties for a single girder with the composite concrete deck. This bridge was selected to provide computations that are easy to follow. For more details, refer to Barker and Puckett's[1] discussion of a similar bridge.

The bridge is statically determinate. The load factors and live-load distribution factors are in accordance with the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*;[2] self-weight and future wearing-surface loads are considered, but the barrier weight was neglected for simplicity. For the sake of brevity, fatigue and shear are not considered in this example.

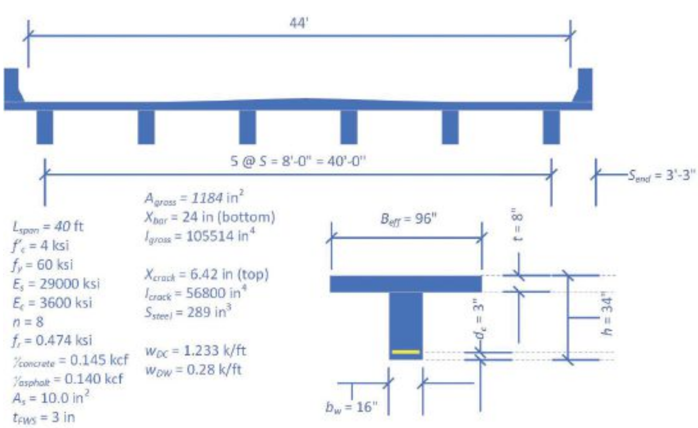


Figure 1. Example bridge. All Figures: University of Nebraska.

Table 1 shows the calculated flexural moments used in the Service I and Strength I limit states listed in **Table 2**, as well as the calculated cracking-moment check. The usual stress-block approach is used to determine the moment capacity ϕM_n. The example design used an area of reinforcing steel to exactly satisfy the Strength I limit state; that is, the performance ratio *PR* = 1 for Strength I (*PR* ≤ 1 indicates that the requirements of the limit state are met). The Service I limit state has a *PR* of approximately 1.0 using a resistance limit of 0.6 f_y. However, the analysis illustrates that the section is expected to crack under the design service load.

The flexural strength M_n is 1504 ft-kip. A quick sensitivity analysis demonstrates the effects of f'_c and f_y on this computation. If f'_c increases from 4 ksi to 6 ksi (that is, 1.5 times), M_n increases by 1.01 times to 1519 ft-kips. However, if f_y increases by 1.5 times to 90 ksi, M_n increases by 1.48 times to 2222 ft-kips. Although this behavior is well known, it is important to note that M_n varies almost linearly with A_s or f_y. From a practitioner's perspective, both A_s and f_y are perceived as deterministic because uncertainties have been integrated into load and resistance factors. In reality, f_y is significantly more influential than A_s on the reliability index β, because fabrication

Table 1. Flexural moments

Parameter	Example	Comment
M_u, ft-kip	1360	$1.25M_{DC} + 1.5\,M_{DW} + 1.75M_{distIM}$
$M_{service}$, ft-kip	856	$1.0M_{DC} + 1.0M_{DW} + 1.0M_{distIM}$

Table 2. Limit-state and cracking checks

Parameter	Example	Comment
Service I		Very close to optimal design
f_s, ksi	35.6	Sum of steel stresses due to all loadings
f_{allow}, ksi	36	$0.6f_y$
Performance ratio, *PR*	0.99	*PR* = 35.6/36 ≤ 1; OK
Strength I		Optimally designed
M_n, ft-kip	1504	*T* × (lever arm)
ϕM_n, ft-kip	1354	$\phi = 0.9$
PR	1.0	*PR* = 1360/1354 = 1; OK
Cracking		
M_{cr}, ft-kip	174	$f_r \times I_{gross}/X_{bar}$
$1.2M_{cr}$, ft-kip	209	$\phi M_n \geq 1.2M_{cr}$; OK
PR	4.1 > 1	*PR* = 856/209 = 4.1; definitely cracks

tolerances result in reinforcing steel bar areas with high accuracy (bias λ practically equal to 1) and high consistency (coefficient of variation *V* close to 0). On the other hand, actual steel yield strength is routinely higher than the nominal value used to calculate strength, and the variability of yield strength from one bar to the next can be greater than the variability of the area because bars may be produced from different heat numbers or mills.

Reliability Analysis

In terms of statistics, all design parameters have bias λ and coefficient of variation *V*. Bias factors are ratios of nominal values to expected mean values. For example, if samples of reinforcing bar with a nominal f_y of 60 ksi are taken from the field to a lab and subjected to tension testing, the samples would likely have a mean of approximately $\lambda(f_y)$ = 1.14(60) = 68.4 ksi. The standard deviation σ_R is determined from the product of the mean and V_R, as $V_R(68.4) = 0.08(68.4) = 5.47$ ksi. **Table 3** provides the statistical data used in this example. Nowak and Collins[3] provide a general discussion of structural reliability.

The following equations calculate the mean values for Strength I resistance and load effects. Coefficients of variation scale these mean values to determine standard deviations.

$$\bar{R} = \lambda_R R_n = 1.14(1504) = 1715 \qquad \sigma_R = V_R \bar{R} = 0.08(1715) = 137.2$$

$$\bar{Q}_{DC} = \lambda_{DC} Q_{DCn} = 1.05(247) = 259 \qquad \sigma_{DC} = V_{DC} \bar{R} = 0.1(259) = 25.9$$

$$\bar{Q}_{DW} = \lambda_{DW} Q_{DWn} = 1.0(56) = 56 \qquad \sigma_{DW} = V_{DW} \bar{R} = 0.25(56) = 14.0$$

$$\bar{Q}_{LL} = \lambda_{LL} Q_{LLn} = 1.2(553) = 664 \qquad \sigma_{LL} = V_{LL} \bar{R} = 0.2(664) = 132.8$$

Table 3. Statistical properties of primary variables used in this example

	Bias λ	Coefficient of variation *V*	Resistance factor or load factor
Flexural resistance *R**	1.14	0.08	ϕ = 0.9 (Strength I)
Component self-weight *DC*†	1.05	0.10	γ_{DC} = 1.25 (Strength I) γ_{DC} = 1.00 (Service I)
Wearing surface dead load *DW*‡	1.0	0.25	γ_{DW} = 1.25 (Strength I) γ_{Dw} = 1.25 (Service I)
Live load *LL*§	1.2	0.1 to 0.20	γ_{LL} = 1.75 (Strength I) γ_{LL} = 1.00 (Service I)

*Mostly dependent on f_y statistics.
†Cast-in-place concrete self-weight.
‡Wearing surface (asphalt).
§Dynamic load effect, live-load distribution, and weights/spacings are combined into one parameter here.

The limit-state function is defined as:

$$g(R,Q) = \bar{R} - \bar{Q}$$

If *g*(*R*,*Q*) is positive, the limit state is met. For simplicity, normal probability distributions are assumed, and β values are computed as follows.

$$\beta_{Strength\ I} = \frac{\bar{R} - \bar{Q}}{\sqrt{\sigma_R^2 + \sigma_Q^2}} = \frac{1715 - 259 - 56 - 664}{\sqrt{137.2^2 + 25.9^2 + 14.0^2 + 132.8^2}} = 3.81$$

Figure 2 illustrates the normal distributions for the load, resistance, and limit state function. β is the number of standard deviations the average of the limit state function is away from "failure," in this case, 3.81 times. The shaded area is the probability of not meeting the limit state, in this case, about 0.06×10^{-3}.

The Service I limit state addresses crack control in the AASHTO LRFD specifications Article 5.6.7, where steel stress is limited to $0.6f_y$, which is used here to represent Service I.

$$\beta_{Service\ I} = \frac{\bar{R} - \bar{Q}}{\sqrt{\sigma_R^2 + \sigma_Q^2}} = \frac{0.6(60)(1.14) - 10.3(1.05) - 2.33(1.0) - 23.0(1.2)}{\sqrt{3.28^2 + 1.08^2 + 0.58^2 + 5.52^2}} \approx 0$$

Another possible limit state of interest is the yield limit, or a fraction of yield to be used for rating. Using similar computations as shown for Strength I and Service I, and assuming f_y is the resistance with service load effects, β is 3.56 for the yield limit. Note that β_{Yield} is similar to $\beta_{Strength\ I}$, as

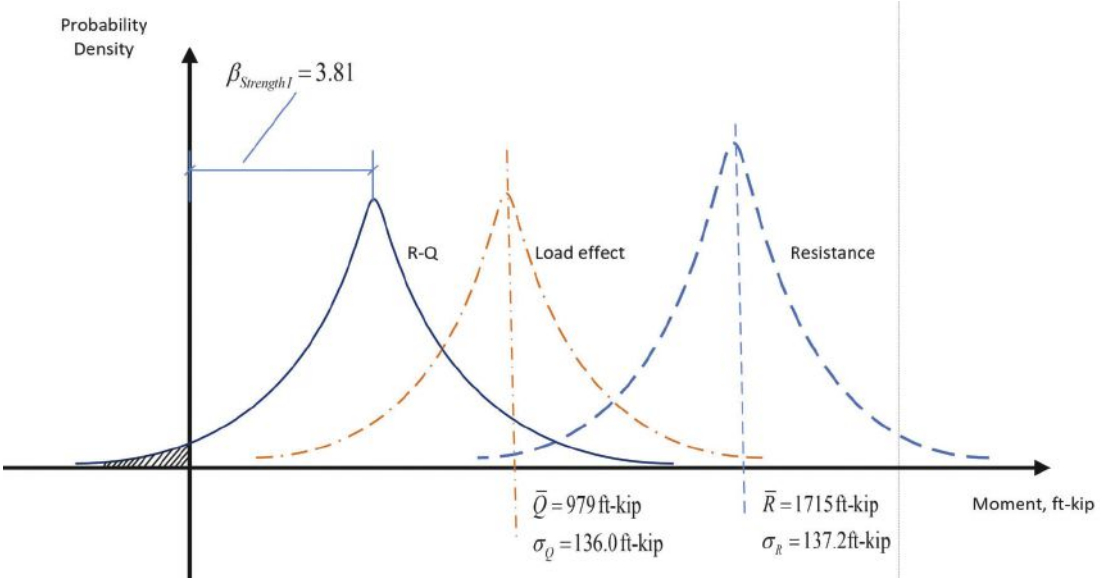


Figure 2. Normal distributions of load, resistance, and limit state functions.

expected, because the moment capacities are approximately the same.

Cracking is not a formal limit state and is provided for insight into the bridge's expected behavior during the 75-year service life. Using the same method again and assuming $\lambda = 1.1$ and $V_R = 0.1$ for M_{cr}, $\beta_{Cracking}$ is –5.6; that is, cracking is expected, which is an obvious result because even self-weight loads exceed any estimate of the cracking moment.

Parametric Study

Designs were performed for T-beam bridges with varied spacings S = 6, 8, 10, and 12 ft; span lengths L_{span} = 35, 40, 45, and 50 ft; and girder depths $h = 0.07 \times L_{span}$ (rounded up) = 30, 34, 38, and, for a deeper case, 50 in. The design was optimized for each case; that is, the performance ratio for Strength I was set to unity. The β values were almost uniform, with a minimum of 3.74 and a maximum of 3.96. This indicates that the calibration is meeting its aim (load and resistance factors are providing uniform reliability) for this class of bridge type for typical geometry and material properties. Further study regarding live-load (platoons) statistical parameters can be readily conducted with a typical structure, as β computations are insensitive to L_{span} and S.

Platoon Example

Assume that a platoon operator can measure and report axle weights and spacings while operating. Consequently, the platoon live-load bias and variance can be lower than assumed for routine operations. **Table 4** provides β values for a range of assumed bias and variance values. Note that because the live load is a function of transverse load distribution, dynamic load allowance, and weight characteristics, λ and V_{LL} cannot be driven to 1 and 0, respectively. The case where $\lambda = 1.1$ with V_{LL} between 0 and 0.1 is a realistic range with β values of 5.2 and 5.6, respectively. Other values provide bounds.

Table 4. Reliability index β with various live-load statistics

Strength I		Live load bias λ		
		1.2	1.1	1.0
Live load coefficient of variation V_{LL}	0.2	3.8	4.3	4.8
	0.1	4.8	**5.2**	5.6
	0	5.3	**5.6**	6.0

Table 5. Potential factor to apply to live load to maintain β = 3.5 (inventory)

Strength I		Live load bias λ		
		1.2	1.1	1.0
Live load coefficient of variation V_{LL}	0.2	1.00	1.10	1.20
	0.1	1.20	**1.30**	1.40
	0	1.30	**1.45**	1.60

Table 6. Potential factor to apply to live load to maintain β = 2.5 (operating)

Strength I		Live load bias λ		
		1.2	1.1	1.0
Live load coefficient of variation V_{LL}	0.2	1.30	1.40	1.55
	0.1	1.45	**1.60**	1.75
	0	1.60	**1.70**	1.90

Because β values increase with lower bias and variance, the opportunity exists to increase platoon live loads to move to a typical design target of β = 3.5, or to a typical operating target of β = 2.5.[4]

Table 5 provides the available increase in live load to maintain β = 3.5. Again, the practical ranges of bias and variance correspond to live-load increases of 30% to 45%.

In AASHTO load rating, the operating level for load rating targets β = 2.5. **Table 6** provides factors for increasing live load to maintain β = 2.5. Again, note the highlighted values.

Discussion

The example and associated computations for a single bridge type and geometry demonstrate the potential to offer a new permit-use case, a platoon permit. The load increases would depend on the operators' ability to invest in technologies to drive the λ and V_{LL} downward in a consistent and likely, reportable manner.

To summarize, when the $\overline{Q}$ and σ_Q are driven downward with better technology to $\overline{Q}_{platoon}$ and $\sigma_{Q\_platoon}$, these changes increase β. Therefore, the nominal platoon live load could be increased to maintain a constant β = 3.85, or to target a design β = 3.5 or an operating β = 2.5, as desired for various operational strategies.

$$\beta_{StrengthI} = \frac{\overline{R} - \overline{Q}_{platoon}}{\sqrt{\sigma_R^2 + \sigma_{Q\_platoon}^2}}$$

The β values for the yield limit computed for the various geometries varies a little around β = 3.5. This assumes the full yield capacity of the reinforcing steel. If the steel stress is limited to $0.6f_y$, for the example bridge, β is approximately zero for the 75-year design life, indicating a 50% chance of exceedance.

Next Article

This example and discussion provide a relatively simple bridge system to demonstrate reliability computations for a conventionally reinforced concrete bridge. A forthcoming article aims to unpack some of the complexities associated with pretensioned concrete bridge girders and evaluation for platoon operations. For a pretensioned concrete girder, the design (number of strands, prestress, and eccentricity) is more complex, involving different loss methods, different live-load factors, gross or transformed section properties, allowable tension, and design specifications that have changed over time (and are still changing).

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2. American Association of State Highway Transportation Officials (AASHTO). 2020. *AASHTO LRFD Bridge Design Specifications.* 9th ed. Washington, DC: AASHTO.
3. Nowak, A. S., and K. R. Collins. 2012. *Reliability of Structures*. New York, NY: CRC Press.
4. AASHTO. 2018. *The Manual for Bridge Evaluation.* 3rd ed. Washington, DC: AASHTO.

PROJECT

Blue Ridge Parkway over Interstate 26

by Christopher Negley, Federal Highway Administration

In October 2019, construction began to widen 16.9 miles of Interstate 26 (I-26) through Henderson and Buncombe counties in North Carolina to ease traffic congestion near Asheville. This $531 million infrastructure project, led by the North Carolina Department of Transportation (NCDOT), will improve overall roadway capacity and meet future traffic demands by widening the interstate below the Blue Ridge Parkway from 4 to 10 lanes. Because the existing seven-span bridge over I-26 conflicts with the wider interstate, it is being replaced with a precast concrete segmental box-girder bridge with a 275-ft-long main span.

After a precast concrete superstructure segment is transported to the jobsite on its open side, it is placed on a sand bed and rotated to its upright position before erection. All Photos and Figures: Federal Highway Administration.

