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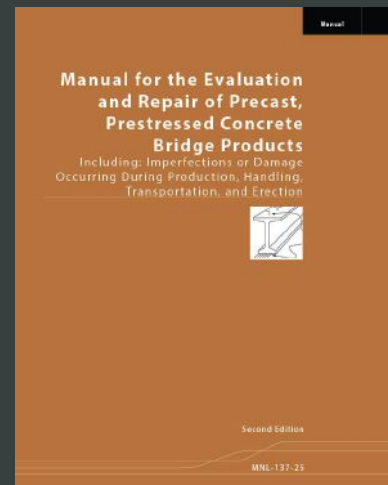
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This new manual is available from the PCI Bookstore.

<https://doi.org/10.15554/MNL-137-25>





Photo: HDR.

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Photo: PCI

In the AI Era, Do We Still Need Standards?

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

Like many of you, I've read a lot about this thing called artificial intelligence (AI). For example, I recently read an article about an AI-produced video that seems to demonstrate new farming techniques.¹ Some farmers trusted what they saw, partly because the video referenced a University of Minnesota study, and tried the new methods. Unfortunately, that study was entirely bogus, an AI hallucination.

That agricultural story got me thinking about our own industry. How do we monitor and control AI hallucinations in bridge design and construction, and how do we continue to set industry goals and expectations via standards? Our entire process and portfolio of tools depend on meticulously developed, fact-based, and laboratory-tested standards. Pick any activity from bridge-plan development through actual construction delivery, and there is a standard or required methodology to follow for success. Applying these standards requires discipline and the expert judgment of well-trained humans. How do we operate safely and efficiently and adhere to standards in the "wild West" of AI? The basic advice is to set boundaries on how AI is used. That is a good starting point, but we also need to understand that AI can, and does, fabricate data—AI tools do not just generate misinformation; they also actively spread disinformation. We cannot simply trust. We must verify.

I have been fortunate to attend several joint meetings of the American Association of State Highway and Transportation Officials (AASHTO) and the National Concrete Bridge Council (NCBC) and observe firsthand their work to develop standards and other industry documents. Of note, work on two new publications on the topic of environmental stewardship is currently underway. The first document is *Guide Specification for Conducting Environmental Life Cycle Assessment of New Bridges*. This standard, which has been drafted by the NCBC and the National Steel Bridge Alliance and edited in collaboration with the AASHTO Committee on Bridges and Structures' members and staff from several state departments of transportation, will help our industry evaluate the lifetime environmental impacts of new structures and

maximize their sustainability. The second publication, *Concrete Bridge Stewardship* (volume 1), catalogs recent concrete stewardship contributions from around the United States as well as a few well-recognized international organizations. The brain power in the meeting rooms where these documents are written and reviewed is enough to cause the lights to flicker. That is the collective energy we must maintain to develop programs and standards to protect our industry.

The success of NCBC's Concrete Stewardship initiative inspired me to investigate what other organizations are doing to strengthen and improve infrastructure programs. One impressive association is Austroads, which serves transportation agencies in Australia and New Zealand. According to its website, Austroads provides "high-quality, practical and impartial advice, information, tools and services to help our members to deliver efficient, reliable and safe mobility to their customers."² One current Austroads priority is to advance techniques in the use of low-carbon concrete, with the goal of carbon reduction in all things concrete. Further, Austroads believes that a national presence and strong, proactive leadership are required to succeed in this effort. I absolutely concur.

Bridge stewardship is a matter of real, human intelligence. The preservation of the current bridge inventory is critically important, and I applaud the collective efforts of those who create, revise, and uphold standards. Our strength is in collaboration and adherence to the standards we develop, test, and deliver. I am interested in leveraging the power of AI, but I don't want to repeat the mistakes of the farmers who didn't verify whether new techniques were based on actual facts.

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1. Stahl, L. 2026. "Is Any of This True?" *Minnesota Crop News* (blog). University of Minnesota Extension. May 27. <https://blog-crop-news.extension.umn.edu/2026/05/is-any-of-this-true.html>.
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Cover

The Y-shaped Wishbone Bridge in Austin, Tex., spans Lady Bird Lake at the eastern end of the Ann and Roy Butler Hike-and-Bike Trail. The three legs converge at a central plaza that offers scenic views and shade for users. Photo: HDR.

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CONCRETE CALENDAR 2026–2027

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

September 30–October 2, 2026

PTI Committee Days

Embassy Suites by Hilton
San Antonio Riverwalk
San Antonio, Tex.

October 11–14, 2026

ACI Concrete Convention

Hilton Atlanta
Atlanta, Ga.

October 15–18, 2026

NRMCA ConcreteWorks 2026

Gaylord Opryland
Nashville, Tenn.

November 8–11, 2026

ASBI Annual Convention and Committee Meetings

Grand Hyatt Riverwalk
San Antonio, Tex.

November 15–18, 2026

CRSI Fall Business and Technical Meeting

Westin Chicago River North
Chicago, Ill.

December 1–2, 2026

2026 World Bridge Engineering Conference

Hyatt Regency Hotel
Miami, Fla.

January 10–14, 2027

Transportation Research Board Annual Meeting

Walter E. Washington
Convention Center
Washington, D.C.

January 18–21, 2027

World of Concrete

Las Vegas Convention
Center
Las Vegas, Nev.

February 1–5, 2027

PCI Convention at The Precast Show

Salt Lake City Marriott
at City Creek
Salt Lake City, Utah

March 7–10, 2027

NRMCA Annual Convention

Marriott Louisville Downtown
Louisville, Ky.

March 21–24, 2027

ACI Concrete Convention

Caesars Palace
Las Vegas, Nev.

April 5–8, 2027

CRSI Spring Business and Technical Meeting

Hilton Beachfront Resort
Hilton Head Island, S.C.

May 3–6, 2027

PTI Convention

Lake Buena Vista, Fla.

June 29–July 1, 2027

International Bridge Conference

Nashville, Tenn.

July 11–15, 2027

AASHTO Committee on Bridges and Structures

Madison, Wis.



2026 NCBC Webinar Series

Whether you engage in bridge design, maintenance, construction, or asset management, NCBC will continue to bring you valuable insights regarding the concrete bridge industry. Each webinar typically starts at 1 p.m. ET. Visit <https://nationalconcretebridge.org> for more information and to register.

October 21: Anchoring to Concrete Part 3: Post-installed Reinforcing Bars and an Overview of Grouted Joints with Confinement

November 18: Silica Fume Sustainable / Low-Carbon Concrete

Check our website for updates and webinar recordings.

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Combating Workforce Development Issues: Delaware's Engineering Outreach Program

by Nicholas Dean and Paul Huhn, Delaware Department of Transportation



A bridge design engineer gives a tour of a recently completed bridge. All Photos: Delaware Department of Transportation.

For years, the civil engineering industry in Delaware has faced an array of workforce development issues, including an unflattering and inaccurate public perception of engineers and the industry, workforce retention challenges, and lack of diversity in the workforce. Today, the Delaware Department of Transportation (DelDOT) is committed to an engineering outreach program to address these issues.

Identifying Workforce Challenges

Engineering is often perceived to be

A project engineer gives a science, technology, engineering, and math presentation to second graders.

a profession characterized by social awkwardness, with the typical engineer being viewed as “nerdy” or “robotic.” In fact, while engineering is a highly technical field, it also requires creative thinking and collaborative skills to solve real-world problems.

Like many state agencies, DelDOT has struggled with its public image. A negative public perception can make it increasingly difficult to attract quality engineers toward careers with the department. Changing stereotypes surrounding engineers and public-sector

work is a major challenge being faced across the industry.

While employee retention is a problem that public agencies have faced for decades, DelDOT has recently had to navigate disproportionately high turnover in the engineering sector as employees have moved to the private sector or retired. In such an experience-driven field, this level of turnover results in a significant loss of institutional knowledge, increased project costs, and project delays.

Civil engineering in the United States has historically not been a demographically diverse career field. According to 2025 U.S. Bureau of Labor survey data,¹ 78% of people employed as civil engineers are white and only about 22% are female. This lack of diversity can limit innovation and hinder career advancement. Homogeneous teams tend to approach challenges with the same perspectives, which can limit the creativity required to develop new solutions or new materials. The absence of minorities and women creates a shortage of role models and mentors, which in turn can stifle the flow of talent, especially into senior management.





Members of the Delaware Department of Transportation's Bridge Design team host a Lego competition at Delaware State Fair, where they speak with competition participants, parents, and other people from all around Delaware about the agency's bridge design work.



Students test their final bridge structures at the Delaware Bridge Design Competition.

Until civil engineering adopts a truly inclusive and diverse model, the industry will continue to fall short of its potential. While DelDOT has taken great strides to rectify this disparity through its outreach efforts, a lack of diversity remains an issue.

Engineering Outreach: How It Started

DelDOT's Bridge Design section has been actively participating in student outreach with schools throughout the state for more than 10 years. However, this effort was not always as prolific and widespread as it is today. What started as a simple idea from a project engineer working on a highly publicized project is

now a statewide effort led by members of DelDOT's Bridge Design team. During the construction of the Charles W. Cullen Bridge, a cable-stayed structure spanning the Indian River Inlet south of Dewey Beach, Craig Stevens, one of the project's construction managers, saw an opportunity to use the bridge to educate Delaware's public.

Stevens began giving tours of the project to local representatives, community groups, and students in the area. These tours were a way of educating people about the project, but they also served a broader purpose by showcasing the field of civil engineering and its impact on

the world and presenting DelDOT employees in a more positive light. By allowing the public to see firsthand how engineering and the hard work by state employees shape their communities, the outreach efforts helped to generate a sense of goodwill toward the department as a whole.

Inspired by the popularity of the project tours, a team of Bridge Design engineers began giving presentations and conducting hands-on activities at elementary, middle, and high schools throughout the state. Today, these presentations cover an array of topics related to bridge construction and the work being performed by DelDOT employees around the state. The intent of these presentations is to continue to generate interest in the field of civil engineering and foster a sense of community.

Engineering Outreach: How It's Going

Today, DelDOT Bridge Design team members give dozens of science, technology, engineering, and math (STEM) presentations and reach more than 2000 students each year. In addition to the bridge engineering presentations, the Bridge Design team participates in career days, interactive STEM activities, and career fairs. These outreach efforts increase awareness of the agency's work around the state, generate interest in the field of engineering and other STEM careers, and offer a recruitment path for potential interns and future employees. Through these outreach efforts, DelDOT's goal is to interact with students who may not realize that these career opportunities



Participants, teachers, and supporters of the seventh Delaware Bridge Design Competition in 2026.



Students present their designs to a panel of practicing engineers at the Delaware Bridge Design Competition.

exist. Instead of targeting groups already likely to pursue engineering, DelDOT uses a broad outreach approach to expose as many students as possible to civil engineering. This work is guided by the principle that the unique backgrounds and experiences of these students can positively shape the engineering industry. While it is important to spark the initial interest in the field, DelDOT recognizes the need to maintain that interest throughout students' educational journeys. For this reason, DelDOT staff engage with students and mentor them as they progress through their academic pathways.

To further cultivate curiosity in engineering among Delaware's students, DelDOT holds a statewide bridge design competition based on a national competition developed by the American Association of State Highway and Transportation Officials. The event, which was first held in 2018, is a hands-on engineering competition for middle school and high school students interested in STEM fields. The program encourages such students to address real-world problems and aims

to inspire them to consider careers in transportation and civil engineering. Students are assigned a specific bridge type and tasked with preparing a research-based proposal, designing and detailing drawings using computer-aided drafting (CAD) software, creating a balsa wood bridge model, and presenting their efforts to a panel of practicing engineers.

The Delaware Bridge Design Competition is an inclusive event for schools throughout the state. There is no limit to the number of participants a school can register, and participation in the event comes at no cost to any of the schools. Because this event is free to students, schools that may not otherwise have the financial means to incorporate engineering into their curriculum can participate.

In 2018, the competition had 32 students from 5 schools. In its second year, excitement surrounding the competition ballooned, and participation grew to more than 200 students from 25 schools. Interest in the competition has continued to increase, with more than 400 students from throughout the state now participating. Over the years, almost 50% of the state's middle schools and high schools have participated. The success of the Delaware Bridge Design Competition is made possible through

the support of DelDOT volunteers, partnering consultant volunteers, and the educators whose goal of teaching STEM-related concepts to students aligns with DelDOT's outreach efforts. The competition has also become a networking opportunity for students by providing them with direct contact with industry professionals.