New Alternatives

The original bridge, built in 1966 with steel I-beams and a cast-in-place concrete deck, carries Blue Ridge Parkway and the Mountains-to-Sea Trail over I-26. The original planning to widen the interstate began in the early 2000s, but a 2003 lawsuit over environmental concerns delayed the project until NCDOT revived it in 2013. In 2015, NCDOT worked with the Eastern Federal Lands Highway Division (EFLHD) of the Federal Highway Administration to complete a National Environmental Policy Act reevaluation. A draft environmental impact statement was approved in 2016. NCDOT proposed replacing this bridge over I-26, with EFLHD taking the lead on proposing new bridge alternatives and designing the new concrete segmental box-girder bridge with in-house staff.

The new Blue Ridge Parkway over I-26 Bridge is a 605-ft-long, three-span, post-tensioned precast concrete segmental box-girder structure constructed using the balanced-cantilever method. The precast concrete superstructure segments carrying the 275-ft-long main span over I-26 vary in depth from 8 to 16 ft. The piers are also composed of post-tensioned precast concrete segments. The oblong pier segments are mostly 8 ft long (one segment is 5 ft 9 in. long). They are 8 ft deep, and widths vary from 17 ft 4.5 in. to 14 ft 8.75 in. for the pier cap. All segments were match cast, and all mild steel reinforcement in

profile

BLUE RIDGE PARKWAY OVER INTERSTATE 26 / ASHEVILLE, NORTH CAROLINA

BRIDGE DESIGN ENGINEER: Federal Highway Administration, Eastern Federal Lands Highway Division, Ashburn, Va.

OTHER CONSULTANTS: Construction Engineer: COWI (Finley Engineering Group Inc.), Tallahassee, Fla.; Engineer of Record: AECOM, Raleigh, N.C.

PRIME CONTRACTOR: Fluor-United Joint Venture, Greenville, S.C.

CONCRETE SUPPLIER: Coastal Precast Systems LLC, Wilmington, N.C. (on-site batch plant)

PRECASTER: Coastal Precast Systems LLC, Wilmington, N.C.—a PCI-certified producer

POST-TENSIONING CONTRACTOR: Structural Technologies/VSL, Columbia, Md.

Rendering of the three-span Blue Ridge Parkway bridge over Interstate 26 with the Blue Ridge Mountains in the background.

View of Interstate 26 through the main-span closure pour opening. The shear keys on the precast concrete segments are visible on the left and right sides of the photo.

the superstructure and piers is epoxy coated. Superstructure segment weights varied from approximately 104 to 183 kips. Pier segment weights varied from approximately 62 to 85 kips, with the pier cap weighing approximately 88 kips.

The typical bridge section carries 10- and 12-ft-wide lanes as well as a new 5-ft-wide sidewalk to transport pedestrians safely across the bridge for the Mountains-to-Sea Trail. The new bridge will carry the Blue Ridge Parkway on a new, improved alignment over I-26 that eliminates a sharp horizontal curve near the bridge. The alignment adjustment allows this segment of the parkway to remain open to the public while the new bridge is constructed. The preliminary abutment grading work on the new alignment began in October 2020.

Innovative Thinking and Creative Delivery

Early in the design phase, EFLHD realized that one of the major challenges in delivering this bridge would be transporting the precast concrete superstructure segments to the site. With the largest segments being 16 ft in height, there would be multiple clearance issues if the segments were transported vertically from a PCI-certified precast concrete facility to the site. The team initially looked for possible batch plant locations for precasting near the proposed bridge. However, an American Segmental Bridge Institute conference in October 2017 that showcased the Sarah Mildred Long Bridge between Portsmouth, N.H., and Kittery, Maine, inspired a new idea to transport each superstructure segment on its side, eliminating the clearance issue. EFLHD presented this idea at the project prebid meeting to address contractor concerns about segment delivery. (The Sarah Mildred Long Bridge was featured in the Spring 2019 issue of *ASPIRE*®.) The contractor's construction engineer implemented a modified version of the idea: instead of a lifting frame, a sand bed was used at the jobsite to rotate the deeper segments to their upright positions in preparation for erection. The segments were analyzed for stresses during the rotation process, with the segment sections fully post-tensioned in the transverse direction before rotation. This innovative method of transporting precast concrete segments allows high-quality precast concrete construction

The Blue Ridge Parkway Bridge before the closure pour of the main span, as seen from Interstate 26. A new alignment of the parkway allows the existing parkway to remain open while the new bridge is constructed.

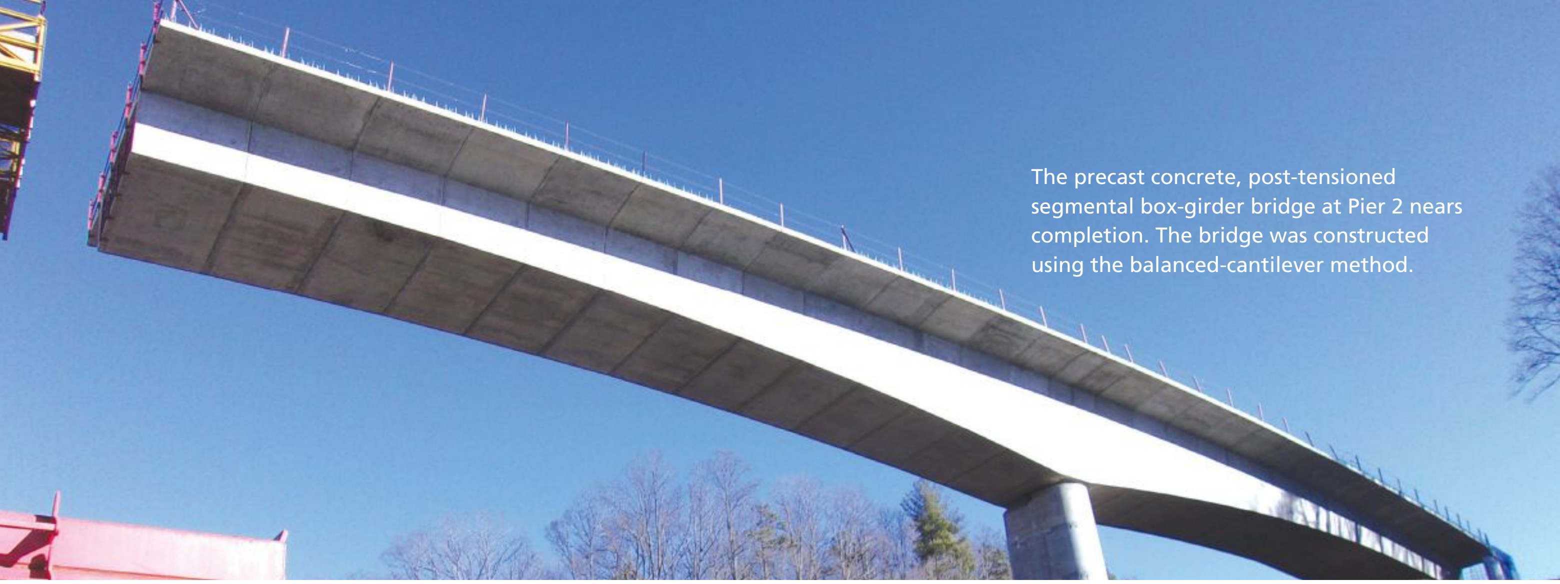
NATIONAL PARK SERVICE, OWNER

OTHER MATERIAL SUPPLIERS: Formwork: EFCO Formwork Solutions, Des Moines, Iowa; erection equipment: lifting frame: Structural Technologies/VSL, Columbia, Md.; post-tensioning: Structural Technologies/VSL, Columbia, Md.; disc bearings: RJ Watson Inc., Alden, N.Y.; modular expansion joints: Mageba USA, New York, N.Y.

BRIDGE DESCRIPTION: 605-ft-long (165-275-165), 36.5-ft-wide, three-span precast concrete, post-tensioned segmental box-girder bridge over Interstate 26 with precast concrete, post-tensioned piers

STRUCTURAL COMPONENTS: 62 precast concrete superstructure segments varying in depth from 8 to 16 ft, with three closure pours, 58 pairs of internal post-tensioning tendons, and four pairs of external post-tensioning tendons, 14 precast concrete pier segments (8 ft tall) with five internal post-tensioning tendons per pier, and cast-in-place spread footing on rock

BRIDGE CONSTRUCTION COST: Approximately $15.5 million ($700/ft^2)



The precast concrete, post-tensioned segmental box-girder bridge at Pier 2 nears completion. The bridge was constructed using the balanced-cantilever method.

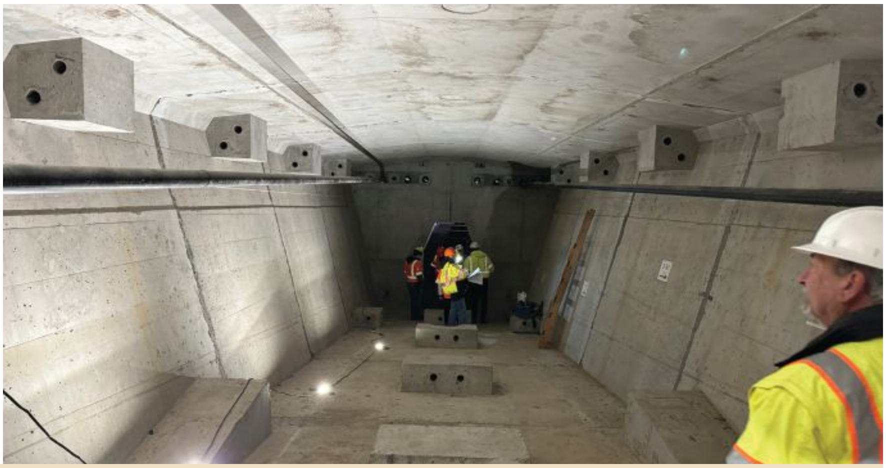
to be considered for deeper sections of longer spans.

NCDOT incorporated the bridge design into their larger design-bid-build project for widening I-26. The bridge was designed by in-house EFLHD bridge design engineers. NCDOT is administering the project, and the National Park Service (NPS) will be the owner of the bridge upon final acceptance. The close relationship between the NPS and EFLHD, based on decades of partnership, played a crucial role in ensuring that the bridge would meet NPS's aesthetic-context sensitivities. Steel formwork was required for the precast concrete pier and superstructure segments to produce smooth, clean lines, and a Natina treatment was used on the galvanized pedestrian rail to produce a rustic weathered look. The abutments were clad with stone masonry that NPS specifically approved for use on the bridge. Lastly, the same stone masonry was used on the custom-designed bridge approach parapet and guard wall.

Design Challenges

The unbalanced spans of 165-275-165 ft over I-26 caused issues with the anticipated 10,000-day, long-term tensile stresses at the soffit of the main span. The main span required eight pairs of bottom-slab, internal continuity tendons, and two pairs of external continuity tendons. Because of the parabolic curve of the superstructure soffit, the addition of internal tendons in the bottom slab resulted in increased tension in the bottom slab near the midspan portion of the 275-ft main span. The external tendons anchored high into the pier-table segments counteracted this tension and added compression back into the joints at midspan. The shorter end spans required four pairs

Interior of Pier 2 cantilever, facing the pier diaphragm.



AESTHETICS **COMMENTARY**

by Frederick Gottemoeller

Interstate 26 (I-26) through northern South Carolina, North Carolina, and eastern Tennessee is one of the most scenic stretches in the Interstate Highway System. In that same area, the Blue Ridge Parkway makes accessible one of the most attractive landscapes on the East Coast. This new bridge, built where the two corridors cross, lives up to the visual quality of its surroundings and the symbolism of its role. The bridge acknowledges the significant expansion of the highway below by spanning it all in a single 275-ft jump. That span's arched soffit creates a visual gateway to the scenic areas beyond. The same arched soffit combines with the reciprocally arched soffits of the side spans to visually reflect the loads on the structure: the bridge is thickest over the piers, where its support forces are concentrated, and thin everywhere else. The setting called for innovation, and the design team provided it, developing methods to make precast concrete segmental box construction possible in this isolated area.

The final necessary step in this endeavor was attention to the finish and details, and the design and construction team provided that, too. Steel formwork for the precast concrete pier and girder segments met the National Park Service's requirements for surface precision, whereas the stone abutment masonry and colored metal for the railings ensured a seamless fit of the new bridge into the aesthetics of the Blue Ridge Parkway. It is unfortunate that travelers on the parkway won't share with the travelers on I-26 the view of the masterpiece that is carrying them across the interstate.

The reinforced-soil retaining wall at the Abutment 2 approach will have native grasses growing on the gabion baskets.

of bottom-slab, internal continuity tendons, and one pair of external tendons. All the internal and external tendons in the superstructure are composed of twelve 0.6-in.-diameter seven-wire ASTM A416,[1] Grade 270, low-relaxation strands. Alternative strand configurations using nineteen 0.6-in.-diameter-strand tendons were evaluated in select locations. However, the benefits of maintaining the same tendon configuration, anchorages, and anchor-block designs at all locations were greater than any benefits gained from using the 19-strand tendons at some locations. For the internal top-slab tendons, one contingency duct that ran the entire length of the cantilever was provided per web. Accommodations for one pair of future external tendons per span, which would allow 22 strands per tendon, were also provided. During construction, the team implemented a contractor-proposed improvement for the external tendons by switching from the embedded bent steel pipes as originally designed to diabolos. (See the Fall 2015 issue of *ASPIRE*® for more details on the benefits of diabolos.)

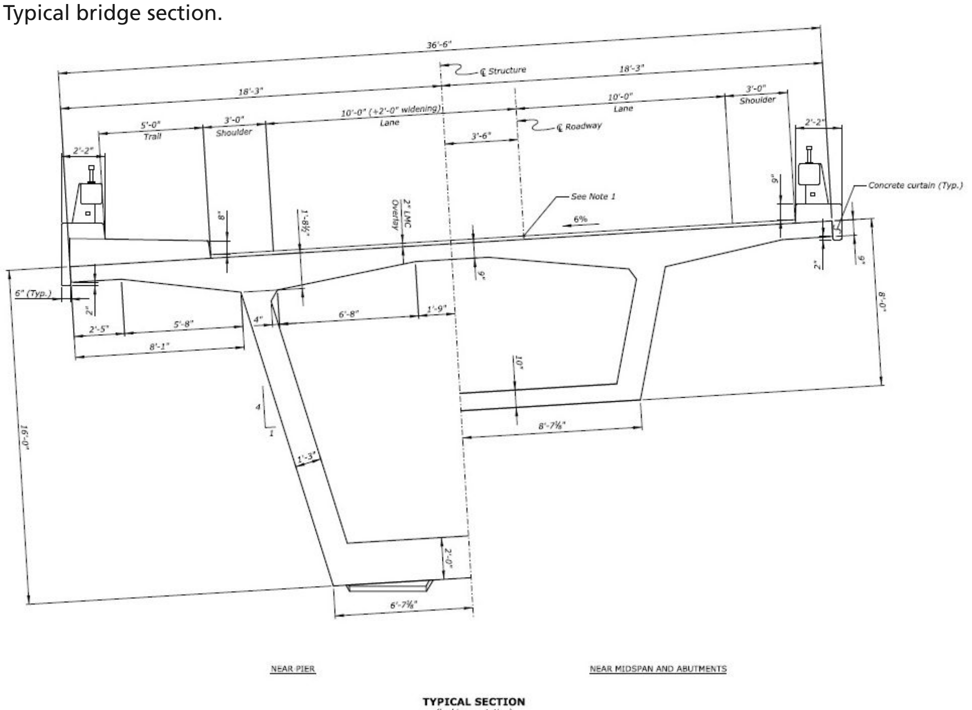
Although the new alignment allowed the Blue Ridge Parkway to remain open during construction of the new bridge, the revised alignment with significant grade changes on the approach road in the mountains of the Blue Ridge Parkway posed its own challenges.