While the impact of these outreach efforts on students cannot be quantified, they have undeniably helped to spark an interest in civil engineering throughout Delaware's schools. DelDOT has hired multiple interns and eventual full-time staff members as a direct result of the Bridge Design team's STEM outreach and the Delaware Bridge Design Competition. The department believes that these community engagement efforts improve perceptions of the public sector, bolster interest in STEM-related career fields, and contribute to a sustainable pipeline for future engineers at DelDOT.

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1. U.S. Bureau of Labor Statistics. 2026. "Labor Force Statistics from the Current Population Survey: Employed People by Detailed Occupation, Sex, Race, and Hispanic or Latino Ethnicity." Last modified February 20, 2026. <https://www.bls.gov/cps/cpsaat11.htm>. 

Participants, teachers, and supporters of the seventh Delaware Bridge Design Competition in 2026.



So, Why Don't You Teach Bridge Engineering?

by Dr. Richard Miller

Workforce development has become an important issue in the bridge design community. The Spring 2026 issue of *ASPIRE*® included an interview with Donn Digamon, state bridge engineer with the Georgia Department of Transportation (DOT), and this issue has an article from the Delaware DOT (page 5), both on workforce development. To effectively serve our readers, *ASPIRE* will continue to publish articles on this subject.

In discussions of workforce development, representatives of state departments of transportation and consulting firms have remarked on the lack of recent graduates who have taken a bridge design course. When the public thinks of civil engineers, they often think of people who design bridges. However, most universities do not offer bridge design, and even when it is offered, students may not take the class. So, why don't more universities offer bridge design and why don't more students take the course?

Let me offer one perspective. Prior to retirement, I was the department head of Civil and Architectural Engineering and Construction Management at the University of Cincinnati. A common question from employers, alumni, students, and parents was why certain classes were offered and others were not. There are many answers to this question. University course offerings are influenced by budgets, the level of interest from students, and whether faculty are qualified to teach certain courses.

According to the U.S. Census Bureau, transportation-related construction is roughly 3% of all construction expenditures in the United States.¹ Most civil engineering students reasonably assume that they will work in the commercial sector and are not

inclined to consider bridge design courses relevant to their career plans. In contrast, courses on reinforced concrete design, which focus on buildings, are usually well-populated.

University and college budgets are very tight. Although tuition costs keep rising, tuition increases are often capped and do not offset inflation, declining government aid, unfunded government mandates, declining enrollment, increasing expenses for information technology, and rising costs of noneducational expenditures such as sports programs and student housing. Some colleges and universities face budget deficits and have laid off faculty and staff. Others have closed programs or have closed entirely. Laws in several states also mandate the closure of low-enrollment programs and/or place limits on the credit hours required for degrees.

Engineering programs have not been immune to financial challenges. My former colleagues in academia report continuing budget cuts. Available research grant dollars have been reduced or eliminated. There is little money to hire new faculty, upgrade facilities, and pay teaching assistants. Some engineering labs have been closed as funding is redirected to areas such as computer science and artificial intelligence (AI).

We are also losing advocates for bridge design education. As faculty committed to research and teaching in bridge-related areas retire or move to other positions, they are not always being replaced. In some cases, the positions are simply eliminated. In other cases, the universities opt to hire nontenured faculty or adjuncts because they are more cost effective than tenure-track appointments. When new tenure-track faculty are hired, they are often

specialists in emerging areas such as computer-aided design and AI. Traditional areas of civil engineering may no longer be attractive to universities. For some universities, the barrier to offering courses in bridge design is a lack of faculty qualified or interested in teaching those classes.

We can debate all of these matters, but as a department head, I had no influence over these internal and external factors. Each year I had to present and justify a budget. To make our budget, each class needed a minimum enrollment. Low-enrollment classes were a problem, and I was constantly asked to justify offering such classes.

Enrollment challenges were especially difficult for electives such as bridge design. From a budget perspective, if 50 seniors each needed six electives, the department should only offer six elective courses with an enrollment of 50. However, that strategy would defeat the idea of electives and would not serve the students or their future employers. My goal was to offer enough electives to serve the needs of the undergraduate and graduate students and to maximize enrollment in those classes.

At the University of Cincinnati, we offered a class called Bridge Engineering every other year. We had about 15 to 20 students, far short of the number that I needed to meet budget-related enrollment expectations, but our robust bridge research program generated sufficient income to justify the class. Most engineering schools do not have enough bridge-related research to make a bridge design course seem cost effective.

Using an adjunct professor to teach bridge design is a possible solution, as the break-even point for an adjunct's

salary is much lower than that for full-time faculty. However, it is difficult to find qualified adjuncts in engineering, and their teaching skills vary widely. Some adjuncts are only available at night, which students don't prefer, and that lowers enrollment. Also, I had to request additional funds to hire an adjunct, and justifying that funding would be difficult if the courses taught by full-time faculty were not full. Rather than offer a new class taught by an adjunct, it was more cost effective to have the students take an existing, high-enrollment class taught by full-time faculty.

So, what can industry do to help this situation? If we are going to have new bridge engineers, it is necessary for industry to find a way to make it happen. There are efforts underway. For example, the Concrete Bridge Engineering Institute (CBEI) at the University of Texas at Austin, is funded by the Federal Highway Administration, more than a dozen state highway agencies, and various construction industry organizations. CBEI's efforts at workforce development have been discussed several times here in *ASPIRE* since the program was introduced in the Summer 2022 issue. The PCI Foundation supported an excellent bridge design studio at Sacramento State University. When the foundation funding ended, PCI West and local precast concrete producers came together to ensure that funding continues. These types of large-scale efforts are helpful, but there are some smaller-scale, local initiatives that can be done as well.

If you are working in the concrete bridge industry and are not already engaged with your local university, reach out to faculty and administrators. The key here is to listen. In my experience, people from industry often wanted to tell university faculty what they were going to do, rather than asking us what we needed.

Work with them to establish a roster of engaged volunteers that will develop a relationship with the faculty and student groups. Ask if they are looking for presentations for meetings or seminars. Such outreach efforts may help you reach students early in their education before they have decided on a specific career path. See the Perspective on page 5 for outreach ideas.

If you are associated with a transportation agency or professional group that holds conferences, provide support for students to attend and then aggressively recruit them. Most faculty will excuse students from classes for professional development, and students may welcome the opportunity to attend.

As I observed at the University of Cincinnati, the birthplace of the cooperative education experience, co-ops and internships can be excellent recruiting opportunities. However, be aware that students may shy away from experiences in bridge engineering because it is so different from what they learn in school, and they may find the co-op or internship experience difficult

or confusing. Patience and strong mentorship are needed.

Finally, consider other ways to support bridge design education at a local university. Simple actions such as agreeing to be an adjunct or helping identify qualified adjuncts help. There are also examples of industrial groups providing funds for course development and paying adjuncts to teach the courses.

I have provided one perspective based on my experience. If you have a different perspective or insight that would be informative to the readers of *ASPIRE*, please contact us. We would love to hear different points of view.

In *Poor Richard's Almanac*, Benjamin Franklin wrote, "If you would have your business done, go; if not, send." This adage reminds us that personal involvement is crucial. If we want bridge engineers, the onus is on us to make them.

Reference

1. U.S. Census Bureau. 2026. "Monthly Construction Spending, March 2026." Published May 7, 2026. <https://www.census.gov/construction/c30/c30index.html>. 

Dr. Richard Miller is the managing technical editor of ASPIRE® and professor emeritus of the University of Cincinnati, where he taught for 36 years. In 2024 he was named a PCI Titan of the Industry.

2027 PCI Committee Days and Technical Conference



Call for Papers and Presentations



Abstracts of technical papers and presentations are now being accepted for the 2027 PCI Committee Days and Technical Conference held in Rosemont, IL.

The PCI Committee Days and Technical Conference is a premier national venue for the exchange of ideas and state-of-the-art information on precast and precast, prestressed concrete design, fabrication, and construction. The event provides an outstanding opportunity for networking, education, and sharing ideas.

Abstracts may be submitted on any topic relevant to precast concrete. There will be two tracks: Buildings/Materials and Infrastructure/Bridges. Don't miss this excellent opportunity to share your knowledge. Submit your abstract today!

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Submission deadline is November 20, 2026.

Papers will be due March 12, 2027.



September 21-25, 2027, in Rosemont, IL

For more information visit pci.org/callforpapers

Wishbone Bridge Connects Austin Through Precast Concrete Innovation

by Lee Frieberg, HDR

When the new Wishbone pedestrian bridge in Austin, Tex., officially opened on February 7, 2026, the automated counter on the structure indicated that more than 21,000 people showed up to mark the occasion. This impressive turnout reflected pent-up demand for a safe and inviting crossing on one of the most heavily used urban trail corridors in Texas.

A fitting addition to Austin's many pedestrian crossings, the bridge provides a long-missing link to complete the city's 10-mile loop trail system. It spans Lady Bird Lake at the eastern end of the Ann and Roy Butler Hike-and-Bike Trail, connecting Longhorn Shores on the south bank, Holly Shores to the north, and a narrow peninsula created decades ago for power plant cooling operations.

The Y-shaped structure features three gently curving legs that converge at a wide central plaza, creating a civic gathering space as well as a bridge. Although the form is unique, the structural system relies in large part on standard precast and prestressed concrete elements.

During design and construction, the project team addressed issues involving complex geometry, environmental constraints, constructability, and cost.

Ultimately, the team delivered a high-quality user experience.

Community Origins and Project Need

The distinctive wishbone shape of the bridge emerged from ideas offered during early public engagement. During the first round of community meetings,

several participants independently drew a Y-shaped bridge that would connect all three shorelines at once. That concept became the community's favorite, receiving roughly twice the support of the next most popular alternative.

By the third public meeting, when the refined wishbone concept was

The Y-shaped Wishbone Bridge in Austin, Tex., spans Lady Bird Lake at the eastern end of the Ann and Roy Butler Hike-and-Bike Trail. The pedestrian bridge connects Longhorn Shores on the south bank, Holly Shores to the north, and a narrow peninsula. The three legs converge at a central plaza. All Photos: HDR.



profile

WISHBONE PEDESTRIAN BRIDGE / AUSTIN, TEXAS

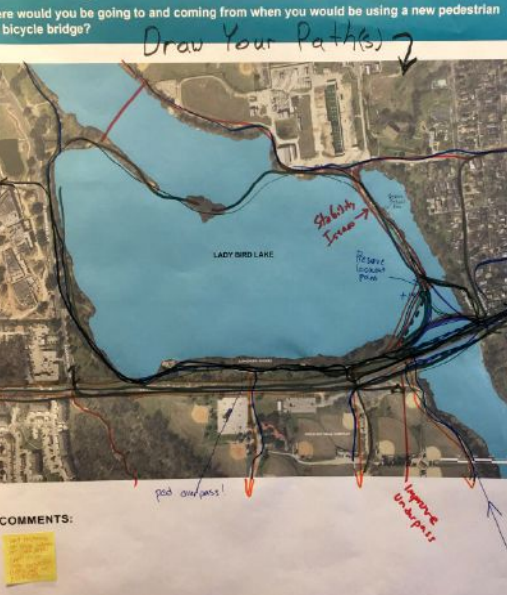
BRIDGE DESIGN ENGINEER: HDR Inc., Austin, Tex.

PRIME CONTRACTOR: Jay-Reese Contractors, Dripping Springs, Tex.

CONCRETE SUPPLIER: Alpha Ready Mix, Hutto, Tex.

PRECASTER: Heldenfels Enterprises, San Marcos, Tex.—a PCI-certified producer

OTHER MATERIAL SUPPLIERS: Formwork: EFCO, Dallas, Tex.; Reinforcement: CMC, Seguin, Tex.



In drawings of their preferred alignment for the new bridge, participants at a community meeting show a clear preference for a Y-shaped structure.

presented, 96% of participants reported being supportive or very supportive of the design. That level of consensus is rare for a major piece of urban infrastructure, and it helped carry the project forward through funding and final design.

Splayed precast concrete girders accommodate the irregular geometry of the unique bridge.



The crane used to erect the precast concrete girders for the Wishbone Bridge, which includes nine in-water piers, is mounted on barges to minimize the project's environmental impact. The plaza cap beams have unequal cantilevers. Due to geometrical constraints, the cap beams carry equal moments during the in-service dead-load condition.

The project also garnered community support because it addressed an evident public need. Before the bridge was built, trail users were funneled onto a narrow sidewalk along the Pleasant Valley Road Bridge and Longhorn Dam. In places, the path was running alongside fast-

moving traffic and maintenance gates and was just 3 to 4 ft wide. With more than 5 million annual users on the trail system, the crossing had become both a safety hazard and a bottleneck.

In 2021, while the Wishbone Bridge was being designed and constructed, an interim improvement was implemented on the Pleasant Valley Road Bridge. That improvement, which widened portions of the sidewalk, was never intended as a long-term solution. Wishbone Bridge provides a fully separated pedestrian and bicycle alternative to using the widened sidewalk on the Pleasant Valley Road Bridge. Together with the new Unity Underpass beneath Pleasant Valley Road, which is 30 ft wide and 16 ft tall, Wishbone Bridge creates a continuous, traffic-free route while improving accessibility and user comfort.

Defining the Scope and Structural Approach

Once the overall form of Wishbone Bridge was established, the next challenge was determining the structural and material specifications. Early in the process, the project team presented the public with a choice: invest in a signature

CITY OF AUSTIN, OWNER

BRIDGE DESCRIPTION: A Y-shaped, three-legged pedestrian bridge approximately 1121 ft in total length

STRUCTURAL COMPONENTS: Twenty-nine TX-62 I-girders; four 15-in.-deep slab beams; 48-in.-deep cast-in-place concrete diaphragms; 8½-in.-thick cast-in-place concrete deck; cast-in-place inverted tee pier caps, columns, and drilled shafts

BRIDGE CONSTRUCTION COST: \$325/ft²



During construction, the precast concrete girders serve as stable work platforms for cast-in-place operations.

structure with high visual impact, or use a conventional, efficient substructure and focus project resources on the elements that the public would use and experience most directly. The community favored the latter approach, prioritizing shade, seating, landscaping, generous widths, and gathering areas.

That decision set the direction for the structural solution. Below the 8½-in.-thick cast-in-place concrete deck, the bridge uses efficient, proven components. Above the deck, the project invested primarily in features that shape the user experience, resulting in a bridge that functions as both a crossing and a destination.

In plan, the bridge consists of three legs framing a central triangular opening. The

superstructure is supported by drilled shafts, which vary in diameter from 2 ft to 5 ft and are founded a minimum of 15 ft into competent rock, and round concrete columns, capped with inverted-tee cast-in-place pier caps and Texas Department of Transportation (TxDOT) standard TX-62 prestressed concrete I-girders. While the pier caps are typical of TxDOT construction, they were custom designed for this application. The pier caps near the plaza have an offset column that, due to the eccentric loading, balances the column dead loads under final dead-load configuration. A slanted earwall hides the “T” shape. The ends of the beams are slightly skewed at the caps near the plaza.

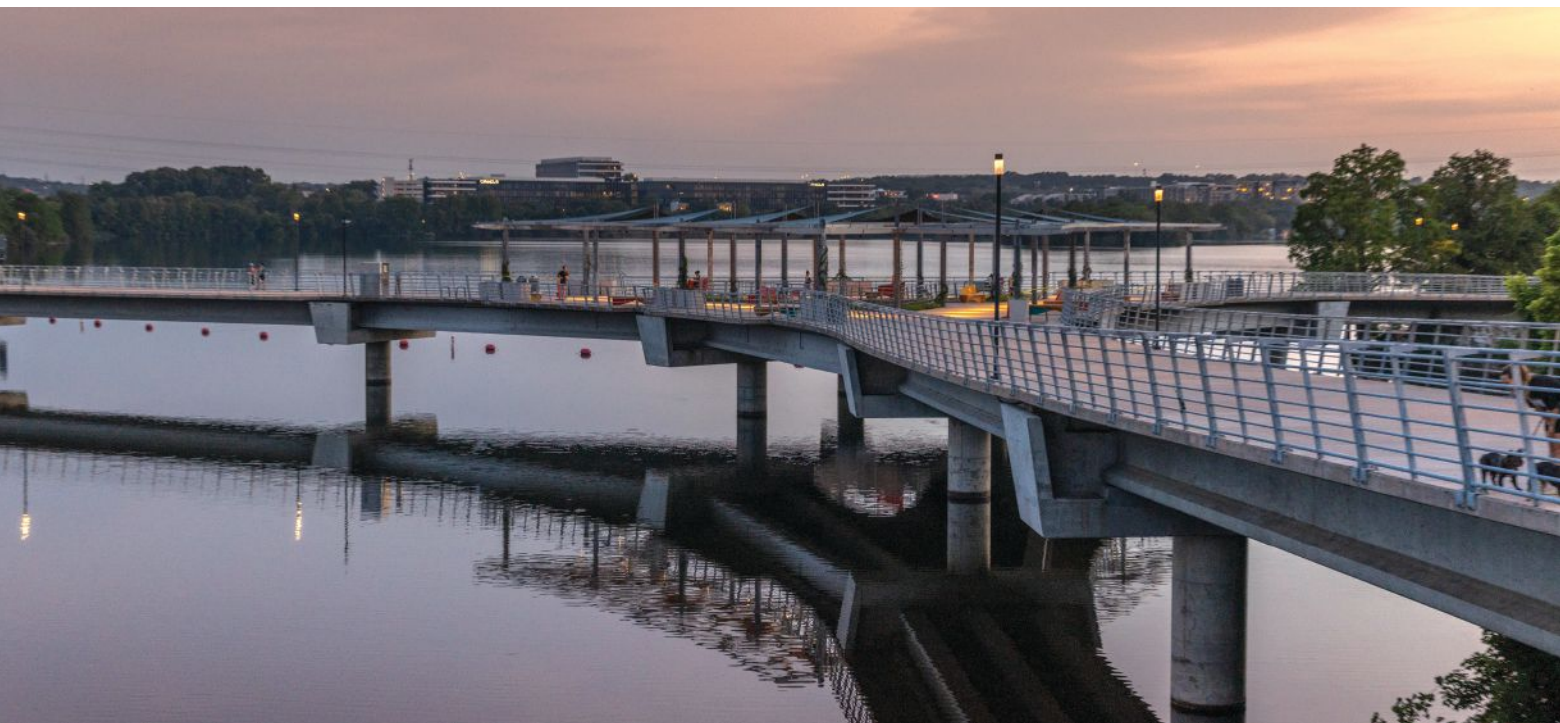
Spans generally range from about 86 to 112 ft; designers chose these lengths

to minimize the number of piers in the water and reduce hydraulic impacts. A constant-depth superstructure was desired, resulting in underuse of girders near the plaza. A range of twelve to thirty-two 0.6-in.-diameter strands was required for the various girders. In total, the bridge includes nine in-water piers.

The use of long-span precast concrete girders offers multiple design benefits. The limited number of piers minimizes the obstruction to flow in the tightly regulated, no-rise lake environment. The use of long spans also reduced environmental disturbance during construction and simplified permitting in the environmentally sensitive corridor. Inverted tee pier caps were selected to maximize vertical clearance beneath the bridge while reducing visual obstruction. 15-in.-deep slab beams located on the two peninsula spans create a shallow superstructure as the bridge profile descends to match the existing grade. These shallow girders were needed to avoid interference with the critical root zones of several protected trees in this area.

The deck width varies significantly, from roughly 16 ft on the approach legs to more than 70 ft at the central plaza; with even greater width measured across the void created by the triangular opening. To support those varying widths, the project team used a creative framing system, including splayed girders and substantial cast-in-place diaphragms

The new Wishbone pedestrian bridge has become a popular amenity for pedestrians and bike riders in downtown Austin, Tex.





The central plaza stands 30 ft above the water, offering shade and scenic views.

that tie the precast concrete elements together.

Working in a Managed Floodplain

One of the most significant constraints on the project was its location within a highly managed floodplain. Lady Bird Lake operates under a strict no-rise requirement. Any measurable increase in water surface elevation, even fractions of an inch, would have required additional approvals. Therefore, the design team worked to minimize the number of in-water supports and carefully modeled hydraulic effects. While early one-dimensional models suggested that the new bridge design might displace water and cause levels to rise upstream, a refined two-dimensional analysis demonstrated that any changes would be localized around pier locations and did not produce backwater effects. This analysis reinforced the decision to use long-span precast concrete girders to limit obstructions.

Structural Complexity of a Three-Legged Bridge

From an engineering perspective, the wishbone geometry introduced challenges well beyond those of a straight crossing. In a conventional bridge, thermal movement can be predicted along a single axis, with expansion and contraction occurring symmetrically. In a three-legged configuration, movement is constrained in multiple directions, creating complex stress paths that must be carefully managed.

To assess these stresses, the design team developed a global structural model and repeated the following steps until

the deck stresses were within specified tolerances: fixities were assigned based on the location of joints, a temperature deformation load was applied, and the resulting deck stresses were evaluated.

Splaying the prestressed concrete girders to follow the geometry required careful detailing. In some areas, girder spacing reached approximately 22 ft, which is far wider than typical practice for girder bridges. Those wide spacings would also need to support significant additional loads from below-deck planters that would measure roughly 4 ft x 4 ft x 18 ft and be filled with soil and vegetation.

The solution was a hybrid system. Precast concrete girders provide the primary load-carrying capacity and long-span capability, and cast-in-place diaphragms and beams distribute loads and accommodate the irregular geometry. This combination preserves the efficiency of precast concrete construction while providing the flexibility needed for architectural features. The precast concrete girders also serve as important building blocks to support the cast-in-place portions of this bridge erected in relatively deep water.

Accessibility issues and slope requirements across the bridge added layers of complexity to the design. One approach connects to Canterbury Street, on the north, which is roughly 15 ft above Longhorn Shores on the south. The central plaza must be nearly level to function as a gathering space. The roadway and bridge designers coordinated vertical profiles and cross slopes to meet accessibility requirements of the Americans with Disability Act (ADA) without creating uncomfortable

transitions or drainage issues. To find a preferred solution, the team went through multiple design iterations, including raising, lowering, or tilting the plaza by 1%, and evaluated the resulting structure for ADA compliance and floodplain impacts. The three-dimensional model was refined repeatedly until all criteria were met.

Constructability and the Value of Precast Concrete

One of the clearest demonstrations of the value of precast concrete came during construction. The site, which is surrounded by parkland, dense development, and water, provided limited options for equipment access. The nearby Longhorn Dam has load restrictions that prevent the use of heavy cranes, and portions of the peninsula are underlain by uncompacted fill unsuitable for large machinery.

Given these restrictions, the project team decided to use modular barges as temporary work platforms. These barges first held drilling equipment that pushed through the sediment and limestone at the lake bottom to create space for substructure columns. Columns and inverted tee caps were cast in place, with the work supported by barges. However, the environmental conditions would have made cast-in-place construction for long spans impractical. Therefore, precast, prestressed concrete girders were fabricated off site under controlled conditions and then delivered and erected using cranes mounted on barges. This approach reduced the need for extensive falsework and temporary supports in the lake. The prestressed concrete girders were fabricated at a nearby plant that was accustomed to producing TxDOT

standard sections, underscoring another advantage of precast concrete: availability. Using familiar, widely produced elements reduced risk and helped keep the project within budget, even as bids initially came in higher than anticipated.

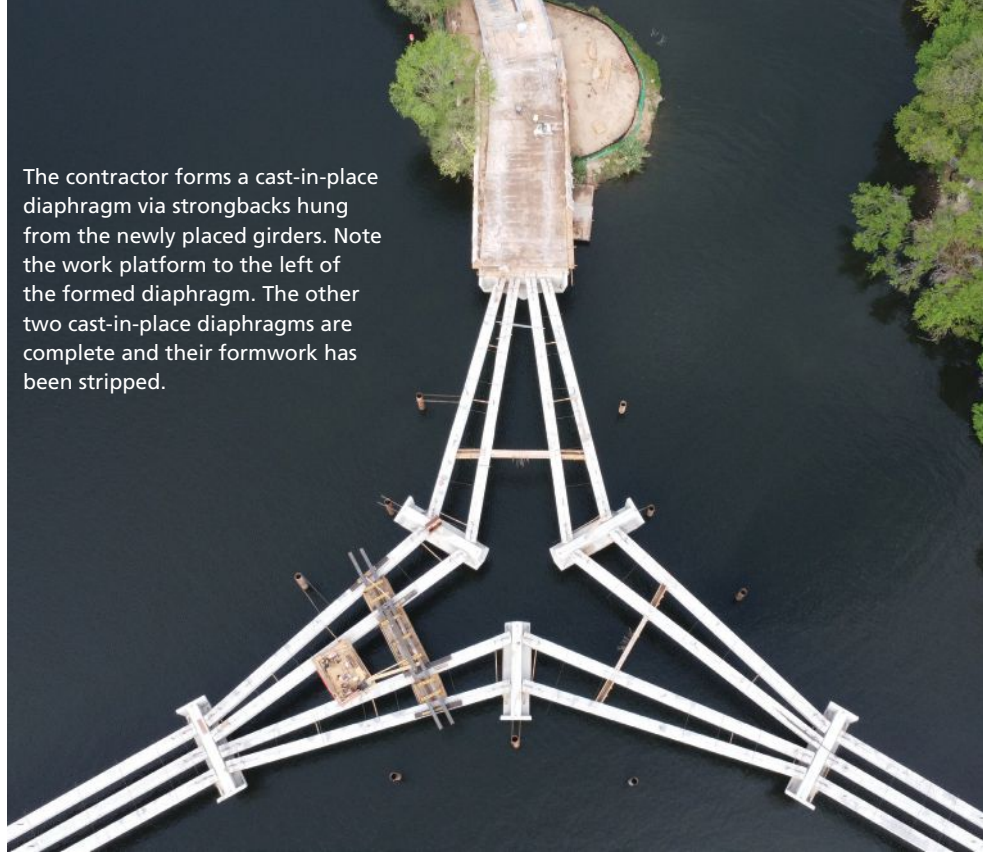
Once the girders were in place, they served as a base for stable work platforms for subsequent cast-in-place operations. Strongbacks were installed to support formwork for large diaphragms and planters, allowing crews to safely place significant volumes of concrete over open water.