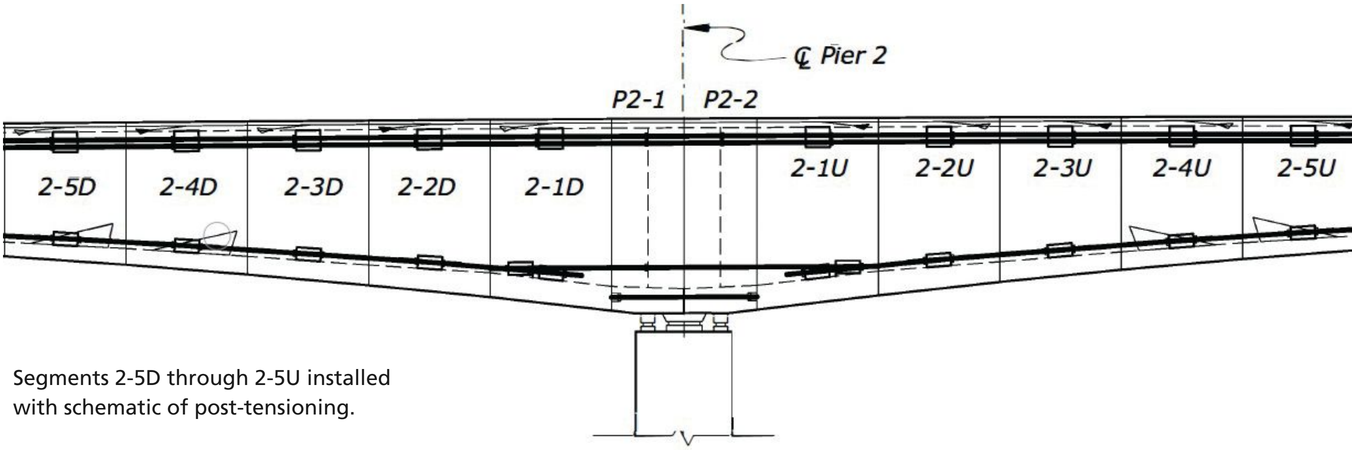
To help NPS meet its mission of preserving "unimpaired the natural and cultural resources of the National Park System for the enjoyment, education, and inspiration of this and future generations,"[2] EFLHD designed a reinforced-soil retaining wall that will eventually have native grasses growing on the gabion baskets.

Construction Challenges

A precast concrete, post-tensioned segmental box-girder bridge is itself a major project, but the I-26 bridge accounted for only about 6% of the total amount for Buncombe County's portion of the project (which involved widening a 7.8-mile stretch known as Section I-4700). Construction of the I-26 bridge had to be coordinated with traffic shifting and the widening of the interstate below. This led to some unintended delays of the I-26 bridge construction, including one in December 2022. During that delay, with superstructure segments 2-5D

Typical bridge section.





Segments 2-5D through 2-5U installed with schematic of post-tensioning.

PARTIALLY ERECTED PIER 2 CANTILEVER

through 2-5U installed, rainwater entered the cantilever. The parabolic shape of the soffit acted as a funnel for water entering the erected cantilever segments. Water then entered ungrouted ducts in the soffit in segments P2-1 and P2-2 through exposed grout tubes. The water subsequently froze inside the ducts, resulting in minor concrete surface spalling inside the segments, which was later repaired. For the repairs, the damaged concrete was removed, the post-tensioning bar duct was grouted, the exposed epoxy coating on the reinforcing bars was repaired, a bonding agent was applied, grout was placed back with a minimum 6-ksi compressive strength, the top of the repair was contoured to drain to the weep hole, and a high-molecular-weight methacrylate sealer was applied on top of the repair.

Conclusion

Except for this minor spalling and the stop-and-go nature of bridge construction, the I-26 bridge project has been moving along smoothly. In mid-May 2024, the contractor completed the final closure pours in the main span over I-26 and near abutment 1. Next, the final continuity and external tendons will be tensioned by the post-tensioning subcontractor, and the bridge rail, sidewalk, and overlay will be completed by the prime contractor. The opening date of the I-26 bridge has yet to be determined, but it is anticipated to occur in late 2024.

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2. National Park Service. 2023. "About Us." https://www.nps.gov/aboutus/index.htm.

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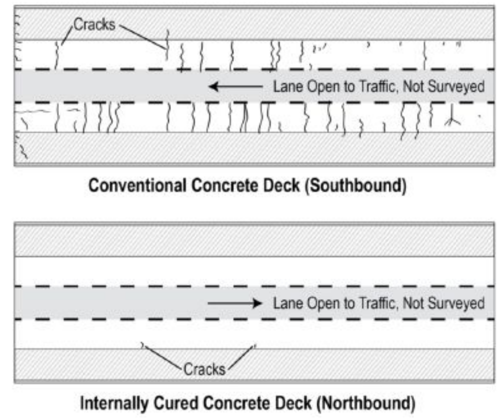
Improving Service Life of Concrete Bridge Decks using Prewetted Lightweight Aggregate

In this issue of *ASPIRE*, the Federal Highway Administration (FHWA) article by Dr. Barrett describes the "Enhancing Performance with Internally Cured Concrete (EPIC²)" initiative in FHWA's current Every Day Counts (EDC) program. This initiative highlights the relatively simple approach of replacing a portion of the conventional fine aggregate with prewetted lightweight fine aggregate to provide internal curing. The higher absorption of manufactured structural lightweight aggregate is used to carry curing water into concrete so the entire body of concrete can more fully hydrate and have the improved characteristics of well-cured concrete. The absorbed water does not contribute to the mixing water (that is, it does not affect the *w/cm*) because it remains within the lightweight aggregate until after the concrete has set and pore sizes in the partially cured cement paste become smaller than the pores within the lightweight aggregate particles. As mentioned in Dr. Barrett's article, projects in Ohio and New York have demonstrated that internal curing can significantly reduce cracking in bridge decks.

The concept of internal curing from absorbed water in lightweight aggregate is not new. It has been known to some concrete technologists since at least 1957 when the beneficial curing effects were reported for lightweight concrete in papers by Klieger and by Jones and Stephenson that were presented during the World Conference on Prestressed Concrete held in San Francisco, CA.



"Enhancing Performance with Internally Cured Concrete (EPIC²)" by Timothy J. Barrett, *ASPIRE*, Summer 2024

Replacing a portion of the conventional fine aggregate with prewetted lightweight fine aggregate to provide internal curing is a more recent approach that provides internal curing but without significantly reducing the concrete density.

More information is available on the EPIC² webpage (see ref. 3 in FHWA article), as well as on the ESCSI webpage: www.escsi.org/internal-curing/

Information on other uses of lightweight aggregate can be found at www.escsi.org

A History of Checks on Web Principal Tensile Stress in Bridge Design Specifications

by R. Kent Montgomery, GM2 Associates

All concrete bridges should be designed for the strength limit state, which controls the safety of the structure. Concrete bridges should also be designed to meet the service limit state, primarily to control cracking, which can have a direct impact on the durability of the structure. As a part of service limit state design, web principal tensile stresses must be kept below limiting stresses to help ensure that there is no shear cracking in the webs. Article 5.9.2.3.3 in the ninth edition of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[1] limits the web principal tensile stress at the service limit state to $0.110\lambda\sqrt{f'_c}$ ksi for all post-tensioned superstructures of any concrete strength and for pretensioned girders with design strengths greater than 10 ksi. (The equations shown in this article are based on values of f'_c in ksi units.) While designing for web principal tension is currently included in design specifications, that was not always the case.

In the United States, limiting web principal tension was first used in the 1970s, for the design of concrete segmental bridges. At that time, AASHTO's *Standard Specifications for Highway Bridges*[2] was the design specification used for bridge design. The AASHTO standard specifications (including later editions) included only strength-based design provisions for web shear ($V_{ci}-V_{cw}$ method) and did not include provisions that limited web principal tensile stresses. However, limitation of web stresses for service loads was used for the design of most concrete segmental bridges and was, arguably, considered standard practice. Early segmental concrete designs used the guidance provided in *Construction and Design of Prestressed Concrete Segmental Bridges* (**Fig. 1**).[3] This book, which is no longer in print, provided guidance to limit the web shear stress to $0.05f'_c + 0.20\ f_x + 0.40\ f_y$, where f'_c is the concrete strength, f_x is the horizontal longitudinal compressive stress, and f_y is the vertical compressive stress. Most often, $f_y = 0$; however, some designs introduce vertical compression in the webs through the use of vertical post-tensioning (typically, post-tensioning bars). This simplified linear version of a web stress limitation facilitates computation and has been used in numerous designs. With the advent of computers for computations, designing to limit the actual principal tensile stress became the norm. No formal limits existed, but limits between $0.0948\sqrt{f'_c}$ and $0.1264\sqrt{f'_c}$ ksi were typically used.

AASHTO's *Guide Specifications for Design and Construction of Segmental Concrete Bridges*[4] was introduced in 1989. This publication included new shear design provisions using a truss model (segmental shear design method), which was discussed in a 1995 American Segmental Bridge Institute newsletter article about the previously adopted provisions.[5] The provisions in the first edition of the segmental bridge guide specifications used a conservative limit on the maximum nominal shear (force) capacity of $0.316\sqrt{f'_c} \times b_w \times d$. Based on industry input and a review of experimental results, the second edition (1999) of the AASHTO segmental bridge guide specifications increased this limit to $0.379\sqrt{f'_c} \times b_w \times d$. (The limits for combined shear and torsion are different than the limits for shear, but they seldom govern. For brevity, this article only discusses the limits for shear.) The provisions in the AASHTO segmental bridge guide specifications also specified the conservative approach of using 45-degree diagonal compressive struts, thereby avoiding the need to provide

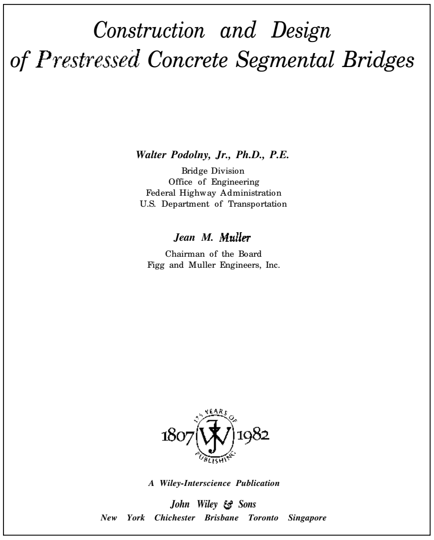

Construction and Design of Prestressed Concrete Segmental Bridges

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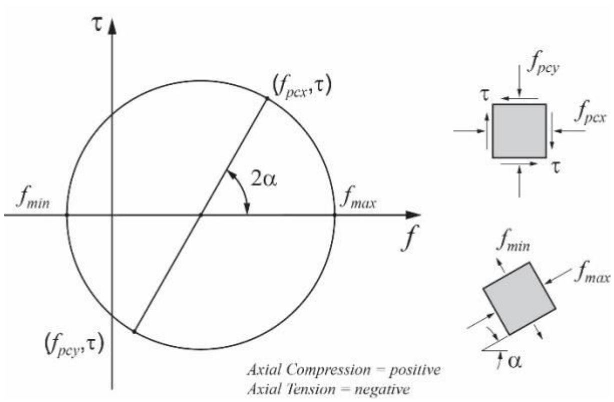
Figure 1. Early segmental concrete designs used the guidance provided in *Construction and Design of Prestressed Concrete Segmental Bridges*,[3] which included limiting web shear stress.

additional longitudinal reinforcement. The shear design provisions in the AASHTO segmental guide specifications were strength based, and there was no requirement to check web principal tensile stress for service loads; however, as previously mentioned, it was standard practice for the design of concrete segmental bridges to limit web principal tension.

The first edition of the AASHTO LRFD specifications was published in 1994. This specification introduced new shear provisions based on modified compression field theory (MCFT) as the primary shear design method. Early editions also included the $V_{ci}-V_{cw}$ method given in the AASHTO standard specifications as an alternative legacy method for shear design. The previously mentioned segmental concrete shear design method was added as an alternative legacy method in the



$$f_{\min} = \frac{1}{2}\left(\left(f_{pcx} + f_{pcy}\right) - \sqrt{\left(f_{pcx} + f_{pcy}\right)^2 + \left(2\tau\right)^2}\right)$$

$$f_{\max} = \frac{1}{2}\left(\left(f_{pcx} + f_{pcy}\right) + \sqrt{\left(f_{pcx} + f_{pcy}\right)^2 + \left(2\tau\right)^2}\right)$$

Figure 2. Mohr's circle analysis for determining principal stresses. Source: Article C5.9.2.3.3-2 of the *AASHTO LRFD Bridge Design Specifications*.[1]

2005 interim revisions to the AASHTO LRFD specifications. All of these methods are strength limit state methods, and no service limit state design provisions were included in early editions of the AASHTO LRFD specifications. The MCFT shear design method offered a potentially more economical design through a reduction in required web thickness. That is because the method uses a limit on the maximum nominal shear capacity of $0.25f'_c \times b_v \times d_v$, whereas the limit for the segmental method is $0.379\sqrt{f'_c} \times b_w \times d$. For a given web thickness, representative calculations have shown that the maximum nominal shear capacity allowed by the MCFT method is on the order of 33% larger than that allowed by the segmental method.

In the early 2000s, a few recently constructed concrete segmental bridges experienced noticeable web cracking. With the previous conservative limit on maximum nominal shear capacity, concrete segmental bridges seldom experienced web cracking. It was thought that the observed web cracking was, at least in part, due to thinner webs allowed by the MCFT method. (It should be noted that bridges properly designed for shear using the MCFT method have adequate strength and safety, and the cracking experienced was only a serviceability concern.) To address these concerns, the AASHTO T-10 Technical Committee for Concrete Design balloted two items in 2004 related to shear design in segmental concrete bridges. The first item was to add a service limit state check for principal tension in the webs of segmental bridges, with a tensile stress limit of $0.110\sqrt{f'_c}$ ksi (for normalweight concrete) to minimize the possibility of web cracking. Note that principal stresses can be calculated from closed-form equations derived from a Mohr's circle analysis (**Fig. 2**). The second item was to add the segmental shear design method to the AASHTO LRFD specifications as an alternative strength-based legacy method. These changes, primarily the web principal tensile stress limit, appear to have solved the observed web-cracking issues.

In 2013, the T-10 committee reorganized Section 5, Concrete Structures, of the AASHTO LRFD specifications to improve organization, clarity, and consistency among the articles. While undertaking this reorganization, the committee recognized that including a web principal tensile stress limit for all types of concrete bridges had value, as any concrete web could potentially crack at the service limit state. Therefore, the committee decided to add provisions limiting the web principal tensile stress to $0.110\lambda\sqrt{f'_c}$ ksi for all concrete post-tensioned bridges and pretensioned girders with concrete design strengths greater than 10 ksi. Pretensioned girders with lower concrete strengths were excluded to limit the calculational burden for girders that have a proven track record with respect to web shear cracking. However, as designers push current limits with ever deeper girders and thinner webs, it is perhaps advisable to check web principal tension for new pretensioned girder sections for which there are no historical performance data. The reorganized Section 5, including the web principal tensile-stress check for most concrete bridges, was incorporated in the eighth edition AASHTO LRFD specifications (2017).

Good concrete bridge design practice should include shear design for both the strength and service limit states. Limiting web principal tension at the service limit state and designing the web reinforcement for the strength limit state is analogous to limiting flexural concrete stresses at the service limit state and providing adequate flexural strength. Both must be performed to ensure a structure that has not only adequate strength and safety but also enhanced durability. Although it took time to develop design specifications that consistently include service limit state design provisions for shear, the AASHTO LRFD specifications now include design provisions for both the strength and service limit states.

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EDITOR'S NOTE

The modification factor for lightweight concrete λ was first included in the AASHTO LRFD specifications with the 2005 interim revisions. Therefore, the factor is included with the stress limits in this article only when referring to specifications after 2005.