Concrete placement itself required innovation. At times, concrete was pumped distances of more than 600 ft from land-based pump trucks. By keeping pump trucks on shore and routing hoses across the deck, the contractor avoided the need for additional barges and minimized disruption in the lake.

Conclusion

Wishbone Bridge demonstrates how standard precast and prestressed concrete elements can be adapted to support ambitious architectural and community-driven goals. By pairing practical, efficient structural components

The contractor forms a cast-in-place diaphragm via strongbacks hung from the newly placed girders. Note the work platform to the left of the formed diaphragm. The other two cast-in-place diaphragms are complete and their formwork has been stripped.



below deck with finishes and amenities that center on the user experience above, the project team delivered a bridge that is technically sophisticated and responsive to its setting.

In this project, standard girders, inverted tee caps and cast-in-place connections were assembled in new ways to

solve complex problems of geometry, hydraulics, and constructability. The result is a bridge that already functions as a landmark and a gathering place for the city. 

Lee Frieberg is a bridge engineer and bridge business class leader for HDR Inc. in Austin, Tex.



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Wallis Annenberg Wildlife Crossing, Agoura Hills, California

by Robert Rock, Rock Design Associates, and Paul Chung, California Department of Transportation

The Wallis Annenberg Wildlife Crossing is the largest wildlife crossing bridge structure in the world. It provides safe and sustainable passage for wildlife across California State Highway U.S. Route 101 (U.S. 101), reducing the potential for collisions and wildlife mortalities on one of the busiest freeways in the United States. Functioning as an ecological stitch that connects fragmented landscapes, the crossing enables natural movement for California mountain lions and other animals that call the Santa Monica Mountains home, thereby playing a crucial role in maintaining regional biodiversity.

The project was constructed by the California Department of Transportation (Caltrans) in partnership with the Mountain Recreation and Conservation Authority (MRCA), the Santa Monica Mountains National Recreation Area, Rock Design Associates LLC, and the National Wildlife Federation. It crosses U.S. 101 west of Liberty Canyon Road in the city of Agoura Hills in Los Angeles County and connects lands owned and managed by MRCA.

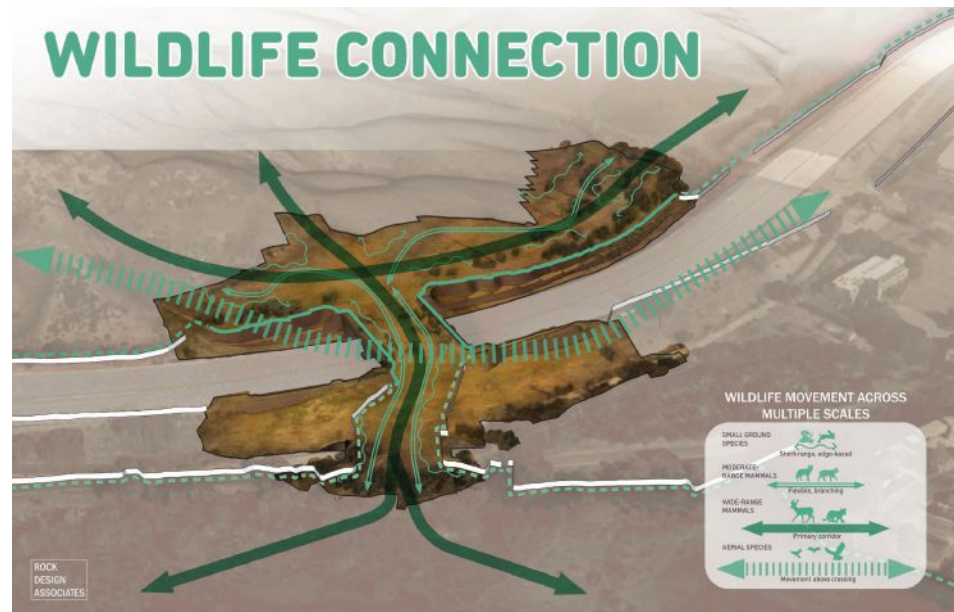
The primary wildlife crossing structure over U.S. 101 is 250 ft long and 174 ft

wide. This two-span vegetated structure crosses 10 lanes of the freeway, carrying more than 6000 yd³ of soil and stone that support the living layer on top. It features a landscape planted exclusively with native plants, offering animals a secure and familiar environment as they move through this open space.

Accelerated Bridge Construction

U.S. 101 is a heavily traveled freeway connecting the California Central Coast to Los Angeles and the Southern California region. Throughout the design and construction process for the wildlife crossing, the project delivery team was

The projected movements of various types of wildlife across the Wallis Annenberg Wildlife Crossing. Figure: Rock Design Associates.



profile

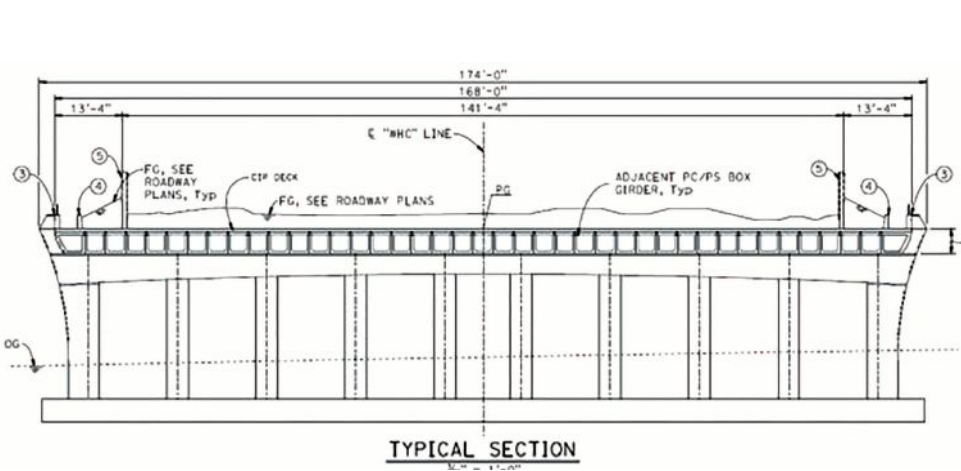
WALLIS ANNENBERG WILDLIFE CROSSING / AGOURA HILLS, CALIFORNIA

BRIDGE DESIGN ENGINEER: California Department of Transportation, Diamond Bar, Calif.

PROJECT DESIGNERS: Lead designer: Rock Design Associates LLC, Chicago, Ill.; project engineer: O'Donna Blackstock, PE, Los Angeles, Calif.

PRIME CONTRACTOR: C.A. Rasmussen, Valencia, Calif.

PRECASTER: Coreslab Structures, Perris, Calif. —a PCI-certified producer

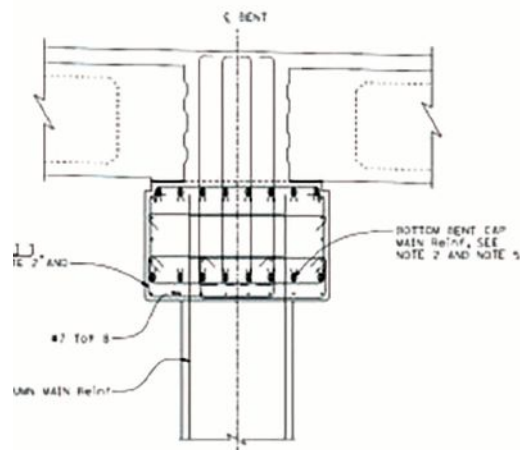


Typical section of the bridge. Note the curved outer surface of the exterior box girders. Figure: California Department of Transportation.

keenly aware of the need to minimize inconvenience for the traveling public. They sought a design that would incorporate bridge construction details and methods that would yield minimal disruption to traffic in both directions.

To accomplish that goal, the design team selected an accelerated bridge construction strategy and specified precast concrete box girders in the design of the superstructure to enable a focused and streamlined on-site construction effort. To further reduce

traffic impacts, the project team specified one-way closures during overnight hours, when traffic volumes were at their lowest. Because precast concrete box girders and a cast-in-place concrete topping slab were used to form the deck, workers could place the 4-ft 10-in.-deep superstructure directly atop the abutments while maintaining a minimum vertical clearance of 16 ft 10 in. Using the precast concrete box girders eliminated the need for the extensive falsework and protection systems typically required with an all-cast-in-place approach.



Reinforcement details of the cast-in-place concrete bent cap. Figure: California Department of Transportation.

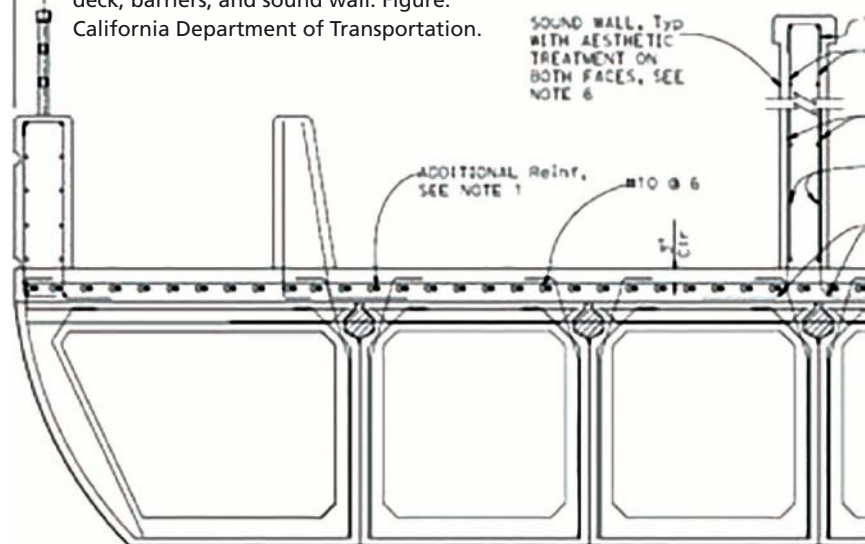
Bridge Design

The superstructure has eighty-two 4-ft 3-in.-deep, 103-ft-long precast, prestressed concrete box girders positioned side by side and topped with a 7-in.-thick cast-in-place concrete deck. Adjacent girders are connected by keyways with projecting bars; no post-tensioning is used. Each box girder is designed with an initial concrete compressive strength of 5500 psi and a final concrete compressive strength of 7000 psi.

Supported by diaphragm abutments and a multicolumn bent in the freeway median, the superstructure girders are integrally connected in the transverse direction by closure placements in a key filled with high-strength, nonshrink concrete grout and the cast-in-place concrete deck and diaphragm. The girders are made continuous in the longitudinal direction through connection by cast-in-place diaphragms at the bent cap and the cast-in-place deck.

The cast-in-place reinforced concrete bent is composed of a transverse drop cap that provides a seat to support the precast concrete girders, with 4-ft 3-in.-square reinforced concrete columns anchored on spread footings. The diaphragm abutments located at

Partial bridge cross section showing some of the reinforcement for the cast-in-place deck, barriers, and sound wall. Figure: California Department of Transportation.