A Crack Is Not a Crack: Mechanics of Reinforced Concrete

by Dr. Oguzhan Bayrak, University of Texas at Austin

There is a common misconception that all reinforced and prestressed concrete structures crack during their lifespans and all cracks are of equal significance. While some types of cracks are common and to be expected, others are not. A cracked concrete component is a structural feature that is trying to tell us its story. The significance, type, width, and the spacing of the cracks all contribute to the tale being told. The question is, can we understand this story?

This article is the first in a series on cracking in reinforced and prestressed concrete structures and why the conclusion that "a crack is a crack" can be greatly misleading. In this series, we will explore different types of cracks, and address why some cracking types and patterns are a cause for concern, why others require routine monitoring without any significant structural implications, and why some types of cracking require immediate action.

In the context of our aging infrastructure and the need to inspect and maintain our concrete bridges while maintaining the safety of the traveling public, this series will focus on the key aspects of structural behavior. To begin, let us start by focusing on the mechanics of reinforced and prestressed concrete before and immediately after cracking.

First, and as a simple example, let us direct our attention to a 6-in.-thick reinforced concrete component with no. 8 reinforcing bars spaced at 12 in. on center (**Fig. 1**). In other words, in this idealized example, we have about 1% reinforcement in the thin section. For context, it is important to note that this example is intended to facilitate discussion, rather than offer representations of best design practices.

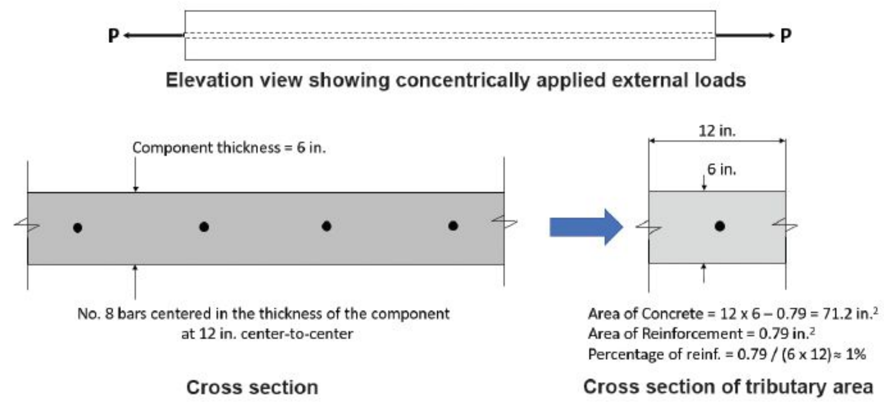


Figure 1. A 6-in.-thick concrete component reinforced with no. 8 bars at 12 in. center-to-center. All Figures and Photos, unless otherwise noted: University of Texas at Austin.

Let us assume that the compressive strength and modulus of elasticity of concrete are 5000 psi and 4000 ksi, respectively. To complete the context for this example problem, let us also assume that the tensile strength of the concrete is 400 psi. Dividing the tensile strength of the concrete by the modulus of elasticity, we can calculate the tensile strain at which cracking will occur: ε_{cr} = 0.4 ksi/4000 ksi = 0.0001 in./in. Assuming that this thin section is uniformly loaded in pure axial tension (Fig. 1), we can make the following observations:

1. Just before cracking, and assuming a perfect bond between the reinforcing bars and surrounding concrete, the strain in the reinforcing bars is equal to the strain in concrete. Multiplying this strain, 0.0001 in./in., by the modulus of elasticity of the steel reinforcement will give us the stress in the reinforcement: 0.0001 × 29,000 ksi = 2.9 ksi. For Grade 60 reinforcing bars, this stress is about 5% of the yield strength of the reinforcing bars. So, if the design criteria for this slab section include a desire to keep the section "crack free" under axial loads in service conditions, we can only use approximately 5% of the yield strength. Typical service-level stresses permitted by the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*,[1] or by any other North American structural concrete design code, is about 36 to 40 ksi. The ratio of the aforementioned numbers indicates that the desire to keep the component crack free could result in the use of impractical levels of reinforcement (that is, more than 10 times what is required). That is neither structurally feasible nor economical. Reinforcement quantities of that magnitude are not recommended in structural designs. Stated differently, before the concrete cracks, reinforcement does not significantly contribute to carrying the load in tension.
2. Upon cracking, concrete sheds the tensile force that it was carrying before cracking. In our example, this force is 71.2 in.^{2} × 0.4 ksi = 28.5 kips for the tributary area of concrete reinforced by each of the no. 8 bars (Fig. 1). Let us make a reasonable assumption that this component is supporting gravity loads, and hence the load level is expected to stay constant after cracking. Therefore, the 28.5-kip force must be picked up by the reinforcement that is crossing the cracks. This will increase the stress in

the reinforcing bars by 28.5 kips/0.79 in.^{2} = 36 ksi. This stress will add to the existing 2.9 ksi in the reinforcing bars. Stated differently, at the location of a crack, the reinforcement will do all the work in supporting the tensile load.

3. In a new loading scenario, let us now assume that the axial tension is introduced by an imposed deformation, as opposed to an externally applied load. That is, let us assume a boundary condition that introduces 0.0001 in./in. of strain due to thermal effects that are present elsewhere on the structure. With that context, we can reasonably assume that the previously described boundary conditions will be maintained and the strain level will remain at 0.0001 in./in. In this case, the loads that will be applied on the remaining portion of the structure by this 6-in.-thick slab component will decrease from (28.5 kips + 2.9 kips) = 31.4 kips to 2.9 kips. In effect, the structural cracking will relieve the restraint forces on the component by about 90%.
4. The width of the crack in the loading scenario described in item 2 would be different (and much wider) than the crack width that would be observed in the scenario described in item 3. Stated differently, the ability of the same percentage of reinforcement to control cracks will vary depending on the boundary condition (load maintained in item 2 and displacement or strain maintained in item 3). In other words, depending on the boundary conditions (imposed load versus imposed displacement), the widths of the observed cracks will be different.

Next, let us consider a scenario in which the actual compressive (and therefore tensile) strength of concrete ends up being much higher than what was specified. With additional tensile strength, the load at which the component will crack increases. That can be an advantage if the component does not crack. However, when the concrete cracks, the observed cracks will be wider than those covered in item 2. In this context, more is not necessarily better. That is, for a given percentage of reinforcement, an increase in concrete material strength that was not accounted for in the original design may lead to wider cracks. If the wider cracking takes place under service loads, that could be a concern.

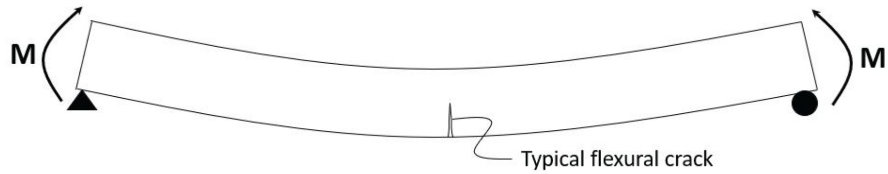


Figure 2. Elevation of a beam under external loads creating a pure-bending condition at midspan.

Next, let us consider beam bending, with the same intent of looking at the strain and stress states just before and at cracking. **Figure 2** illustrates an idealized beam-bending example that we will use to further our discussion. Let us assume the depth of the flexural tension zone in the case of pure bending is about the same as the entire thickness of the component we considered in Fig. 1, approximately 6 in., and the mild reinforcement is placed as shown. **Figure 3** shows that the concrete stress varies linearly through the depth just before flexural cracking, which implies that the force that will be released upon flexural cracking is about half of the force (due to the triangular distribution) considered in Fig. 1, if all else remains the same or comparable. With that setup for comparison established, if we use 1% flexural reinforcement in our beam, we expect the force to be picked up by the flexural tension reinforcement to be approximately half of the tension force we previously determined in the axial load case. Given that the percentage of reinforcement is the same in both examples, we expect the additional stresses that will be picked up by the flexural tension reinforcement to be less than (about half) the 36 ksi we previously calculated. Comparatively speaking, we then expect the "average" crack width to be smaller. The challenge in making this comparison relates to the fact that the flexural crack is widest at the bottom face of the beam. It is somewhat narrower at the location of the flexural tension reinforcement, and as we approach the neutral axis, the crack closes. We see no crack near the neutral axis (point of zero stress) as we move up toward the flexural compression side of the beam.

Let us now take the next step and look into the behavior of a typical pretensioned concrete beam. When typical strands are tensioned in a precast concrete plant, the

Figure 3. Strain and stress states at midspan of the reinforced concrete beam depicted in Fig. 2 just before cracking.

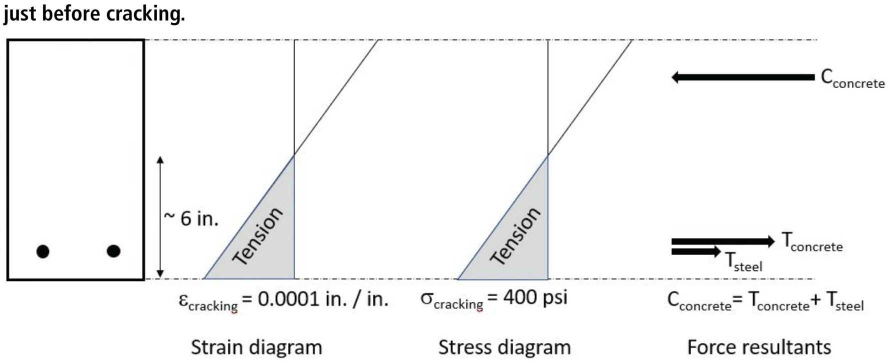
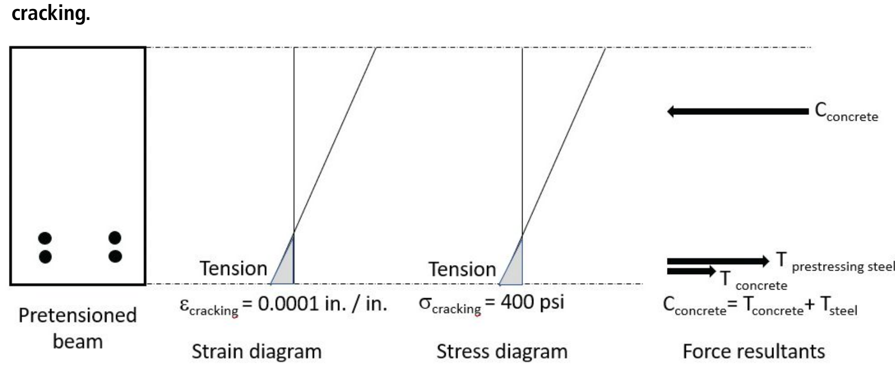


Figure 4. The strain and stress states at midspan of a pretensioned concrete beam just before cracking.



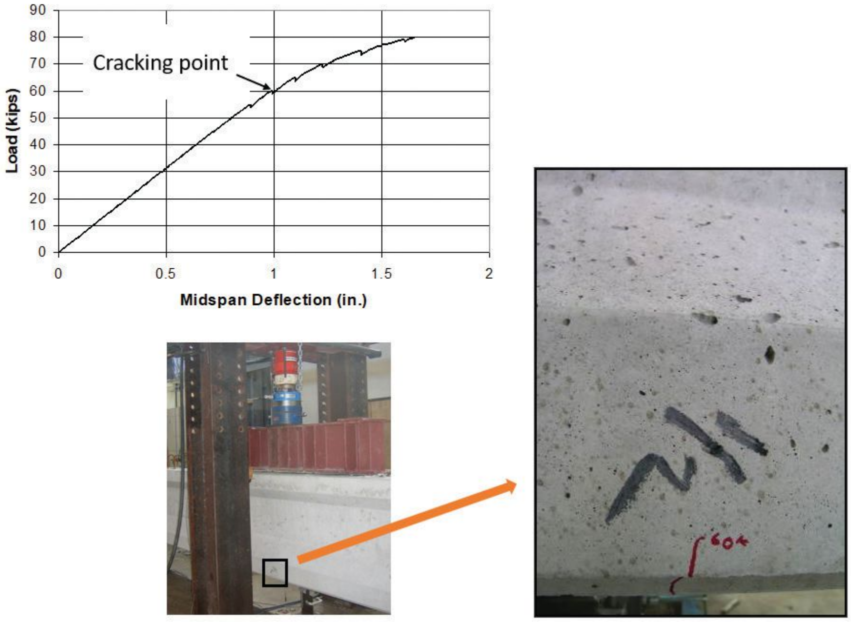


Figure 5. Cracking in a pretensioned concrete I-beam.[2]

stress in those strands is initially slightly over 200 ksi. Let us assume that after the prestress is transferred, the stress in the strands drops to 165 ksi due to elastic shortening of the beam, relaxation of the strands, and creep and shrinkage of concrete. Due to the effects of prestressing, just before flexural cracking under externally applied loads (such as those shown in Fig. 2), the neutral axis would be near the bottom face of the beam, as necessitated by the sectional equilibrium (**Fig. 4**). At cracking, the stress in the strands would be approximately 170 to 180 ksi, and as such, the incremental stress increase in the strands would be a small fraction of the initial prestress.

Figure 5 shows a load-deflection plot from laboratory testing of a pretensioned concrete beam that exhibits the gradual change of the flexural stiffness (that is, the effective EI) when cracking occurs.[2] In this setting, both the formation and propagation of flexural cracks are quite different from those observed in comparable reinforced concrete beams. A key advantage of pretensioned concrete relates to this behavioral attribute. As a consequence, the percentage change in the stress in the strands crossing the flexural cracks is much smaller in magnitude in relation to the percentage change seen in reinforced concrete beams. Naturally, this observation is coupled with smaller crack widths (Fig. 5).

As discussed, cracks differ among axially loaded components, ordinary reinforced concrete beams, and pretensioned concrete beams. A thorough understanding of loading conditions, boundary conditions, material properties, and structural behavior is a prerequisite to our analysis of cracked concrete components. In forthcoming articles, we will look at various other types of cracking.

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Updates to Lifting Loops: Provisions and Research

by Dr. Rachel Chicchi Cross, University of Cincinnati

Prestressing strand (ASTM A416[1]) is commonly used as lifting loops to transport precast concrete components from the casting yard to the project site. The prestressing strand is mechanically bent and the ends are embedded into the concrete to create loops that project out of the concrete surface and allow attachments for rigging (**Fig. 1**). The loops can be embedded with straight ends, bent ends, or broomed (flared) ends. This type of lifting device is readily available and economical, and if it is properly designed and detailed, it can achieve sufficient strength and ductility under lifting loads.

In a survey conducted in April 2019 of 35 PCI-certified precast concrete producers,[2] 95% of the participants reported that they use the safe lifting loads provided in the *PCI Design Handbook*[3] to determine their lifting-loop configurations, and 38% indicated that they had conducted their own in-house testing. The *PCI Design Handbook* and the *PCI Bridge Design Manual*[4] have long been the standard sources of guidance for safe lifting-loop practices.

Research on lifting loops is limited, and understanding in this area has been based primarily on work published in 1974,[5] as well as some short-embedment testing published in 2009.[6] None of that work included studies on lifting loops formed from 0.6-in.-diameter prestressing strand. Recognizing a need for more understanding and guidance on lifting-loop behavior, PCI funded an experimental testing program at the University of Cincinnati through the 2019 PCI Dennis R. Mertz Fellowship, as well as an extensive follow-up study to determine the safe lifting loads for 0.6-in.-diameter strand loops under vertical loads.[7,8] Inclined loading (that is, the lifting loops at an angle less than 90 degrees with respect to horizontal) was not studied in this project, but previous research provided some data on inclined load capacities.[5,6]

Updates to the *PCI Bridge Design Manual*

The fourth edition of the *PCI Bridge Design Manual*, which was released in 2023, has adopted some of the findings from the University of Cincinnati work. However, these findings could not be incorporated into the forthcoming ninth edition of the *PCI Design Handbook* in time for its publication. The primary recommendations and updates to Section 3.2.4.4.1 of the *PCI Bridge Design Manual* are as follows:

- *The factor of safety is taken as 4 for lifting loops made of conventional strand* (270-ksi ultimate strength). This is consistent with the eighth edition of the *PCI Design Handbook*.
- *The concrete strength must be a minimum of 3000 psi at the time of handling.*
- *A safe working load for a 0.6-in.-diameter single-strand loop embedded at least 24 in. is 12 kips.* Previously there was no guidance for this strand diameter.
- *A safe working load for a 0.5-in.-diameter single-strand loop embedded at least 24 in. is 10 kips.* This is consistent with the eighth edition of the *PCI Design Handbook*.
- *The use of shackles is recommended to ensure even loading of the loops in multiple-strand lifting-loop conditions. A hook or bent portion of a shackle should not be used through multiple-strand loops.* Experimental results showed a reduction in strength of at least 12% when a hook lifting device, as opposed to a straight pin shackle, is used.[8]
- *The need to crush the pipe sleeves before bending the strands of multiple-strand lifting loops is emphasized.* Sleeves fabricated from conduit or pipe are commonly used around multiple strands in a loop to keep the individual strands together and at relatively the same elevation. Locating each strand of a multiple-strand lifting loop at the same elevation is crucial to ensure even loading of the loop; failure to do so could result in progressive and premature failure of the loop.[8] **Figure 2** shows that simply bending the strands within the sleeve is not adequate to ensure even loading of the strands. The sleeve must be crushed

Figure 1. A lifting loop of conventional strand is embedded before concrete placement. Photo: University of Cincinnati.

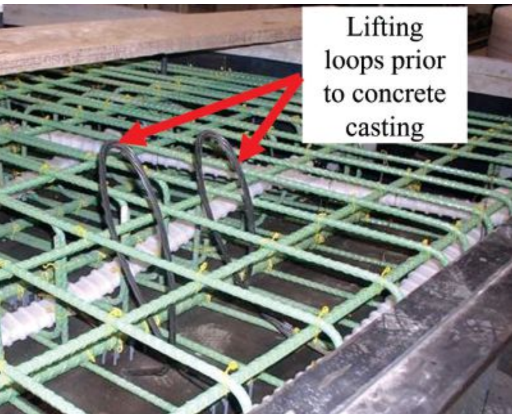

Figure 2. Comparison of multiple-strand loops with crushed and uncrushed pipe sleeves. Uncrushed pipe sleeves do not adequately keep the individual strands together and at the same elevation. Photo: Prestress Services Inc.

Figure 3. Researchers observed that failure modes varied depending on the configuration of the lifting loops. From left to right, the photos show pullout, strand rupture, and side-face blowout failure modes. Photos: University of Cincinnati.

before bending, otherwise, a consistent elevation of each strand loop cannot be maintained.

- *Mohs hardness of aggregates influences lifting-loop pullout strengths*. While soft, coarse aggregate with a low Mohs hardness (less than 3.8) was not specifically tested in the University of Cinncinati work, that type of aggregate has been shown to produce lower bond strengths than harder aggregates. The designer is cautioned to consider the bond quality of the strand being used, as well as the Mohs hardness of the aggregate, to determine whether more-conservative values of lifting-loop capacities are warranted.
- *Stainless steel strand should not be used for lifting loops because it is less ductile than conventional strand*. In cases where conventional strand cannot be used due to corrosion concerns, the factor of safety of stainless steel lifting loops should be increased from 4 to 6.

Potential Forthcoming Changes to the *PCI Bridge Design Manual*

There is a desire from the bridge community to include a safe working load for lifting loops with embedments of 36 in. or more, which is not currently in the *PCI Bridge Design Manual*. Loops with 24-in. embedments primarily fail in pullout (**Fig. 3**), where the bond between the prestressing strand and the loop is not strong enough and the strand pulls out of the concrete. This issue dictated the safe lifting loads; however, most precast concrete bridge girders are deep components that would enable longer embedments.

The University of Cinncinati investigators studied loops with 36-in. embedments and 6-in. bent legs and found that the strength of these loops were largely controlled by the rupture strength of the strand (Fig. 3).[8] In other words, by using deeper embedments, pullout could be precluded and the strength of the loop could be increased significantly. Results from this study proposed a 21-kip safe lifting load with the deeper 36-in. embedment. This proposal may not apply if the distance from the strand to the edge of the concrete component is small enough (less than 3 in.) to cause side-face blowout (Fig. 3). Note that the current *PCI Bridge Design Manual* does not specify a safe lifting load for deeper embedments, although it does permit the user to presume a uniform bond stress of 100 psi, which equates to approximately 21 kips for a 36-in. embedment with 6-in. bent legs.

The University of Cincinnati investigators also found that multipliers of 1.9 and 2.8 could be used for double- and triple-strand configurations,[8] compared with the current recommendations of 1.7 and 2.2, respectively.[3,4] It is presumed that the current recommendations were based on shorter embedments where pullout or side-face blowout would control the loop capacity. For deeper embedments, where concrete failures are precluded, the multipliers of double- and triple-strand configurations were shown to align more closely with the number of strands in the loop (that is, 2 and 3, respectively). Tests of quadruple-strand configurations resulted in a multiplier of 3.3 because side-face blowout began to control. Lifting-loop designs for bridge components with deep embedments may be able to take advantage of these increased multipliers. The current edition of the *PCI Bridge Design Manual* does not include any modifications to the original multipliers, but this issue will be considered, along with safe lifting loads, for a forthcoming addendum to the manual.

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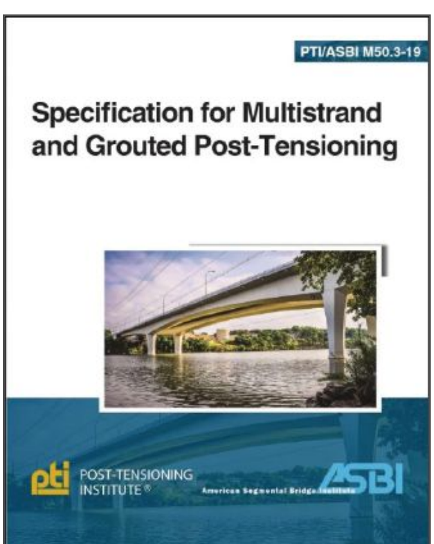
Post-Tensioning Institute Launches Post-Tensioning System Prequalification Testing and Certification Program

by Miroslav Vejvoda and Tim Christle, Post-Tensioning Institute

The Post-Tensioning Institute's (PTI's) Post-Tensioning System Qualification Testing and Certification program was developed to standardize the approval process for post-tensioning (PT) systems and provide independent certification of multistrand and grouted PT systems for use in bridges and other structures. The certification of a PT system under this program indicates that the system meets all requirements and is in conformance with PTI/ASBI M50.3, *Specification for Multistrand and Grouted Post-Tensioning*.[1] The program is intended to provide uniform objective acceptance criteria, validation that the PT systems were tested and met those criteria, and an online registry of approved systems.

The PT systems used in bridges are typically multistrand systems but also include some high-strength bar systems. In PT, high forces are applied with relatively small spaces for the anchorages, which necessitates proprietary systems with several anchorage sizes to accommodate project needs. These PT systems represent the main reinforcement for a bridge and therefore experience not only very high forces but also demands on durability based on the tendon protection level (PL) assigned for the project. There is also an increasing need for a means of monitoring a PT system in place or even for a replaceable system, which would extend the life of the structure. PTI/ASBI M50.3-19 specifies the required tests that must be successfully conducted to ensure compliance. The specification also requires that the testing be conducted or validated by an accredited laboratory to maintain high-quality standards.

PTI/ASBI M50.3-19 specifies the tests required to qualify a system.



The typical components for a commonly sized multistrand tendon will include a bearing plate, a wedge plate, wedges, strand, a grout cap, grout vents, transition trumpet, duct, and local zone reinforcement (commonly called confinement reinforcement). The force from a strand is transferred through the wedges into and through the wedge plate and then into the bearing plate. The bearing plate, together with the confinement reinforcement, transfers the load into the concrete component.

Testing Requirements

PTI/ASBI M50.3-19 specifies a series of performance tests used to initially qualify, or "prequalify," a PT system. These tests are required to demonstrate that the system can meet the various load capacities, ductility, durability, and other essential characteristics for the various components. The system used in the field must match the system tested without substitution of other components. Examples include the efficiency test, which is designed to test the strand–wedge–wedge plate connection by loading it to 95% of the actual ultimate tensile strength of the strand.

PTI/ASBI M50.3-19 also specifies the requirements and procedures in the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Construction Specifications*[2] for a special anchorage device test consisting of a concrete block with the anchorage embedded, with the local zone and skin reinforcement as required. One of the three specified test procedures—cyclic, sustained, or monotonic loading—is followed. Crack widths are measured and, at the end of the test, the force is typically increased to a minimum of $1.1F_{pu}$ ($1.2F_{pu}$ for the monotonic loading test) or to failure. F_{pu} is the ultimate load of the largest tendon that the anchorage device is designed to accommodate. Finally, the wedge plate is tested to failure at a minimum of 120% of the minimum ultimate tensile strength. Currently, it is also required that the wedge plate deflection remain under the specified limit after it has been loaded to 95% minimum ultimate tensile strength and the force released.

For PT systems that are designed to remain unbonded, additional dynamic tests are necessary. Some tests focus on the ducts. A series of duct tests is performed to validate that the material and thickness of the duct will be sufficient for typical tensioning operations using the minimum acceptable radius of curvature established by the duct supplier. Tests for flexibility of the duct, strength and function of the duct components, and pressure tests are also part of the testing protocol. Leak-tightness testing of the entire system is also required for PL2 and PL3.

Variations in State Specifications

The current requirements for PT systems and their installation vary across the United States. Accordingly, the PT system approval process and testing requirements also vary. As noted in National Cooperative Highway Research Program (NCHRP) Synthesis 562, *Repair and Maintenance of Post-Tensioned Concrete Bridges*,[3] results from a survey of bridge owners indicated that several specifications are referenced for post-tensioning. The report also noted that several states model practices and specifications of other states. One of the conclusions noted was, "While many state DOTs [departments of transportation] are referencing key guidance documents (such as PTI/ASBI M50.3 and PTI M55.1) or other states' specifications when developing or updating their own PT Specifications, non-uniformity is significant from state to state." The differences in specifications are often quite small, but the differences can sometimes be large enough to necessitate additional testing of a PT system to satisfy local requirements. If a PT system is not prequalified, the system may be reviewed and approved at the project level. Thus, there are many differences between states and projects, all requiring different acceptance criteria for the PT systems.

PTI CRT70 Prequalification Testing and Certification Program

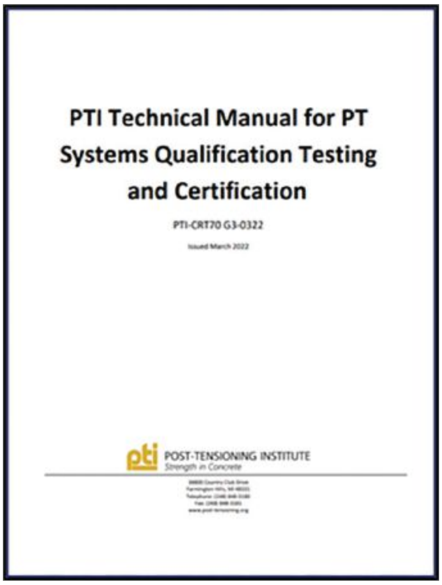
To establish a consistent method of qualification and certification for PT systems, it has been suggested that prequalification testing and certification of the systems could take place in a central location where PTI would administer the verification and certification processes. With decades of certification experience from the ANSI-accredited PTI plant-certification program, PTI launched Committee CRT-70: PT Systems Qualification Testing and Certification in 2015 to create the PTI Prequalification Testing and Certification Program.

With the general consensus of owners, PT system suppliers, engineers, contractors, manufacturers, and others, the first edition of PTI/ASBI M50.3 was published in 2012. The goal of this joint PTI-ASBI effort was to provide a national consensus-based standard that could be referenced. The second edition of the specification was published in 2019, and the committee is currently working on the third edition, expected to be published in late 2025 or 2026. Some state DOTs have adopted PTI/ASBI M50.3, with minor exceptions noted. Some major projects have also adopted PTI/ASBI M50.3 as the project specification. As noted in the Fall 2023 *ASPIRE*® article "Recently Approved Changes to the Ninth Edition *AASHTO LRFD Bridge Design Specifications*," an agenda item was approved during the 2023 meeting of the AASHTO Committee on Bridges and Structures to reference PTI/ASBI M50.3-19 and PTI M55.1-19, *Specification for Grouting of PT Structures*,[4] align the documents, and incorporate the protection level concept in the forthcoming 10th edition of the AASHTO LRFD specifications.[5]

There are three key PTI program manuals, outlined in the following paragraphs, that define the technical, administrative, and management structure requirements that the PTI CRT-70 Committee has established for this program. PTI's goal is to keep these program documents updated and to simultaneously publish new editions of each document when updates occur.

PTI-CRT70 G3-0322, *PTI Technical Manual for PT Systems Qualification Testing and Certification*,[6] addresses the requirements from the M50.3 specification and provides a checklist for each protection level for use during audits. When a PT system supplier submits a system for certification, they must include documentation matching the G3 checklist, provide all test data and evaluations, and demonstrate that the system meets all the requirements of PTI/ASBI M50.3. Each system undergoes two audits by two independent agencies. One audit will cover all the questions in the G3 checklist; the other audit will focus on the critical items in the G3 checklist, as determined by PTI CRT70. Resubmission of documentation

Night view of form travelers being used during concrete segmental construction of the 212-ft span over Sepulveda Boulevard in Los Angeles, Calif. Several spans use post-tensioned box girders (see the Fall 2023 issue of *ASPIRE*® for details). Photo: Los Angeles World Airports.



PTI Technical Manual for PT Systems Qualification Testing and Certification

PTI-CRT70 G3-0322

Issued March 2022

pti POST-TENSIONING INSTITUTE
Strength in Concrete

PTI-CRT70 G3-0322 provides a checklist for each tendon protection level for use during audits.

and possibly additional testing might be necessary when the M50.3 specification changes or when the system changes regarding materials or dimensions.

PTI CRT-140 G2, *PTI Quality Management System Manual for Certification Programs*,[7] outlines the management structure of all PTI certification programs, including the ANSI-accredited PTI plant-certification program, and the procedures for monitoring performance and quality. It provides procedures on how to handle and respond to complaints and how to resolve appeals, including time frames for several steps. In a case of a complaint from the end user, PTI collects the documentation, which is sent to the agency for evaluation. PTI determines certification based on the evaluation report from the agency. In the case of an appeal by a PT system supplier, the PTI Certification Advisory Board establishes an appeals board with a balance of interests to review the appeal. The appeals board's decision is final.

A key element of the program is an online registry of certified PT systems maintained by PTI. PTI CRT70 G1-0722, *PTI Administrative Manual for PT Systems Qualification Testing and Certification*,[8] outlines the PT system application process, audits by independent agencies, PT system certification, re-reviews, and the registry of certified systems. The registry includes the supplier's name, the system designation, including the number of strands or bar size, and a link to nonconfidential system drawings. State DOTs and other owners can access additional supporting information as required. However, access by the general public is restricted because the documentation contains proprietary information about the systems. The registry will be maintained by PTI, and any changes in certifications, such as new certifications, conditional certifications, or suspended certifications, will be reflected in a timely manner.

Conclusion

Reference to the PTI/ASBI M50.3-19 specification as part of the forthcoming AASHTO LFRD design and construction specifications,[5,9] especially as it relates to testing, will serve to establish uniform requirements for PT systems. A uniform specification will make it easier to communicate the requirements and apply them consistently to all projects. Any regional or special requirements can always be added, but a consistent baseline of system testing helps maintain a threshold of quality. Consistency also facilitates the ability of industry members to communicate and train to the current standards more effectively to ensure that the systems used in the field match the prequalified approved systems. All stakeholders are encouraged to work together on the PTI/ASBI M-50 Committee to continue to enhance the specification.