CALIFORNIA DEPARTMENT OF TRANSPORTATION, OWNER

BRIDGE DESCRIPTION: Two-span, 250-ft-long, 174-ft-wide precast, prestressed concrete box-girder wildlife crossing with cast-in-place concrete bent, columns, and abutments

STRUCTURAL COMPONENTS: Eighty-two 4-ft 3-in.-deep, 103-ft-long precast, prestressed box girders; 7-in. cast-in-place concrete deck; cast-in-place reinforced concrete bent and diaphragm abutments; 4-ft 3-in.-square cast-in-place concrete columns

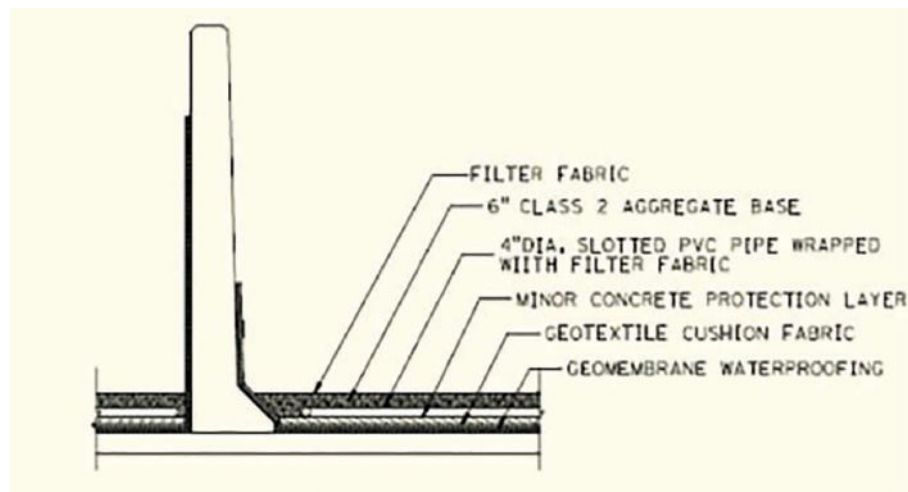
BRIDGE CONSTRUCTION COST: Approximately \$114 million

AWARDS: 50 Most Influential Projects of 2022 by the Project Management Institute; San Fernando Valley Audubon Society 2026 Conservation Award

both ends of the bridge are designed to resist lateral forces and enhance the bridge's stability. Lateral forces include earth pressure from the soil behind the abutment and seismic lateral forces. These abutments are supported on spread footings, the construction of which was complicated by intense rainfall events early in the project.

Design Loads

The design of the superstructure, substructure, and foundation accounts for the dead load of the deck protection materials, vegetation, and soil. Whereas the designs of freeway bridge structures are typically governed by the live load associated with highway traffic, the controlling load in this case is the dead load of the vegetative layer on top of the structure. The unfactored dead load from the engineered soil included a maximum of 2 ft 11 in. of expanded shale at 97 lb/ft³, a maximum of 2 ft of horticultural saturated soil at 120 lb/ft³, and a maximum of 10 in. of Class 1 permeable material at 120 lb/ft³. The design live load uses an HL-93 truck load to account for incidental truck movement for maintenance and emergency activities after the bridge is completed. Seismic design for the structure uses a mean earthquake magnitude of 6.71 and a



The bridge deck and the roadway below are protected by multiple layers, which are also designed to permit drainage. Figure: California Department of Transportation.

design peak ground acceleration of 0.39g. No nonstandard features are required to resist the seismic loads because the design peak ground acceleration is not high.

Deck Layout and Landscape Buffers

The width of the bridge deck accommodates three distinct functional areas. The central deck area serves as the natural habitat for wildlife crossing over the freeway, and each side is bordered by an 11-ft-tall, exterior cast-

in-place concrete sound wall in addition to a robustly vegetated buffer landscape. The sound walls and vegetated buffer landscapes serve as a protective barrier for wildlife traversing the structure by reducing the impact of vehicle noise and artificial lights from the traffic below. The central deck area is overlaid with between 1 and 4 ft of engineered soil to create artificial topography, provide drainage, and support native vegetation. This area contributes to habitat restoration and features indigenous coastal sage scrub plants. The vegetated



AESTHETICS COMMENTARY

by Frederick Gottemoeller

When designing a wildlife crossing, designers typically must hide the artificiality of the crossing from its wildlife users. The Wallis Annenberg Wildlife Crossing posed this design challenge to an extreme degree since the wildlife corridor that crosses U.S. Route 101 and Agoura Road is an entire watershed with flowing and intermittent creeks. This crossing project specifically required complex and innovative design solutions to address two separate areas of concern: First, the water and drainage under the watershed topography that forms the wildlife corridor itself have to be controlled. Second, the crossing—which is carried over the highway and secondary road on two separate structures with a total 125 ft of vertical grade separation from end to end of the watershed—requires sound and visual barriers to separate the natural and highway environments.

The construction that controls water and drainage is all buried and is not a topic for aesthetic discussion. However, the barrier design is aesthetically important. From the wildlife side, the structural edges of the crossing structures themselves need to look just like the rest of the watershed topography. From the highway side, the edges need to look attractive to vehicle occupants and must evoke some recognition of the function of the whole project. This aspect of the crossing design affects road users' reaction to the project itself and will influence public support for future projects of this type.

First, let's consider what the wildlife sees. The central deck area serves as the natural habitat for animals crossing over the freeway, and each side is bordered by robustly vegetated buffer

landscape ending at a concrete sound wall. So, the wildlife should be happy.

Second, let's consider what highway users see. They see the other side of the concrete sound wall, which is fronted by 10 ft of sloped earth on which vegetation is grown to soften the appearance of the wall. Then they see the rounded edge of the first box girder, a concrete parapet wall, and fence support that hides a narrow maintenance walkway. Finally, they see that the soffit of the multicolumn median bent has a slight upward arch, further evoking the mountain landscape. In a quick glance, they take in all rounded surfaces and vegetation recalling the natural environment being carried overhead. That should impress the highway users, reassuring them that their highway dollars are being put to good use.

It seems to me that both the wildlife and the highway users are going to like what they see, and both animals and humans will feel that the Wallis Annenberg Wildlife Crossing is a natural component of their two very different environments.



The first precast concrete girder is placed. Shoring is used for girder support until the bent caps and girders are connected through the cast-in-place concrete keys and placements. Photo: California Department of Transportation.



Construction on the precast concrete girders continues. Photo: California Department of Transportation.



Construction of the cast-in-place concrete sound wall. Photo: California Department of Transportation.



Between 1 and 4 ft of engineered soil creates artificial topography, provides drainage, and supports native vegetation. Photo: California Department of Transportation.

buffer landscapes outside the concrete sound walls also allow for secure access by personnel, facilitating routine inspections and future maintenance of the bridge.

Deck Protection and Drainage System

The bridge deck is safeguarded with multiple protective layers that serve distinct functions in maintaining the deck's integrity and supporting green infrastructure. Starting from the bottom of the profile and working up, a preformed layer of geomembrane waterproofing creates a seamless barrier across the deck surface, preventing water from penetrating into the structural components. This protection is essential to preserve the bridge deck and minimize the potential impact of water on the freeway below. The waterproofing membrane is topped with a layer of geotextile cushion and filter fabric, which is covered by 3 in. of general purpose concrete and 6 in. of Class 2 aggregate base to form the base filtration layers. These layers of fabric and aggregate are designed to permit water to flow through while reducing the potential for soil particles to reach the drainage

system so that drainage atop the bridge deck is consistent.

A perforated polyvinyl chloride pipe drainage system is also incorporated into this profile to efficiently channel water away from the deck, working in tandem with the filter fabric to preserve the deck's structure and functionality. In the first stage of the project, the perforated pipes were tied into a manifold on the north side of the freeway and routed to the storm sewer. As the project's second stage is completed, this connection will be rerouted so that the manifold is disconnected from the storm sewer and allowed to drain into the adjacent landscape. This connection will feed a new ephemeral creek corridor connected to Liberty Canyon Creek, which will facilitate beneficial hydrology and support wildlife movement.

Soil Profile and Hydrology

The design team developed the soil profile to mimic the natural profile on either side of the crossing structure. The engineered soil profile—which is intended to provide essential nutrients while also ensuring proper drainage to enable healthy vegetation growth

on the bridge deck—consists of a series of soil layers that facilitate drainage, host beneficial fungi and other soil microorganisms, and support the vegetated surface. The modest topography atop the bridge is designed to channel positive surface flow during storm events into the new ephemeral creek corridor, tying into the restored hydrology north of the freeway. By supporting plant life, the growing media contributes to the long-term sustainability of the bridge's green infrastructure, enhancing both its environmental benefits and aesthetic appeal.

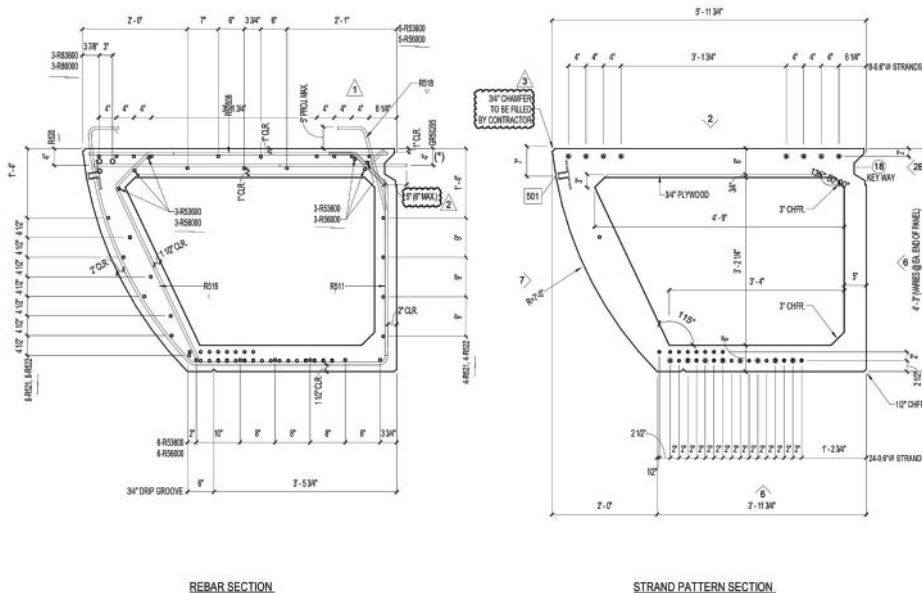
Aesthetic Design

A project of this scale requires a combination of engineering, science, and artistry. The design team took care to ensure that the design aesthetic was both striking and humble, with subtle design features that allow the wildlife crossing to be immediately recognizable while also blending into the landscape.

The exterior faces of the precast concrete box girders have gently curved surfaces, which were a challenge during



The central deck area serves as natural habitat for wildlife crossing, and each side is bordered by an 11-ft-tall sound wall and vegetated buffer landscape. Photo: Rock Design Associates and National Wildlife Federation.



Reinforcement details of the curved exterior box girders. Figure: Coreslab Structures.

production. The precaster had to ensure the formwork was adjusted precisely to ensure a uniform finish on the exposed surface. Colored precast concrete was used for this project, which created unique concrete mixture challenges. See the sidebar, "The Importance of Color," for information on the challenges related to color.

The tall vertical faces of the sound walls are pushed back from the edge and fronted by vegetation to soften the structure and let the landscape carry across the structure to visually connect the mountain habitat on either side from the vantage point of drivers on the freeway. The columns at the central bent also have subtle curves, and the arch in the bent cap subtly shifts the viewer's perspective, softening what is otherwise a tectonic mass carrying the weight of the mountain over the freeway.

Construction Timeline

Construction began following a groundbreaking ceremony on Earth Day, April 22, 2022, and the project is being constructed in two stages. In stage 1, which was completed in June 2025, the scope of work included the Annenberg Wildlife Crossing over U.S. 101 and supporting roadway items within the Caltrans right-of-way.

A large, 500-ton crane was used for erection, which had to be partially disassembled every night to keep the freeway lanes open. This daily

A curved box girder is delivered to the site. Photo: Coreslab Structures.





Precast concrete girders in the precast concrete manufacturer's yard. Photo: Coreslab Structures.

reassembly and disassembly limited the available installation time.

The scope of work for stage 2, which is currently underway, includes the vegetated wildlife crossing structure over Agoura Road, relocation of utility lines, earthwork, and restoration of wildlife habitat adjacent to U.S. 101 and Agoura Road. The earthwork operations for the 13-acre site require

the movement of more than 3 million ft³ of earthwork to restore a site with approximately 125 ft of grade change from north to south. 

Robert Rock is president and CEO at Rock Design Associates LLC in Chicago, Ill. Paul Chung is assistant chief of the Division of Program/Project Management for District 7 at the California Department of Transportation in Los Angeles.

The Importance of Color

An engineered wildlife crossing must blend into the existing landscape. Therefore, the design team for this project asked the precast concrete bridge producer to achieve a specific brown color for all the elements of the bridge structure. Typical mixture proportions specifying color have 5 to 10 lb of colorant per cubic yard of concrete. This small amount of colorant does not significantly affect the fresh concrete properties, and adjustments are not typically made to the anticipated concrete properties for quality control purposes.

For this project, to achieve the required brown color, the amount of colorant added was 54.45 lb/yd³, which is slightly over 2 lb/ft³. In the final mixture, the additional colorant changed the unit weight of the concrete by 1.5 lb/ft³, which is close to the allowable variation of 2 lb/ft³ stated in PCI quality control documents.

Quality control personnel noticed this variation. The precast concrete fabricator adjusted the unit weight calculation to include the colorant, which then gave the correct unit weight for the concrete. None of the other concrete properties were affected.

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Grass Lake Wildlife Overcrossing: Restoring Habitat Connectivity Through Innovative Bridge Engineering

by Habib Hotaki and Kerry Molz, California Department of Transportation

Collisions between vehicles and wildlife represent a persistent safety and ecological challenge across Western U.S. transportation corridors. Along U.S. Route 97 in Siskiyou County, Calif., elk movement routes intersect a major highway corridor, creating both a public safety hazard and a long-term barrier to ecological connectivity.

Between 2015 and 2020, vehicles killed more than 50 deer and 16 elk along this specific two-lane stretch near Grass Lake,¹ a collision rate typical of this region. U.S. 97 bisects seasonal movement paths for elk and mule deer across open grasslands, oak woodlands, and higher-elevation habitats.

To improve safety for both wildlife and the traveling public, the California Department of Transportation (Caltrans) devised an overcrossing that spans U.S. 97 about 26 miles from the Oregon border. Although several animal species benefit from the overcrossing, the land bridge was sized for elk because they are large animals and collisions with them cause major safety and financial issues for motorists. Mule deer, pronghorns, and elk are the primary users of the overcrossing, while black

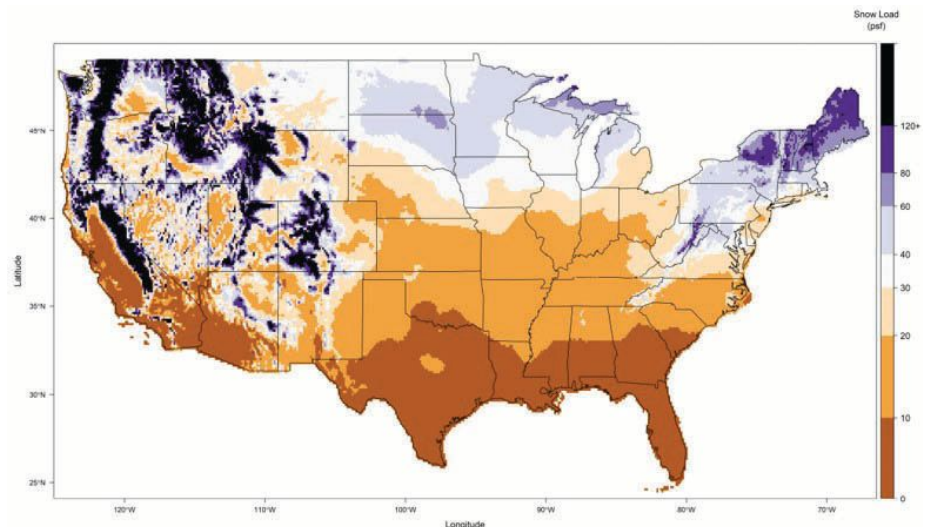
bear, wolf, bobcat, and other animals will also use the land bridge.

To further accommodate species-specific crossing behaviors, Caltrans has also built a precast concrete box culvert beneath the highway, 1.2 miles south of the overcrossing. The culvert is large enough for all local species, but it is meant for deer and smaller animals. Although the culvert complements the

wide land bridge, this article focuses only on the overcrossing.

The Grass Lake Wildlife Overcrossing was developed to restore habitat connectivity and improve roadway safety. The project has also advanced bridge engineering practice through a novel approach to snow-load design, an innovative prefabrication strategy, and the first use of glass-fiber-reinforced

A map showing proposed design ground snow-load requirements for the United States based on the results of the 2020 National Snow Load Study.⁴



profile

GRASS LAKE WILDLIFE OVERCROSSING /SISKIYOU COUNTY, CALIFORNIA

BRIDGE DESIGN ENGINEER: California Department of Transportation (Caltrans) Division of Engineering Services, Sacramento, Calif.

OTHER CONSULTANT: Caltrans District 2, Redding, Calif.

PRIME CONTRACTOR: McCuen Construction Inc., Loomis, Calif.

CONCRETE SUPPLIER: Sousa Ready Mix LLC, Mount Shasta, Calif.

polymer (GFRP) reinforcement in California state highway infrastructure.

Site Selection

In November 2019, Caltrans and subject matter experts from Montana State University, Western Transportation Institute, California Department of Fish and Wildlife, U.S. Forest Service, and California Deer Association conducted multidisciplinary field studies to evaluate potential wildlife crossing locations. The objective was to align structural feasibility with documented ecological corridors. Caltrans, in collaboration with the team of regional conservation partners, considered connectivity, engineering and construction challenges, and site constraints and selected a location east of Grass Lake, approximately 1.4 miles north of Deer Mountain Road.

The selected location corresponds to documented elk movement corridors described in the California Department of Fish and Wildlife's *Elk Conservation and Management Plan*.² U.S. 97 bisects these routes, along with those for deer, black bear, mountain lions, and wolves, creating a functional barrier. Without a grade-separated crossing, elk either attempt at-grade highway crossings, which may lead to vehicular collisions, or avoid traditional migration paths, contributing to habitat fragmentation and population loss.

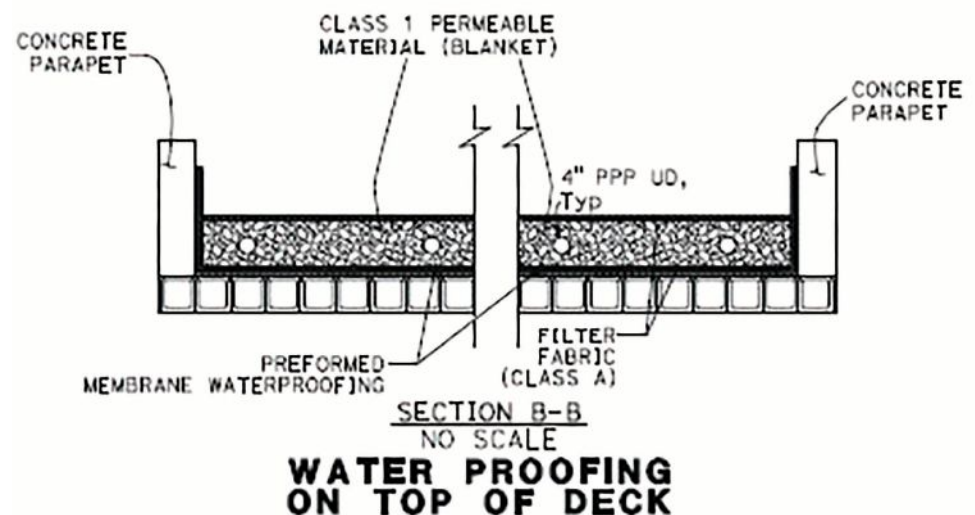
Site and Constructability Considerations

The remote location of the project significantly influenced decisions about the structural configuration. Because site access was limited and the delivery route for ready-mixed concrete was long, the design team chose to minimize cast-in-place concrete operations. Instead, the design team chose to

use 35 adjacent precast, prestressed concrete box girders to erect the 100-ft-long, 140-ft-wide superstructure. The box girders each contain thirty-five 0.6-in.-diameter strands and were designed with an initial concrete compressive strength of 7000 psi and a final concrete compressive strength of 8000 psi.

The box girders are topped with 8 in. of gravel for drainage and a maximum of 2.5 ft of noncompacted soil. Because there was no composite topping on the girders, cast-in-place concrete end diaphragms along the abutment seats were used to connect all girders together and to make the whole superstructure move together in case of any lateral movement. These diaphragm elements used GFRP bars tied together with mild steel bars, which extended from the box girders. No transverse post-tensioning was used.

A cross section of the structure shows the waterproofing membrane, drainage layer, and permeable blanket on top of the deck before the installation of 24 in. of lightweight, uncompacted soil and native vegetation. Figure: California Department of Transportation.



The precast concrete option shortened the construction duration, improved quality control, and limited environmental disturbance at the site and in the surrounding habitat. Projects that use cast-in-place concrete need falsework and temporary support for fresh concrete, and the erection schedule must include time to cure the concrete. By using precast concrete components, the project did not need falsework, eliminated on-site cure time, avoided long-term lane closures, reduced traffic disruptions, and improved safety for both motorists and construction workers.

Although the location of the site posed construction challenges, the geotechnical conditions at the site were advantageous. The seismic demand is relatively low (peak ground acceleration = 0.17g), and the foundation soils are competent. As a result, deep pile foundations were

CALIFORNIA DEPARTMENT OF TRANSPORTATION, OWNER

PRECASTERS: Girders: Con-Fab California LLC, Lathrop, Calif. —a PCI-certified producer; box culvert: Precast Unlimited Inc., Redding, Calif. —a PCI-certified producer

OTHER MATERIAL SUPPLIERS: Glass-fiber-reinforced polymer reinforcement: Camblin Steel Services, Roseville, Calif.

BRIDGE DESCRIPTION: 100-ft-long, 140-ft-wide single-span precast, prestressed concrete box-girder bridge on cast-in-place concrete abutments with spread footings

STRUCTURAL COMPONENTS: Thirty-five 4-ft wide by 4.5-ft tall precast, prestressed concrete box girders; cast-in-place concrete abutments

BRIDGE CONSTRUCTION COST: \$14 million (\$7 million for bridge items only)



The Grass Lake Wildlife Overcrossing is composed of precast, prestressed concrete box girders. The girders, which each contain thirty-five 0.6-in.-diameter strands, were designed to achieve an initial concrete compressive strength of 7000 psi and a final concrete compressive strength of 8000 psi. Photo: California Department of Transportation.

not necessary and shallow foundation systems could be used.

Unconventional Loading Conditions

Snow Loads

The design loads for conventional bridges in the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*³ are based on the assumption that snow loads are either negligible or offset by reduced traffic activity. (See Article C3.9 in the AASHTO LRFD specifications.)

Those assumptions are not valid for the Grass Lake Wildlife Overcrossing, which will not be plowed, will experience sustained snow accumulation, and will lack traffic-induced thermal or mechanical clearing effects. Also, unlike conventional highway bridges, the wildlife crossing will not be subject to fatigue from traffic loading, chemical deicing exposure, or deck wear. Consequently, the crossing will have significantly different deterioration mechanisms than those of a conventional bridge, and its expected design life is longer. Given these circumstances and the lack of wildlife crossing-specific design standards, the design team developed a probability-based snow-load philosophy to determine the loadings for this project.

At the Grass Lake site, the elevation is above 5100 ft, and snow accumulation is both routine and potentially severe. Seasonal snow accumulation in the region is typically moderate, but 4 to 5

ft of wet snow can fall during extreme storm events. For example, at Mount Shasta, which is just 16 miles away from the crossing, storms have delivered more than 15 ft of snow. Published estimates for ground snow loads at or near the site of the crossing vary widely, from 40 to 300 lb/ft².⁴ Project-specific snow load criteria were developed by the load committee of the Caltrans division of engineering services (DES).

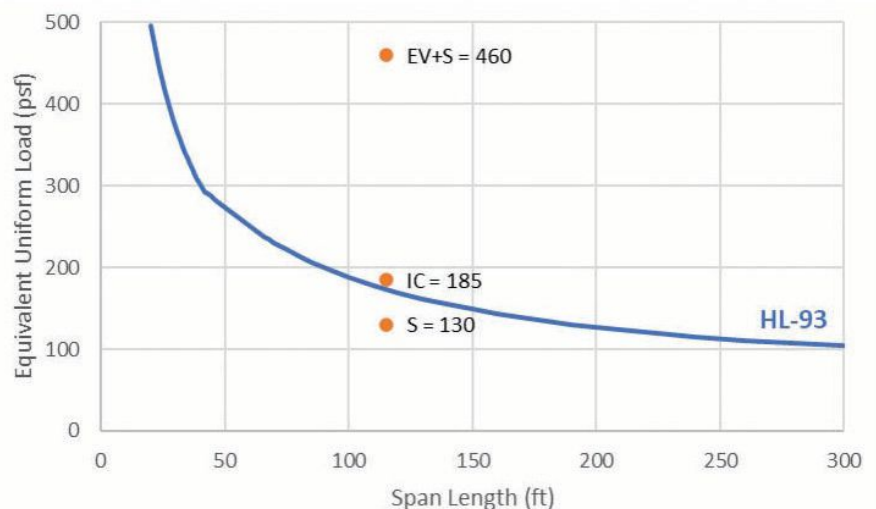
To determine load effects for the crossing, the design team compared equivalent midspan moments from the AASHTO HL-93 design vehicular loading with a uniform snow-load accumulation and determined that for a 100-ft span, the moment from snow loading would approach that of the vehicular live load. The AASHTO HL-93 load was only used to compare the traditional live load with snow load. H20-type design vehicles were used to design for the vehicular

load for this bridge and account for maintenance vehicles.