The PT system qualification testing and certification program invited PT system suppliers to initially submit up to three systems each by the end of June 2024. Currently, all submitted systems are being audited and evaluated by the independent agencies, with the results expected by the end of September 2024. Those submitted systems that meet all requirements will be certified and listed in the PTI registry in October 2024. Additional systems can be submitted for certification consideration after the initial batch of systems is certified and listed in the PT system registry.

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Fine Ceramic Inserts for Precast, Prestressed Concrete Projects

by Hiroshi Iwashita, Japan Life Co. Ltd.

Corrosion of steel anchors in concrete can lead to anchor detachment, structural damage, and loss of strength in the component in which the anchors are embedded. Therefore, to enhance the durability of concrete structural elements that use anchors, corrosion-resistant, nonmetallic inserts such as fine ceramic inserts (FCIs) are of interest to engineers and contractors. FCIs are made of 96% alumina, a material whose composition and hardness are stable over time. Alumina is a nonmetallic, ceramic compound also known as aluminum oxide (Al_2O_3), which does not react in fresh concrete. FCIs are manufactured using a one-piece molding method, which makes the inserts consistent and relatively flawless in their quality. In addition, standard jigs are available to facilitate the installation of FCIs in formwork.

FCI Capacity

Table 1 shows the shear (thread-stripping) strength of FCIs compared with steel nuts and demonstrates that the performance of FCIs is comparable to that of steel inserts. Investigators at the University of Houston[1] evaluated the tensile strength of FCIs using the concrete cone breakout method specified in the American Concrete Institute's *Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318R-19)*.[2] FCIs have not yet been evaluated for seismic loadings.

The University of Houston study was an experimental investigation of the tensile and shear concrete breakout capacities of a single cast-in FCI. The tensile tests were performed with FCIs located at the centers and edges of concrete blocks. The shear tests were performed with inserts positioned at varying distances from the block's edge.

Test results from the University of Houston investigation show that the strength of an FCI can be reasonably evaluated using the formulas specified in ACI 318-19. The results from the 76 experimental tests exceeded the ACI prediction for the concrete cone breakout. The general failure modes for most of the tested specimens were similar to the predicted failure modes defined by ACI. The only exception was that in the shear tests, the FCI experienced high stresses at the top part of the insert, which caused the FCI to fracture.

Thus, the results from the experimental program within the scope of this work show that the use of the ACI 318-19 equation will provide a relatively conservative estimate of the concrete capacity of the FCI insert based on only unfactored capacities. Further studies and reliability analyses are needed to assess the use the ACI 318-19 factors or evaluate new factors for the FCI insert system.

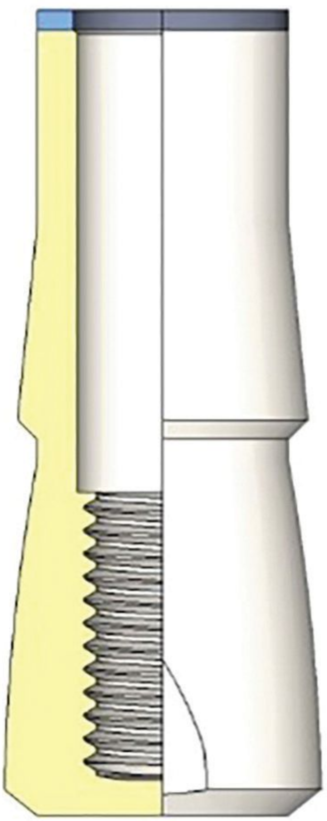


Illustration of a fine ceramic insert. All Figures: Japan Life.

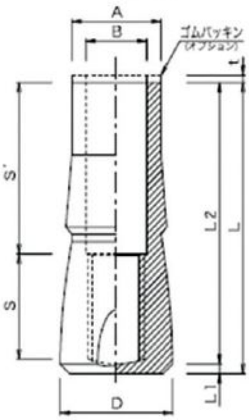
Salt and Alkali Resistances

FCIs have been tested to assess their resistance to salts and alkalis. In accordance with the technical evaluation certificate "Construction Methods for

Table 1. Comparison of the shear (thread stripping) strength of a fine ceramic insert with steel nuts.

Fine ceramic insert size	Fine ceramic insert breaking load, kN	Fine ceramic insert breaking load, kip	Shear (thread-stripping) capacity of steel nut, kN	Shear (thread-stripping) capacity of steel nut, kip
M8	29.9	6.7	21.6	4.9
M10	56.5	12.7	34.2	7.7
M12	80.1	18.0	51.4	11.6
M16	130.9	29.4	95.8	21.5
M20	216.6	48.7	154.4	34.7
M22	289.0	65.0	190.9	42.9
M24	342.6	77.0	222.4	50.0

Note: The specified shear capacity values for steel nuts are based on the Japanese Industrial Standard JIS B1052, *Mechanical Properties of Steel Nuts*, strength category 5 (nuts, coarse thread).



Standard Dimensions — in mm

Product's Code (FCI)	Screw	FCI (Main body)							
		L	L2	L2	D	A	B	S	S'
M10N x 43	M10	43	2	41	22	17	11	18.5	23.5
M12N x 60	M12	59.5	2	57.5	24	19	13	21.5	32
M12N x C4	M12	79	2	77	24	19	13	21.5	54.5
M16N x 65	M16	65.5	2	63.5	33	25	17	24	38.5
M16 x 75	M16	75.5	2	73.5	33	25	17	24	48.5
M16N x 85	M16	85.5	2	83.5	33	25	17	24	58.5
M16N x C111	M16	106	2	104	33	25	17	24	79
M20N x 100	M20	100	3	97	42	28	21	33	63
M22N x 110	M22	110	4	106	45	31	23	37	69
M24N x 120	M24	120	4	116	50	33	25	40	75

Fine ceramic insert types and dimensions.

Hanging Scaffoldings of Prestressed Concrete with Application of Fine Ceramic Insert," issued by the Japan Institute of Construction Engineering in November 1988, it was confirmed that salt and alkali resistance pose no problems for FCIs.

Financial Implications

FCIs are mainly manufactured in Japan and China. The costs of FCIs have been successfully controlled to be equal to or less than the cost of stainless steel inserts of the same size. While the price of stainless steel fluctuates according to the market prices of nickel and chrome, the cost of alumina materials is quite stable.

Conclusion

The experimental results showed that FCI anchors performed well, providing concrete breakout capacity that conservatively satisfies the requirements of the ACI 318-19 equations. Further finite element method analysis will be carried out at the University of Houston to provide a capacity table for shear and tension, as well as equations for edge conditions.

FCIs can contribute to the longevity and durability of U.S. concrete bridges because their corrosion resistance helps prevent potential paths for deterioration.

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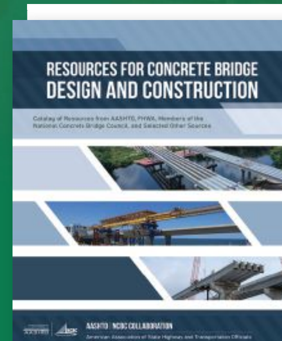
2. American Concrete Institute (ACI). 2019. *Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318R-19)*. Farmington Hills, MI: ACI.

Hiroshi Iwashita is a sales executive at Japan Life Co. Ltd in Tokyo.



Post-Tensioning for the North Split Reconstruction Project

by Melissa M. Mariano, DYWIDAG-Systems International

The North Split Project in downtown Indianapolis, Ind., has signifcantly enhanced infrastructure and improved traffic flow in central Indiana by improving the Interstate 65/Interstate 70 interchange. (For an overview of the North Split Project, see the Spring 2024 issue of *ASPIRE*®.) A key component of this project involved the use of modern construction methods to construct an integral bent cap with longitudinal post-tensioning. The interchange geometry was a challenging constraint, which led to some bridges with long spans, high skews, and limited available clearance. The straddle bent addressed these challenges where Interstate 65 spans an Interstate 70 entrance ramp at a sharp 74-degree skew.

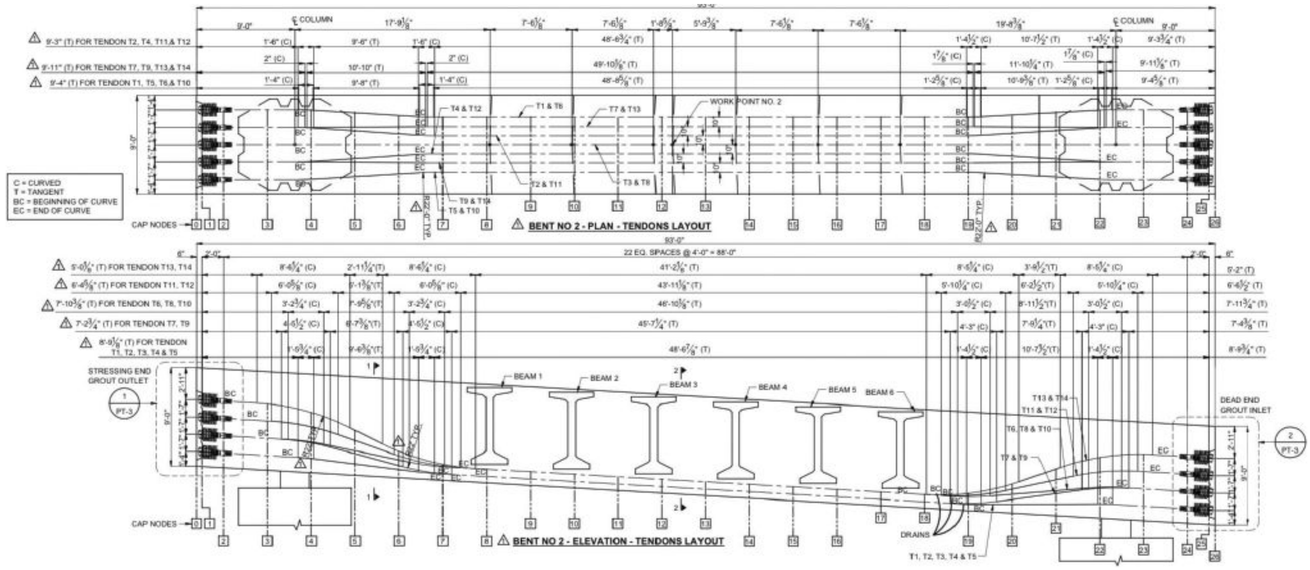
Introducing the post-tensioned straddle bent into the design allowed for longer structure span lengths while maintaining a shallower structure depth and providing the required minimum vertical clearance for the roadway traffic below. The use of the post-tensioning system was essential for supporting the heavy traffic loads passing through this interchange. For the cast-in-place concrete straddle bent that spans approximately 93 ft using a 9 ft × 9 ft cross section, the team used an innovative DYWIDAG multistrand-system design consisting of 14 tendons, each with nineteen 0.6-in.-diameter strands. One challenging aspect of the design was the prestressed concrete beams that are supported by the intermediate bent and whose ends are built integrally into the cast-in-place concrete bent cap. The versatility of the tendons allowed the tendon profile to accommodate the prestressed concrete beam ends that were embedded in the pier cap. After concrete placement, tensioning of the post-tensioning tendons could proceed in a single phase when the bent cap concrete reached a strength of 6000 psi and the deck concrete reached 4000 psi. A technician certified by the American Segmental Bridge Institute (ASBI) performed the tendon grouting in accordance with the grouting procedures specified in the Post-Tensioning Institute/ASBI's PTI/ASBI M50.3, *Specification for Multistrand and Grouted Post-Tensioning,* and PTI M55.1, *Specification for Grouting Post-Tensioned Structures*.[1,2]

Originally, DYWIDAG's scope was for material supply only. However, that scope expanded when the contractor, Superior Construction, approached

A colloidal grout mixer was used to prepare the grout. The tendon grouting was performed by certified personnel in accordance with Post-Tensioning Institute/American Segmental Bridge Institute grouting specifications.

The approximately 93-ft-long straddle bent used 14 longitudinal post-tensioning tendons—each with nineteen 0.6-in.-diameter strands—within the 9 ft × 9 ft cross section.

Falsework towers (far left) supported the prestressed concrete beams and the formwork of the straddle bent as the integral, cast-in-place concrete, post-tensioned cap was constructed. All Photos and Figures: DYWIDAG-Systems International.



The plan and elevation views of the straddle bent cap show the challenging geometry and limited available space for the 14 post-tensioning tendons. The post-tensioning system ensured that the minimum vertical clearance requirement could be met.

DYWIDAG to provide expertise in implementing the post-tensioning system. In addition to materials, DYWIDAG provided post-tensioning shop drawings, theoretical elongation calculations for tendon installation and tensioning, and a grouting plan. The partnership flourished under the expert guidance of DYWIDAG's project and field management team, whose expertise in post-tensioning systems was instrumental in delivering outstanding results. This collaboration helped the construction proceed smoothly throughout material supply, installation, tensioning, and grouting of tendons and exemplifies the importance of industry partnerships in delivering successful infrastructure projects that meet high standards of quality and safety.

With the innovative use of incorporating an engineered multistrand post-tensioning system into a straddle bent, in which the supported beam ends are embedded, the North Split Project sets a benchmark for future expansion of post-tensioning methodology in transportation projects.

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1. Post-Tensioning Institute (PTI) and American Segmental Bridge Institute (ASBI). 2019. *Specification for Multistrand and Grouted Post-Tensioning*. PTI/ASBI M50.3-19. Farmington Hills, MI: PTI.
2. PTI. 2019. *Specification for Grouting of Post-Tensioned Structures*. PTI M55.1-19. Farmington Hills, MI: PTI.

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Concrete Materials for Bridges at the Concrete Bridge Engineering Institute

by Dr. Kevin Folliard, Dr. Thano Drimalas, Dr. Racheal Lute, Dr. Oguzhan Bayrak, and Gregory Hunsicker, Concrete Bridge Engineering Institute

The vision for the Concrete Bridge Engineering Institute's (CBEI's) Concrete Materials for Bridges course was described in the Winter 2023 issue of *ASPIRE*®. The first edition of that course, which took place January 2–3, 2024, received excellent feedback, and the upcoming course schedule has been released. This course is an important piece of the first pillar of CBEI's three-pillar concept.

Course Overview

The Concrete Materials for Bridges course covers a significant amount of information in a two-day, in-person format. The course is held at the CBEI facility within the University of Texas at Austin's J. J. Pickle Research Campus. The program begins with a Concrete 101 module, which provides a basic introduction to concrete and its consitutent components. It then delves into more detail on constituent materials—portland cements, supplementary cementitious materials, chemical admixtures, and aggregates—and their sources and interactive chemistry. After providing information on all the materials that make up concrete, the course shifts to focus on the fresh, hardened, and durability properties that are most relevant to bridge design, construction, and maintenance. The emphasis throughout the course is on potential durability issues that can affect bridge components and how to design for the intended service life.

The course highlights recent trends, such as the proliferation of portland-limestone cements and the ongoing shortage of fly ash, and explores new technologies, such as the use of ultra-high-performance concrete (UHPC) in accelerated bridge construction. Changes related to achieving sustainability goals are noted. Other important topics include concrete cracking, internal curing, alkali-silica reaction, sulfate attack, and delayed ettringite formation. The relevance of each of these durability issues to various bridge components is described, with an emphasis on how to build bridges that can meet or exceed their target design lives.

Hands-on Exercises and Tours

The Concrete Materials for Bridges course includes opportunities for hands-on interactions, including mixing small batches of concrete with various admixtures, touring the concrete outdoor exposure sites and the Concrete Materials Laboratory, and using the ConcreteWorks software in a small-group setting.

Various hands-on demonstrations were presented during the initial course, including demonstrations of the effects of various supplementary cementitious materials and chemical admixtures on heat generation, workability, and setting time (**Fig. 1**). A demonstration of the strength and ductility of UHPC was also given, culminating in the students "walking the plank" (**Fig. 2**).

Course participants also toured the outdoor durability exposure sites,

Figure 1. Participants in the Concrete Materials for Bridges course mix premeasured amounts of cementitious materials and water in a hands-on demonstration showing heat of hydration. All Photos: Concrete Bridge Engineering Institute.

Figure 2. A course participant "walks the plank" in a demonstration of the strength and ductility of ultra-high-performance concrete.

Figure 3. During a tour of concrete exposure sites, Dr. Thano Drimalas describes expansion measurements on specimens.

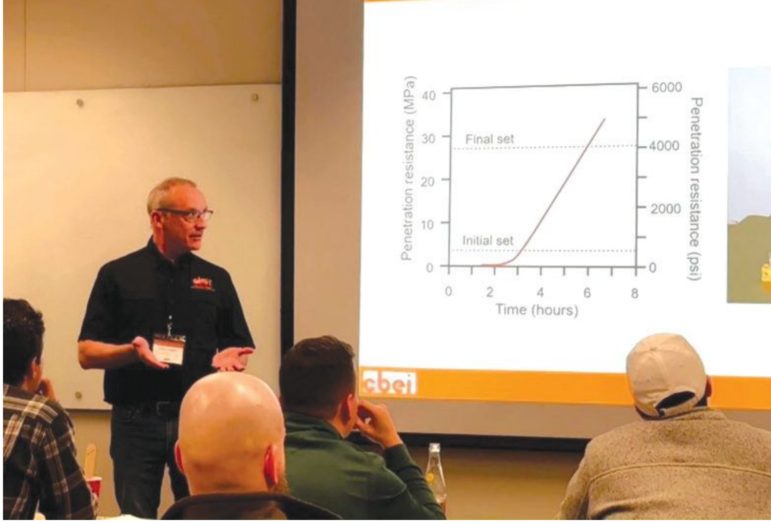

Figure 4. Dr. Kevin Folliard presents classroom instruction. Each participant receives a notebook with course slides and other pertinent information.

known as "The Cementary," and saw firsthand the detrimental impact of durability issues such as alkali-silica reaction, delayed ettrinite formation, and external sulfate attack (**Fig. 3**).

On the second day of the course, ConcreteWorks, a software program developed at the University of Texas at Austin with funding from the Texas Department of Transporation, was demonstrated to the class. Then, the participants broke into small groups to design the concrete mixture proportions for a bridge pier in a marine environment with the goal of achieving a 100-year service life. After completing their designs, the groups presented their approaches to the class, highlighting how they proposed to prevent thermal cracking, delayed ettringite formation, and chloride-induced corrosion. Because each group was given different constraints, such as being subjected to a shortage of supplementary cementitious materials, each team developed unique solutions to this challenging problem. Despite the intentionally challenging scenarios each group faced, they all successfully met the goals of the project.

Instructors and Students

Dr. Kevin Folliard (**Fig. 4**), Dr. Thano Drimalas (Fig 3), Dr. Racheal Lute (all from the University of Texas), and Anton Schindler (from Auburn University) are instructors for the course. Typically, two instructors are engaged per module, and each instructor has a wealth of research experience and industry knowledge.

The course is intended for a maximum of approximately 25 students per offering. The small class size facilitates interaction, questions, and group projects.

The course can benefit a wide range of participants, including those working in materials, structural design, inspection, and maintenance of bridges. The first class included participants from several state transportation agencies, as well as from the Federal Highway Administration (**Fig. 5**). Registration for future courses will be open to the general public as well as transportation agency participants.

CBEI Bridge Deck Construction Inspection

In addition to offering the Concrete Materials for Bridges training program, CBEI is focusing on the institute's second pillar, the Bridge Deck Construction Inspection program. This certification course is currently in development and scheduled to be available in early 2025. The course will use full-scale bridge components to illustrate the proper methods of bridge deck construction. Various construction and design defects will be incorporated within the training specimen to illustrate how they can be identified in the field before installation, and highlight their potential impact on durability. The Summer 2023 issue of *ASPIRE* provides more details of the Bridge Deck Construction Inspection Program.

For more information on CBEI and to register for courses, please visit CBEI's website: www.cbei.engr.utexas.edu.

Figure 5. Participants and instructors from the first offering of the Concrete Materials for Bridges course, held January 3–4, 2024.

The authors are all members of the Concrete Bridge Engineering Institute and colleagues at the University of Texas at Austin. Dr. Kevin Folliard is the Walter S. Bellows Centennial Professor in the Department of Civil Engineering; Dr. Thano Drimalas is a research associate; Dr. Racheal Lute is a research associate and lecturer; Dr. Oguzhan Bayrak is a Distinguished Teaching Professor, holder of the Cockrell Family Chair in Engineering #20, and director of the Concrete Bridge Engineering Institute; and Gregory Hunsicker is a research engineer.

The Positive Impact of University Research on Workforce Development

by Dr. Bruce W. Russell, Oklahoma State University

At the 2024 Precast/Prestressed Concrete Institute (PCI) Convention in Denver, Colo., Dr. Eric Matsumoto gave a presentation to the PCI Committee on Bridges highlighting the phenomenal success of the PCI Foundation's Precast Bridge Studio, which he developed at Sacramento State University in collaboration with the California precast concrete bridge industry and industry partners such as state transportation agencies. In Dr. Matsumoto's program, undergraduate students are taught prestressed concrete design, and they have the opportunity to participate in the layout, selection, scheduling, and design of precast, prestressed concrete bridges through their Senior Design Capstone projects. Dr. Matsumoto's data and testimonials demonstrate that students have flocked to the program, welcoming the opportunity for hands-on experiences with industry mentors (consultants, precasters, and contractors) and real-world applications. By all meaningful measures, the program he developed has been wildly successful. His presentation also showed how the program can serve as an outreach model for PCI and other potential industry partners seeking to connect with engineering and related technology students and address the immediate need for workforce recruitment and development. (For a comprehensive description of the PCI Foundation–sponsored Precast Bridge Studio at California State University, Sacramento, see the Spring 2023 issue of *ASPIRE*®.)

Those of us who work in academia fully understand how much time and effort Dr. Matsumoto has invested to put this program together and to sustain it over several years. The Precast Bridge Studio is a true showpiece. I personally want to recognize the incredible effort that Dr. Matsumoto invested in the Precast Bridge Studio and express my admiration for him.

As our discussion in the PCI Committee on Bridges meeting evolved, there was an implicit lingering question as to whether other universities should implement similar programs. I believe more universities will be willing to participate in similar programs with collaboration from the PCI Foundation and other industry partners, and that Dr. Matsumoto's efforts can serve as an example for others.

During the discussion, it also occurred to me that many of my friends and colleagues in the prestressed concrete industry—especially those serving on technical committees and making significant contributions to the industry through their research, designs, and personal creativity and ingenuity—were initially attracted to the field of prestressed concrete while working at a university research laboratory, often as a graduate assistant working directly with a professor. One of my purposes in writing this article is to point out that college professor "types" and their research students have helped drive the growth of the prestressed concrete industry. Professors and students working toward a research goal advance not only knowledge and science but also the development of a primed workforce for the prestressed concrete industry. So, how do these students become a part of PCI and other segments of the prestressed concrete industry?

"Research students have helped drive the growth of the prestressed concrete industry."

My own involvement with PCI and the industry began after several years of industrial work experience, mostly in the heavy construction industry, when I found myself at the University of Texas at Austin. There, under the direction of my PhD advisor—my mentor and friend, Dr. Ned Burns—my research was focused on prestressed concrete. Specifically, my research topic was the use of debonded strands in precast concrete bridges. It was Dr. Burns who took me to my first PCI Committee Days meeting in Chicago, where I presented my research on strand bond to the Bridge Committee in the spring of 1990. The Bridge Committee met in a cramped converted hotel room, where no more than 20 people gathered around a single conference table—the space was crowded, but full of energy! I remember vividly a fascinating presentation about the Shelby Creek Bridge in Kentucky. That exciting project still provides a road map for creative engineering, demonstrating firsthand the amazing possibilities of building with prestressed concrete. Given my professional experience and my PhD research topic, my decision to join PCI as a student member in 1990 was a "no-brainer." I have been an active member since that first meeting.

As a new faculty member in 1992, my first research student was the recipient of a PCI-sponsored Daniel P. Jenny Research Fellowship in 1994. We performed strand bond testing that included the "big block" test—a modified "strand-bond pull-out test"—and friction tests on prestressing strand. His work was published in the *PCI Journal*, and I am proud that he has worked in the precast concrete industry for 30 years.

Two other research students of mine were recipients of PCI-sponsored fellowships. One student made concrete cylinders that were loaded at one day of age and, based in part on that research, the concrete compressive stress limit immediately after transfer of prestress was increased from 60% to 70% of f'_c in ACI 318-08.[1] Another student worked on strand bond and helped develop the standard test for strand bond that was later adopted as ASTM A1081.[2]

I have worked with other PhD students who have continued to contribute to the precast concrete industry as professors. One is currently an associate professor at Mississippi State University. His PhD work developed high-performance concrete (HPC) with locally sourced aggregates. That work became the basis for material selection and mixture proportions for HPC in precast, pretensioned concrete bridge beams. Another former student's work focused on the need for air entrainment in HPC. He also studied prestress losses and is currently head of civil engineering at the University of Arkansas.

Among the more than 40 graduate students whom I have advised, 7 are currently teaching, or have taught, at various engineering colleges from Ohio to California, and more than half of the students have been directly involved with the precast, prestressed concrete bridge industry at some point in their careers.

So, my message to *ASPIRE* readers is that the research performed in the prestressed concrete field, whether supported by PCI or others within the precast, prestressed concrete industry, is important not only for the research itself, but for developing the workforce for the industry. I hope that I have illustrated that research funding and partnerships with universities can multiply the impact of a single research project.

In closing, I relate the following anecdote. At PCI Committee Days in 1994, I was asked how to get more professors involved in PCI. The importance of the question was not lost on me then, nor should its importance to the industry be underestimated. My answer was straightforward and to the point: provide more research funding opportunities, and the professors will come like horses to water. I believe that PCI has done that, and the continuous, committed stream of funding for university research has helped grow our industry as much as it helps develop the industry's workforce.

In this context, it is clear that the Precast Bridge Studio developed by Dr. Matsumoto should be replicated wherever possible. Dr. Matsumoto, the PCI Foundation, and the California Precast Association have provided an example to the industry of a program that is effective in attracting students. At the same time, PCI's continual efforts to support university research through research grants, and specifically through the growth of the Daniel P. Jenny Fellowship program, have also effectively advanced workforce development.

To understand the magnitude of this success, compare the 2024 Committee on Bridges to that of 1990. In 2024, bridge-related committee meetings were held in a ballroom at the Denver Convention Center for three full days, and attendance at some of the meetings surpassed 200. When I compare those meetings with my first Committee on Bridges meeting, I have to say that the industry has succeeded through the years in attracting capable, talented, creative, and committed individuals.

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EDITOR'S NOTE

In addition to studio support from the PCI Foundation, universities can apply for research fellowships to fund precast concrete research. Since 1971, PCI has awarded over 150 fellowships. These fellowships have advanced the precast concrete industry through financial support of graduate engineering students and research while also engaging faculty in the precast concrete industry, introducing students to the benefits of precast concrete, and connecting students, faculty, and PCI members for future networking opportunities. For more information, visit pci.org/Fellowships.

Approved Changes to the Ninth Edition *AASHTO LRFD Bridge Design Specifications* and New Concrete Bridge Resources

by Dr. Oguzhan Bayrak, University of Texas at Austin

This article focuses on changes to the ninth edition of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[1] regarding decks with partial-depth precast concrete panels and post-tensioned bridges that were approved at the May 2023 meeting of the AASHTO Committee on Bridges and Structures (COBS). These changes were prepared by the AASHTO Concrete Committee (previously, AASHTO Technical Committee T-10). The changes will be included in the forthcoming 10th edition of the AASHTO LRFD specifications.[2] In addition to the specification changes, this article discusses new resources for concrete bridges that have been approved by AASHTO COBS and are being developed.

Design of Decks with Partial-Depth Precast Panels in Negative Moment Regions (Working Agenda Item 226, COBS Agenda Item 28)

The use of partial-depth precast concrete panels continues to gain popularity. Accelerated deck construction, reduced labor costs, and improved construction quality and deck performance are commonly cited reasons for this trend. A recent Federal Highway Administration (FHWA) publication identifies this technology as an underused, potentially promising technology.[3] (For details of the FHWA report, see the FHWA article in the Winter 2024 issue of *ASPIRE*®.) Over the years, the use of partial-depth precast concrete panels has extended from concrete bridges to superstructures that use steel girders, curved bridge applications, and, recently, spliced-girder bridges. The use of precast concrete panels over the interior bent (negative moment region) creates unique engineering challenges. One of those challenges relates to the design of reinforcement in the negative moment regions within the decks. For continuous steel girders, Article 6.10.1.7 of the AASHTO LRFD specifications requires that the total cross-sectional area of the longitudinal deck reinforcement be at least 1% of the cross-sectional area of the concrete deck and that it be placed in two mats with two-thirds in the top and one-third in the bottom. The commentary indicates that when precast concrete panels are used as deck forms, the reinforcement placement recommendation can be waived at the discretion of the engineer.

A conservative interpretation of AASHTO LRFD specifications may result in the calculation of the top mat reinforcement on the basis of the full deck thickness (typically 8 to 10 in.) and placement of this reinforcement in the cast-in-place (CIP) portion of the deck (**Fig. 1**). This approach is overly conservative and results in impractical quantities of reinforcement. If we acknowledge that the lower portion of the deck already has well-defined joints at the panel boundaries, we can better appreciate the intent of the reinforcement placed within the CIP concrete. That intent is to control the widths of the cracks, should they form within the CIP portion. This working agenda item offers clarification that for decks with partial-depth precast concrete panels, regardless of whether the panels are supported by steel or concrete girders, the 1% reinforcement requirement should be calculated for the CIP portion of the deck and placed in that portion. The proposed changes implement the recommendations of Ge et al.[4]

It is important to recognize that in concrete spliced-girder construction, the negative moment reinforcement is provided in the form of post-tensioned tendons, and typical structural designs do not rely on the deck reinforcement as a negative moment reinforcement. It is also important to recognize that some owners, in an effort to better control the width and distribution of deck cracks in the negative moment region, use a stress limit for the deck reinforcement under service loads. For example, limiting stress in the top mat reinforcement placed in the direction of traffic to 18 to 20 ksi will result in narrower cracks than cases in which stresses are allowed to reach 40 ksi under service loads.

This change is not intended to replace all design considerations. Rather, it is intended to provide clarity on the 1% requirement in its application to decks that use partial-depth precast concrete panels.

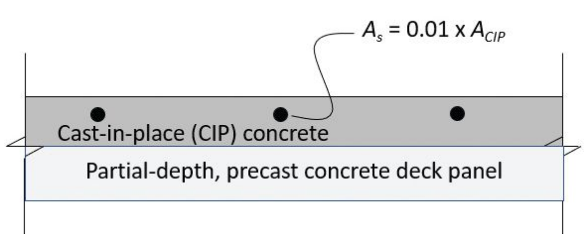

Figure 1. The forthcoming 10th edition of the *AASHTO LRFD Bridge Design Specifications*[2] will clarify the provision for longitudinal deck reinforcement in the negative moment region. When partial-depth precast panels are used, the 1% requirement should be based on the cast-in-place portion of the deck and placed in that portion. Figure: University of Texas at Austin.

Clarifications for Post-Tensioned Bridges (Working Agenda Item 228, COBS Agenda Item 26)

This working agenda item affects

Sections 5 and 10 of the AASHTO LRFD specifications and is intended to establish uniformity in bridge design and construction practices and the best available techniques recommended by the Post-Tensioning Institute (PTI) and the American Segmental Bridge Institute (ASBI). Consistent requirements for post-tensioning are intended to establish cost uniformity for expected levels of durability. Different protection levels, environmental exposure conditions, and owner-defined service life expectancy, can be invoked in design and construction. This working agenda item accommodates variations in requirements based on the aggressivity of the environmental exposure conditions or for regional requirements. The various protection levels cover conditions that include very dry conditions with little risk of corrosion, presence of freezing and thawing cycles, moderate or heavy use of deicing chemicals, and exposure to seawater and/or airborne salt.