When the snow load S and the load from 3 ft of earth fill EV (330 lb/ft²) are combined, the total gravity load exceeds the HL-93 load pattern. The 3-ft of earth fill included 2 to 2.5 ft of noncompacted soil layer and the 8-in.-thick permeable (gravel) drainage layer. Thus, the governing load condition is $EV + S > HL-93$. This governing load condition is atypical in bridge engineering.

The design team adopted a dual-level snow-load framework for the overcrossing design in accordance with the project-specific snow load criteria developed by the DES loads committee. A 50-year snow event with a snow load of 125 lb/ft² was used for the Service Limit State/Strength Limit State, and a 500-year snow event with a snow load of 160 lb/ft² was used for the Extreme

Snow loads versus AASHTO HL-93 vehicular loads. Note: EV = vertical earth fill load; IC = ice load; S = snow load. Figure: Habib Hotaki.





Finished in 2025, the Grass Lake Wildlife Overcrossing provides safe passage for many animals across U.S. Route 97 in northern California. Photo: California Department of Transportation.

Event Limit State. The 50-year value aligns with the historical calibration of snow loads in the American Society of Civil Engineers' *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7).⁵ The 500-year event reflects the extended service-life expectations (100 to 150 years) for wildlife crossings, and guidance from the AASHTO LRFD specifications for extreme hazard evaluation beyond service life.

Wind Effects from Solid Fencing

To help guide animals, many wildlife crossings have see-through mesh or woven-wire barriers that provide ample visibility. The openness allows wind to pass through, prevents snow buildup, and creates a safe environment for animals. However, for the Grass Lake project, the design team chose an 8-ft solid steel wildlife fence to block oncoming vehicle headlights, muffle traffic noise, and help funnel the animals through the crossing. This solid wall introduced increased wind-load surface area, parapet overturning moments, and demand on diaphragm and edge beam systems. As a result, the parapet systems required enhanced stiffness and reinforcement to resist lateral wind-induced effects.

Approximately 6 miles of wildlife-exclusion fencing were installed to guide animals toward the overcrossing. The northern end of the fence line connects to Grass Lake, which serves as a natural barrier.

Creating an Inviting Overcrossing

Another aspect of the project that involved unconventional engineering was the bridge deck, which needed to be durable, be landscaped to resemble the site's natural environment, and invite wildlife use. To protect the top flanges of the precast, prestressed concrete box girders, workers installed a waterproof membrane and an 8-in.-thick gravel drainage layer over the girders. Crews then planted native low-growth and shallow-rooted shrubs in 24 in. of lightweight, uncompacted soil placed over the drainage layer. Heavier landscape items such as logs and boulders were placed at the deck approaches along with larger plants that guide the wildlife onto the bridge.

Innovative Prefabrication Strategy

For the Grass Lake project, GFRP bars were embedded in the precast concrete girders, which also had conventional steel reinforcing bars. The use of two

different types of reinforcement can lead to construction or durability issues. Therefore, extensive collaboration and coordination between the GFRP bar manufacturer and the precaster was required to accomplish the prefabrication work.

One constructability issue that was solved through collaboration involved embedding the GFRP bars together with steel reinforcing bars inside the precast concrete girders. Carbon steel should not come into contact with GFRP because contact causes a galvanic reaction. Although the steel bars had epoxy coating to prevent galvanization when in contact with GFRP bars, the small, uncoated metal bar ties had to be replaced with plastic zip ties. The bar chairs were also plastic rather than steel.

Glass-Fiber-Reinforced Polymer Reinforcement

GFRP reinforcement is a noncorrosive and lightweight alternative to traditional mild steel reinforcement. It has been used in bridge projects in coastal and snowbelt regions to prevent corrosion from marine environments or deicing chemicals. However, the Grass Lake Wildlife Overcrossing represents the first application of GFRP on a highway project in California.

On this project, GFRP reinforcement in transverse end diaphragms connects the precast concrete girder system and parapet elements along the exterior girders. The design team chose GFRP for its corrosion resistance, high strength-to-weight ratio, electromagnetic neutrality, and reduced maintenance. This application demonstrates increasing confidence by Caltrans in alternative solutions for structural longevity in challenging environments.

Conclusion

By integrating wildlife movement data, nontraditional load modeling, and advanced materials such as GFRP, the Grass Lake Wildlife Overcrossing project sets a precedent for future wildlife-crossing infrastructure in complex mountain environments. Motion-activated cameras show that animals such as mule deer began using the crossing before construction was complete. The structure is also expected to benefit other species, including mountain lions, black bears, and gray wolves.

These types of projects are critical to making roadways safer for both wildlife and motorists by reducing collisions. Many other wildlife crossings are being constructed or have been recently completed in California and elsewhere. For example, the Wallis Annenberg Wildlife Crossing in Agoura Hills, Calif., which spans U.S. Route 101 in Los Angeles County, is expected to open in December. (See the Project article on page 16 in this issue for more information on that project.)

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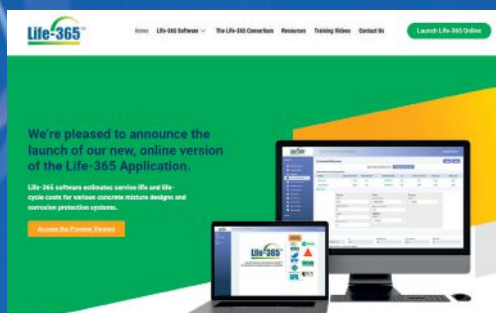
Habib Hotaki is a senior bridge engineer for the California Department of Transportation (Caltrans) Division of Engineering Services in Sacramento. Kerry Molz is the office chief for program/project management at Caltrans District 2 in Redding.

Life-365 Online Version (WebApp) Released



In 1999 a consortium was formed within American Concrete Institute's (ACI) Strategic Development Council (SDC) to fund development of a consensus model to estimate the service life and life-cycle costs of concrete mix designs. Shortly afterwards, the first version of Life-365 was released. During the last 20-years the software has gone through continuous updates, added features and a major User Interface redesign.

In November 2024, an online (WebApp) version of Life-365 was released. This version allows users to run this service life and life-cycle cost model in a web browser on a desk-top, laptop, tablet, or mobile phone. All features, functions, and ability to print reports remain fully intact.



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Innovative Vertically Segmented Precast Concrete I-Girders Span the Tigris River

by Hany Nasr, Ahmed A. Rawdan, and Dr. Maher K. Tadros, e.construct

Located in one of the most prominent urban districts in Baghdad, Iraq, Al-Masafi River Crossing Bridge forms a key component of a 2.24 mile (3.60 km) transportation corridor spanning the historic Tigris River. Because this bridge is positioned within a highly visible and architecturally sensitive setting adjacent to one of the city's landmark suspension bridges, it was important that the project present a structural solution that would deliver efficiency and durability without competing visually with its iconic neighbor. The new bridge is a three-span structure featuring a 360 ft (110 m) main span and two 262 ft (80 m) back spans.

Meeting Multiple Design Objectives

From the early stages of design development, the project team pursued a solution that would reconcile three primary objectives: structural efficiency, construction within site constraints, and architectural appeal. Multiple structural systems were evaluated and then narrowed to two options: a conventional cast-in-place, balanced-cantilever segmental box-girder system, and a precast concrete spliced I-girder system. While the box-girder solution offered structural efficiency and torsional rigidity, it was ultimately deemed less favorable due to its high construction complexity and the need for specialized formwork, heavy erection equipment, and an extended construction timeline.

In contrast, the precast concrete solution offered improved constructability and speed of erection, which were major advantages in the congested urban environment with limited staging areas. However, this approach had its own challenges. It was difficult to design a precast concrete system capable of achieving the required span lengths

while maintaining a slender structural profile. Additionally, when designing the components, the design team had to consider the weight limits and vertical clearances of the bridges on the hauling route between the precast concrete facility and the project site.

Additional constraints further shaped the design direction. The bridge had to maintain adequate navigational clearance over the Tigris River. The combined requirements led the design team to choose an innovative solution.

Vertically Segmented Precast Concrete I-Girders

The superstructure system consists of a vertically segmented precast concrete I-girder configuration. This is the first bridge where segmentation is done in the horizontal direction between I-girder segments, and vertically between certain I-girder segments and the haunches below them at the piers. The design accommodates segment weights of up to 330 kip (150 metric tons) and maximum segment depths of 9.8 ft (3.0 m). These specifications were governed by hauling conditions through a carefully mapped route. This design approach significantly simplified logistics while meeting structural performance requirements.

A critical engineering challenge emerged at the intermediate piers, where negative moment regions demanded a structural section that was significantly deeper than the precast concrete girders (approximately 14.75 ft [4.50 m] versus 8.2 ft [2.5 m] maximum). To resolve this issue, the design team developed a variable-depth, U-shaped precast concrete haunch that integrates vertically with the I-girder to produce a fully composite girder varying in depth from 8.2 to 14.75 ft (2.5 to 4.5 m) (Fig. 1). The connection

was detailed to ensure full structural continuity, enabling the assembled system to behave as a monolithic member under both service and ultimate (strength) load conditions. This innovative concept builds on findings from earlier full-scale experimental testing performed by Tadros and Girgis at the University of Nebraska.¹

Structural Configuration

The bridge superstructure is composed of four continuous girder lines, each assembled from seven precast concrete I-girder segments and two U-shaped precast concrete haunched segments ranging between 131 and 164 ft (40 to 50 m) in length (Fig. 1). These girders have wide top flanges that also act as "subpanels" for a cast-in-place concrete deck slab, thus avoiding the need for deck forming. The gaps between the top flanges of adjacent I-girders were filled using narrow hanging formwork from the top deck.

The superstructure is supported by architecturally expressive cast-in-place Y-shaped piers at the intermediate supports and by a single-column pier at the ends of the back spans. Beyond their visual impact, the Y-shaped piers improve load distribution, reduce effective span lengths, and enhance the overall stiffness of the structure.

Jointless Superstructure, Integral with Piers

An important feature of the 885-ft-long (270-m) bridge is that it has a fully continuous, jointless superstructure. Expansion joints are eliminated within the main river span and are only located at the ends of the back spans. Furthermore, there are no bearing devices between the superstructure and the piers; such devices would require significant maintenance and periodical replacement. This strategy

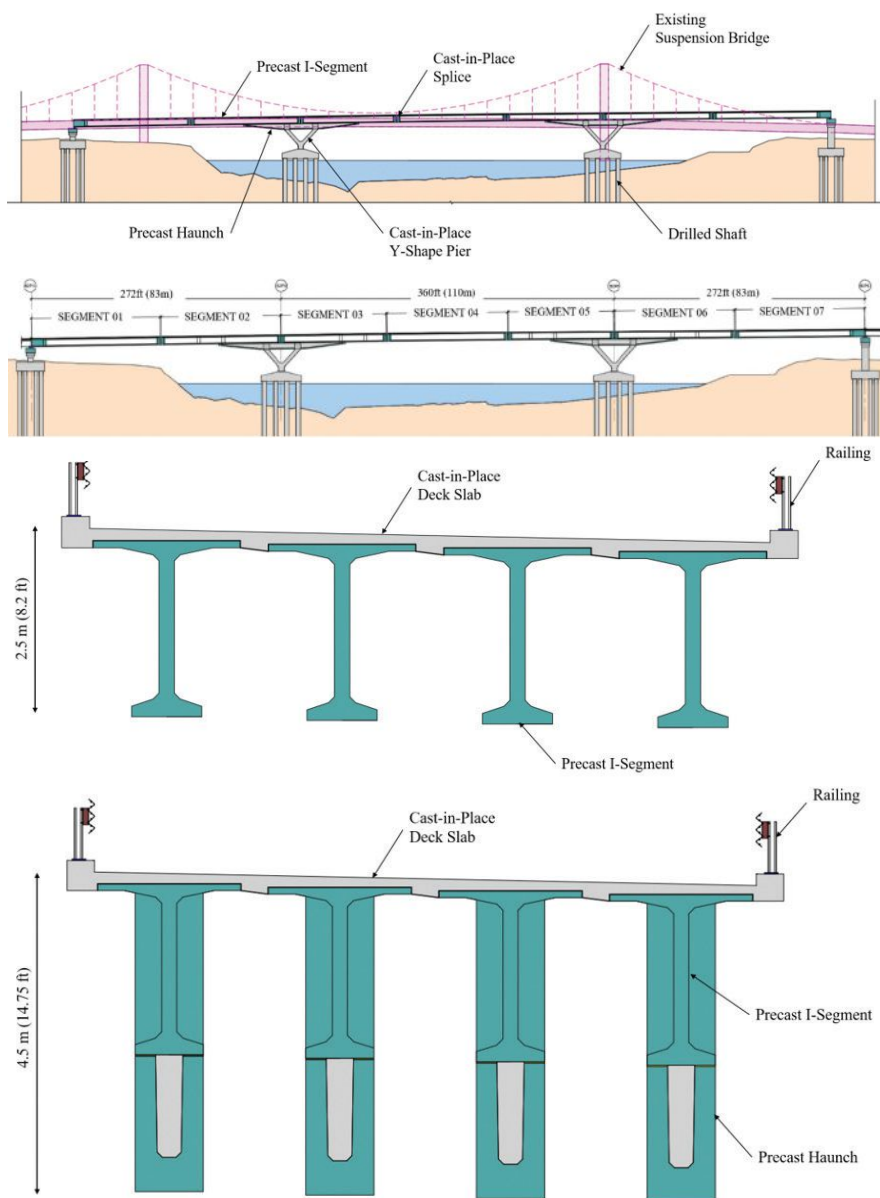


Figure 1. Rendering of the Al-Masafi River Crossing Bridge showing, from top to bottom, the proposed bridge elevation with the nearby existing bridge in the background, bridge segmentation, typical cross section at midspan, and typical cross section over the Y-shaped pier. All Figures and Photos: e.construct.

should help improve rideability and reduce maintenance.