In addition to the design and construction benefits, some of the potential benefits of standardizing specifications in the post-tensioning industry include the ability to deliver more widespread and more effective training, as well as consistent inspection and proper installation of post-tensioning systems. This working agenda item extensively references consensus-based documents such as PTI/ASBI M50.3-19, *Specification for Multistrand and Grouted Post-Tensioning*[5] and PTI M55.1-19, *Specification for Grouting of Post-Tensioned Structures*.[6] The design recommendations within this working agenda item propose changes to Section 5 of the AASHTO LRFD specifications, and the construction aspects propose changes to Section 10. Moving forward, compliance with PTI/ASBI M50.3-19 and PTI M55.1-19 will ensure compliance with the AASHTO LRFD specifications, and vice versa. See the Spring 2024 issue of *ASPIRE* for other working agenda items related to segmental concrete bridges and post-tensioning.

Resources for Concrete Bridge Design and Construction (Working Agenda Item 229, COBS Agenda Item 30)

The durability, versatility, architectural appeal, and cost advantages offered by concrete bridges have led to their widespread use in the United States and around the world. Significant research and development efforts have been funded by a variety of sponsors, including federal and state governments, industry representatives, and others. There is now a wealth of information on concrete bridges dating back to the start of the 20th century, as well as some earlier sources. The National Concrete Bridge Council (NCBC), a council of allied industry organizations, provides a vast listing of such resources on their website: https://nationalconcretebridge.org.

The document developed as Working Agenda Item 229 is the first product developed under the AASHTO and NCBC collaboration agreement. This document lists resources for concrete bridge practitioners made available by AASHTO, FHWA, NCBC members, and selected other relevant sources. It is intended to be a catalog or "bookshelf" of important resources from these organizations for the design and construction of concrete bridges. From onboarding a new generation of bridge engineers to providing resources to those who want to refresh their knowledge by studying documents that form the basis of our current concrete bridge design and construction practices, this document is a key publication. It is intended to assuage the workforce-development challenges we currently face in this country by providing a concise front-end listing of all available resources commonly used in the concrete bridge industry.

AASHTO *Guide Specifications for Structural Design with Ultra-High-Performance Concrete* (Working Agenda Item UHPC, COBS Agenda Item 29)

AASHTO developed *Guide Specifications for Structural Design with Ultra-High-Performance Concrete*[7] based on research performed by FHWA, research and development sponsored by PCI, and other research efforts in the United States and around the world. The development of the guide specification was driven by the need for explicit guidance on how to design bridges using ultra-high-performance concrete (UHPC), whose failure mechanisms may differ from those of conventional concrete. The guide specification addresses both reinforced and prestressed UHPC applications and provides the guidance that owners and designers need to predict the capacity of components in a framework that is consistent with the AASHTO LRFD specifications.. The Perspective article by Tom Murphy and Oguzhan Bayrak in the Winter 2024 edition of *ASPIRE* covers key attributes of this publication. The appendices addressing detailed material qualification and conformance testing, compatible with the design provisions in the guide specification, are currently under development and will be balloted when ready.

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7. AASHTO. 2024. *Guide Specifications for Structural Design with Ultra-High-Performance Concrete*. Washington, DC: AASHTO.

CONCRETE CONNECTIONS

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

IN THIS ISSUE

https://www.pci.org/workforce
https://www.fhwa.dot.gov/innovation/everydaycounts/edc_7/strategic_workforce_development.cfm
The Perspective articles on pages 10 and 42 discuss strategies for workforce development, a critical topic within the transportation industry in general and the concrete bridge industry in particular. The PCI Workforce Development webpage and the Federal Highway Administration's Every Day Counts EDC-7: Strategic Workforce Development webpage, available at the first and second link, respectively, offer a variety of resources and references on this topic.

https://myfwc.com/wildlifehabitats/wildlife/panther/wildlife-crossings
https://www.txdot.gov/35nexcentral.html
The webpages at these links provide information about two projects designed by Consor, the subject of the Focus article on page 5. The webpage at the first link presents the benefits and successes of the Florida Department of Transportation's strategic wildlife crossings. The second link leads to a project overview for the Interstate 35 Northeast Expansion Central Project near San Antonio, Tex. That overview includes links to the project's social media pages, which provide additional construction pictures and videos.

https://waterfrontseattle.org/waterfront-projects/marion-street-bridge
This is a link to the Waterfront Seattle webpage for the Marion Street Pedestrian Bridge, which is the subject of the Project article on page 12. The webpage includes a link to a time-lapse video of portions of the construction, as well as project presentations.

https://doi.org/10.15554/pcij69.3-03
The recently published *PCI Journal* article titled "Experimental Investigation of Multiple-Strand Lifting Loops" can be accessed via this link. This research project regarding lifting loops for precast concrete components and associated updates to the recently released 4th edition of the *PCI Bridge Design Manual* are discussed in the Concrete Bridge Technology article on page 31.

https://www.post-tensioning.org/education/ptapplications/bridges.aspx
The NCBC Member Spotlight article on page 33 features the Post-Tensioning Institute (PTI) and discusses its new program for post-tensioning system certification. PTI's website has a wealth of information about many applications for post-tensioning, including the excellent resources for bridge applications that can be found at this link.

https://cbei.engr.utexas.edu
This is a link to the Concrete Bridge Engineering Institute's website, which provides details and schedules of upcoming training programs as well as registration information. The initial offering of the Concrete Materials for Bridges course is the subject of the article on page 40.

https://www.youtube.com/watch?v=b6WREFmacaM
This is a link to the Federal Highway Administration's "Concrete Clips" video on internal curing. The FHWA article on page 47 discusses internal curing of concrete decks and presents the Enhancing Performance with Internally Cured Concrete ($EPIC^2$) initiative.

https://store.transportation.org/Item/CollectionDetail?ID=259
AASHTO recently published the *Guide Specifications for Structural Design with Ultra-High Performance Concrete*. This link leads to a webpage with a description of the publication, a link to the table of contents, and purchasing information. This new guide specification is mentioned in the LRFD article on page 44.

OTHER INFORMATION

https://nap.nationalacademies.org/catalog/27747/quality-processes-for-bridge-analysis-models
The National Cooperative Highway Research Program's *NCHRP Synthesis 620: Quality Processes for Bridge Analysis Models*, available at this link, documents the current practices of state departments of transportation (DOTs) for quality assurance and quality control of structural analysis models. The synthesis discusses processes used by in-house DOT engineers for design and processes for quality control of designs performed by consultants and submitted to the DOT.

https://nationalconcretebridge.org/2024-ncbc-webinar-series
The National Concrete Bridge Council offers a free webinar series dedicated to high-quality concrete bridge construction and stewardship. The July webinar will cover "A Review of Precast Segmental Bridge Design Resources." Visit this link to register for upcoming webinars or to view recordings of previous offerings.

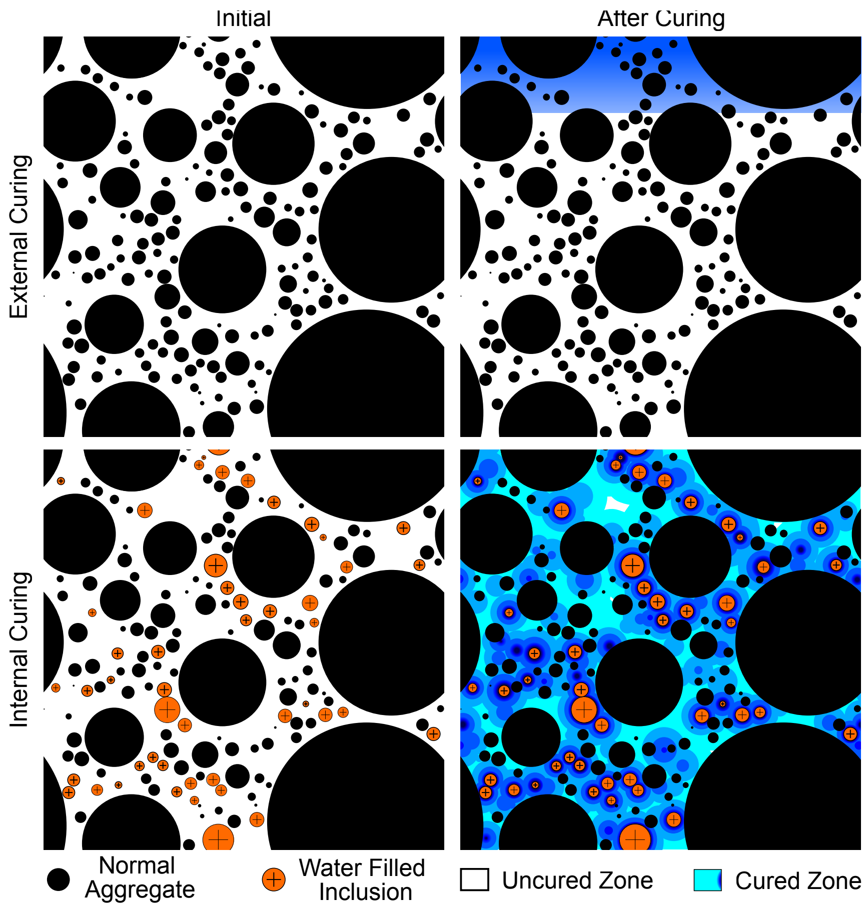
Enhancing Concrete Bridge Deck Performance with Internal Curing

by Dr. Timothy J. Barrett, Federal Highway Administration

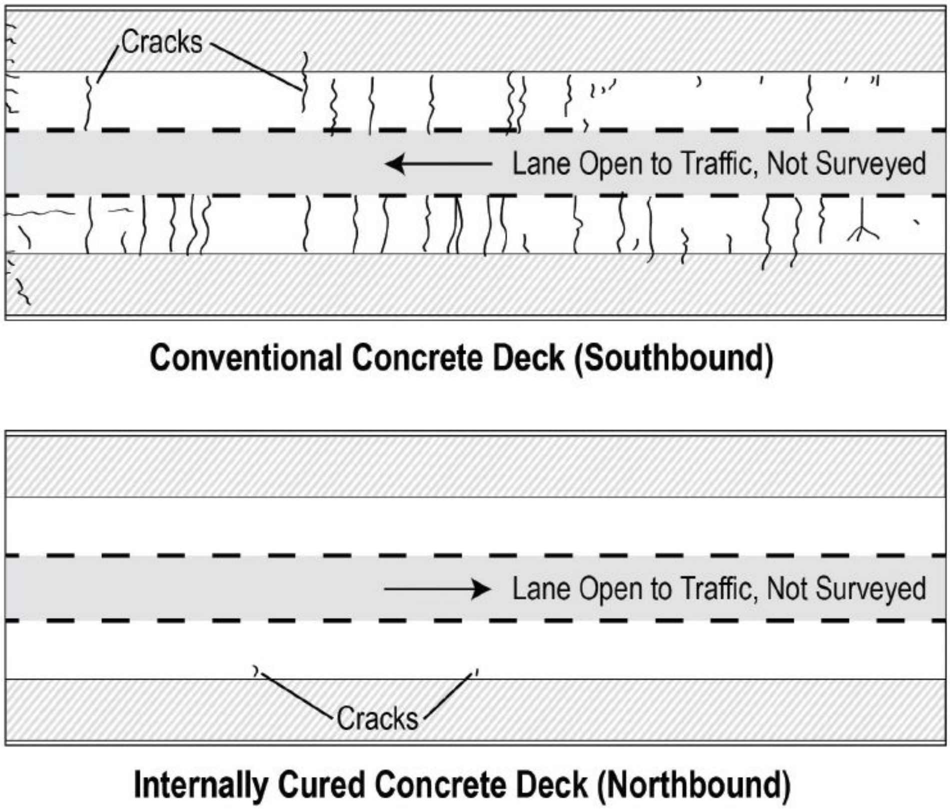
Bridge decks are frequently a limiting factor in achieving long service lives for bridges. Therefore, owners have continually innovated to find solutions to improve the performance of decks, keep them in service longer, and reduce their maintenance and preservation needs. In a recently completed U.S. Domestic Scan, *Scan 22-01: Recent Leading Innovations in Design, Construction, and Materials Used for Concrete Bridge Decks*,[1] panel members from participating state departments of transportation identified and ranked 27 innovations being leveraged to enhance the performance and extend the service of concrete bridge decks. Among these innovations is a promising technology called internal curing.

Internal curing is a concrete material–level technology that addresses the inherent shrinkage due to cement hydration (autogenous shrinkage), which is particularly challenging in concrete with a low water–cementitious material ratio (*w/cm*). While internal curing has been extensively studied in the laboratory, deployment in the field has been intermittent and the technology is more commonly known to concrete materials engineers than within the structural bridge engineering community.[2] In recognition of the technology's potential and the need for targeted deployment to accelerate its adoption, internal curing was included in the Federal Highway Administration's (FHWA's) Every Day Counts (EDC) program, under the Enhancing Performance with Internally Cured Concrete (EPIC²) initiative.[3] EDC, now in its seventh round, is a state-based model that identifies and deploys proven yet underused innovations.

All cement and supplementary cementitious materials that use hydration to gain strength inherently demand curing water. If the curing water is not supplied or readily available at the time it is needed, particularly during early ages when strength is rapidly developing, pores in the concrete begin to empty and concrete shrinkage results. This self-generated shrinkage due to internal water loss acts in the same manner as drying shrinkage due to external water loss. Internal curing effectively addresses this issue. The key difference between conventional methods of curing and internal curing in low *w/cm* concrete is that external curing results in well-cured concrete near the surface but self-drying elsewhere, whereas internal curing uniformly cures the concrete throughout its entire volume. Internal curing is achieved by hiding the required curing water in a suitable vessel within the concrete at the time of production. In practice, a portion of the normalweight fine aggregates is typically replaced with prewetted lightweight fine aggregates to take advantage of



Schematic comparison of conventional curing methods that result in well-cured concrete on the surface (top) versus an internal curing method (bottom) where the prewetted lightweight aggregates enable cured zones throughout the entire concrete volume. Figure: Federal Highway Administration.



The results of a crack survey of a bridge deck (plan view) after one year in service illustrates the enhanced performance from internal curing compared with conventionally cured concrete. Figure: Federal Highway Administration, created using data from the Ohio Department of Transportation.[8]

the high-absorption capacity of the latter type of aggregates. Alternative methods of supplying internal curing water continue to emerge in the market and include the use of superabsorbent polymers, cellulose and other polysaccharides, and some natural pozzolans such as rice husk ash. Use of internal curing technology is a design-time decision that mitigates a substantial source of concrete shrinkage, reducing the potential for cracking and providing a path to longer service life for bridge infrastructure elements, particularly bridge decks.

When the New York State Department of Transportation (NYSDOT) piloted internal curing technology in 17 multispan bridge decks from 2008 to 2013, the agency found that cracking in the internally cured high-performance class of concrete was reduced by two-thirds compared with the standard high-performance class of concrete.[4] Service-life estimations on two of the pilot installations gave the estimated time to corrosion initiation as 71.5 years on average.[5] NYSDOT has since fully institutionalized internal curing, which the agency now requires for all multispan bridge decks. (See the Summer 2019 issue of *ASPIRE*® for details of the NYSDOT internal curing requirement.) The impact of the enhanced performance is now being realized in service, with 60% of bridge decks installed in New York State in the past decade having a condition rating of 9 (excellent), the highest condition rating available.[6] A more recent example of internal curing in Ohio[7] was featured in the August 31, 2023, issue of the FHWA's biweekly newsletter, *EDC News*.[8]

With current industry guidance, the material cost increase for internal curing is anticipated to be on the order of 20%, which typically increases total project costs by less than 5%. In return, the enhanced performance from internal curing employed in higher-performance classes of concrete has been estimated to result in reducing life-cycle costs by 29% to 70%.[7,9,10]

The EPIC^2 initiative is making resources available for engineers and owners to design, specify, and construct with internally cured concrete and recommends the primary application of the technology in bridge decks.[3] Forthcoming resources include a case study summary report of selected pilot installations, a reference document for construction specification and design guidance, and an updated tool for internally cured concrete mixture design.

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