Design Verification Processes

The selected precast concrete solution was developed as a value-engineered alternative to the cast-in-place box-girder concept. Comprehensive structural analysis was performed using three-dimensional structural engineering and finite element analysis software, including detailed construction-stage modeling to evaluate time-dependent effects such as creep, shrinkage, and prestress losses. To further validate the design, the engineer arranged for two independent third-party reviews. The multiple studies provided confidence in the predictions of structural behavior throughout production, erection, and long-term service conditions.

As the project advanced, the project team developed custom solutions for each of the engineering and construction challenges they encountered. These challenges and custom solutions included the following:

- The top flange of the I-girder was extended to eliminate the need for stay-in-place deck subpanels.
- The spans were shortened by designing an efficient Y-shaped pier at the main span.
- Attempts to develop a self-supporting erection system without shoring were abandoned due to the resulting excessive deflections.
- Even with the addition of the significant haunch, the negative moment section at the face of the haunch was enhanced with additional reinforcement and steel plate to meet flexural strength requirements.

Substructure Modeling

A major analytical consideration for this project involved accurately representing the substructure behavior and its interaction with the superstructure. The design team evaluated several modeling assumptions before adopting a final scheme that incorporated partial shaft length with fixed supports at the design scour depth. The shaft caps were modeled using their actual geometry and stiffness properties, resulting in a realistic simulation of the bridge's lateral response.

Two shaft cap locations were studied: one above the river level and another below the riverbed. The submerged option theoretically offered structural advantages due to lower lateral stiffness, leading to reduced restrained forces in the superstructure; however, the above-water configuration was selected because it promised improved constructability.

Construction-Stage Stability

Construction-stage analysis identified the need for temporary shoring systems to control deflections during erection. These systems provided geometric precision, alignment control, and structural stability before fully composite action among the bridge elements was established.

Composite Connections

Special attention was given to the interface regions between bridge elements, particularly at the connections between the cast-in-place Y-shaped piers, precast concrete haunch segments, and precast concrete I-girders. High-strength, nonshrink grout was specified for these joints to provide reliable shear transfer, dimensional stability, and long-term durability, and to ensure fully composite behavior under the final loading condition. Although the original research by Tadros and Girgis³ demonstrated that cast-in-place concrete could be used successfully for these joints, the designers for this project took a more conservative approach by specifying a highly flowable, nonshrink grout. In addition to improving constructability by facilitating placement within the confined joint regions, the nonshrink grout provides excellent bond characteristics and minimizes shrinkage, contributing to the long-term performance of the connection.

Three interface shear-critical planes (identified by the blue, red, and green

color coding in Fig. 2) were evaluated at the vertical segmental connection between the precast concrete haunch girder and the precast concrete I-beam to ensure fully composite action of the assembled component. Each interface was designed in accordance with the interface shear provisions in the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*² to provide reliable horizontal shear transfer across the precast concrete-to-precast concrete connection. Although the AASHTO LRFD specifications permit a demand-to-capacity (D/C)

ratio of up to 1.0, additional interface shear reinforcement was intentionally provided to limit the D/C ratio to 0.60 or less. Because this innovative vertical segmental connection detail was being implemented for the first time in a bridge project, the design team chose this conservative design approach to provide additional design margin and boost confidence in the long-term performance of the connection.

Compared with a U-beam or box beam, an I-beam has a relatively small bottom flange. Therefore, at the termination of the precast concrete haunch section, the

ultimate flexural capacity of the I-girder section is governed by compression-controlled behavior. To enhance the load-carrying capacity at this location, a hot-dipped galvanized steel plate was integrated into the bottom flange region of the girder (Fig. 3). The steel plate provided additional compressive resistance. In addition, large compression reinforcing bars were placed in this highly congested area.

Prestressing Strategy

Pretensioning was used during fabrication to resist handling and self-weight forces. Thirty-six 0.6-in.-diameter (15.7-mm)

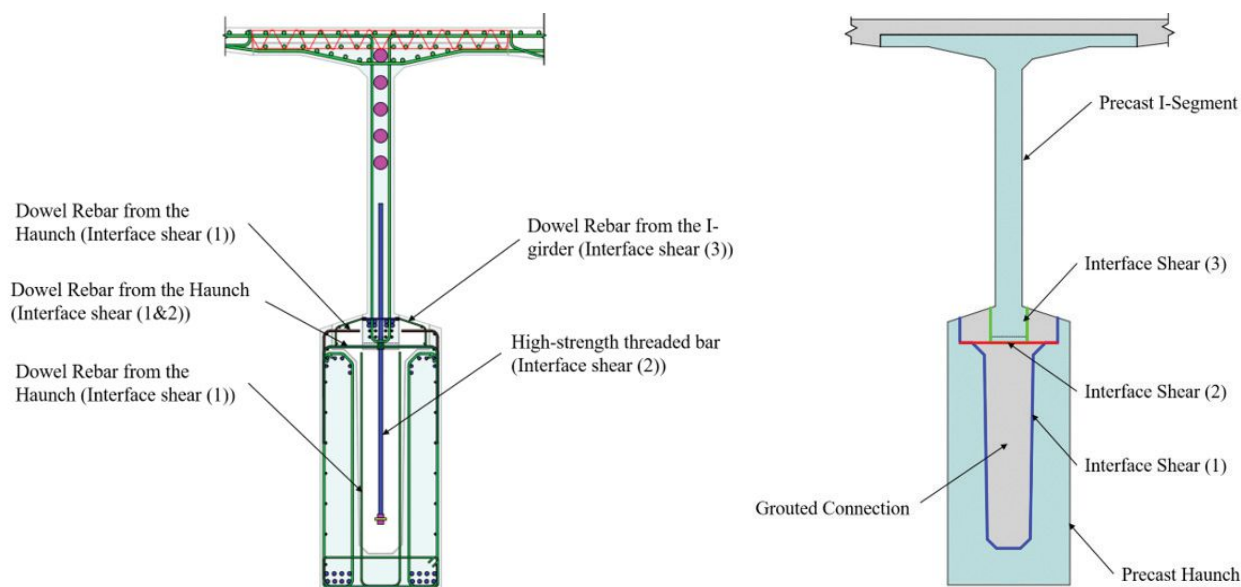


Figure 2. The vertical segmental connection between the precast concrete haunch girder and the precast concrete I-girder was designed using interface shear reinforcement (left) and by evaluating the critical planes for interface shear (right) to ensure that the haunch behaves with full composite action.

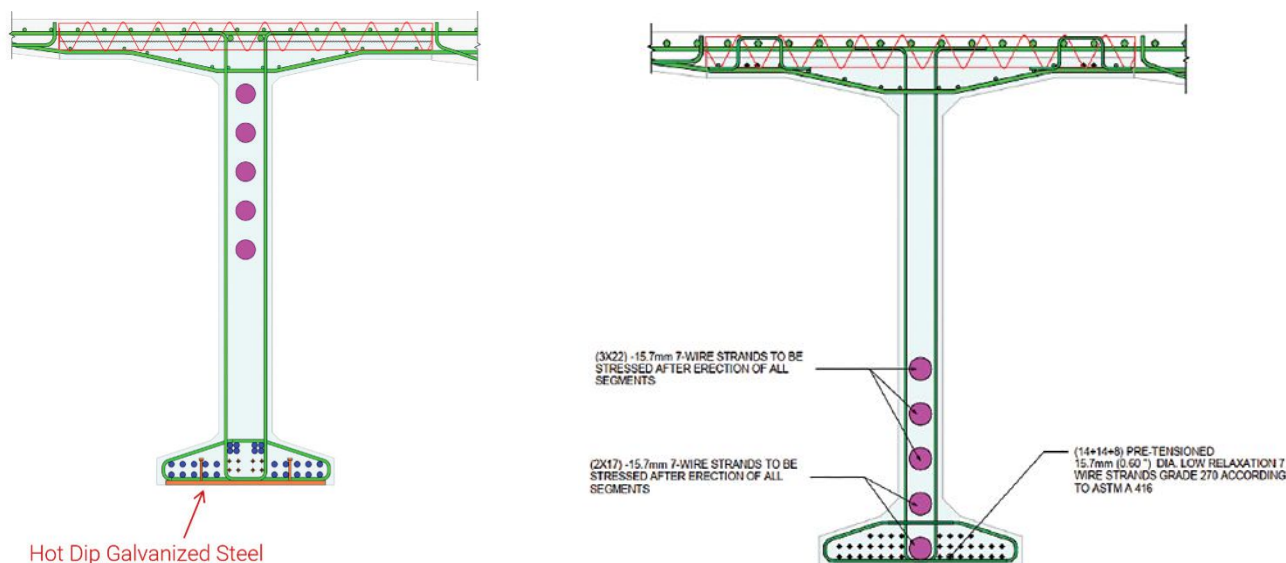


Figure 3. Hot-dipped galvanized steel plate details at the termination of the precast concrete haunch section.

Figure 4. Arrangement of post-tensioning and pretensioning reinforcement in the I-girder cross section at midspan.

straight strands, which were debonded at the ends, were used (Fig. 4). Following erection, continuity was established using draped post-tensioning tendons. The bonded tendons were tensioned from both ends.

Quality Assurance with Full-Scale Assembly at the Plant

To mitigate construction risk, a trial assembly of the critical connection of the precast concrete haunch to the I-girder was conducted at the fabrication facility using the actual components. This “measure, verify, and adjust” approach allowed the project team to validate dimensional tolerances, connection geometry, and erection procedures under controlled conditions (Fig. 5).

Construction Sequence Strategy

The construction of the bridge followed a carefully planned sequence. Construction began with the installation of cast-in-

place drilled shafts and caps, with precast concrete wall panels serving as cofferdams and permanent formwork for the shaft caps to reduce temporary work and accelerate the construction schedule. The cast-in-place Y-shaped piers were then constructed, with temporary shoring systems used for support during the early stages of girder erection (Fig. 6).

Following completion of the substructure, workers erected precast concrete haunch segments over the intermediate piers (Fig. 7), and the precast concrete pier caps for the back spans were installed, enabling efficient integration of the substructure and superstructure. Precast concrete I-girder segments were subsequently erected and connected through cast-in-place splice joints and diaphragms to establish structural continuity.

Once the girder system was assembled, post-tensioning was performed to develop

full continuity and achieve the required structural performance. Next, the cast-in-place composite concrete deck slab was constructed, and then the temporary supports were removed, allowing loads to be transferred to the completed structure. The bridge was completed with the installation of precast concrete barriers and the application of the wearing surface to the final road profile.

Conclusion

The Al-Masafi bridge, which is expected to be opened to traffic in mid-2027, represents the early adoption of an innovative segmented precast concrete I-girder system. The challenging urban environment of Baghdad inspired the team to develop a composite haunch that was spliced to the I-girder. The record span achieved in this design gives hope for many more similar applications of spliced I-girder bridges that are spliced both horizontally and vertically. This vertically segmented concrete system has now moved from concept to reality and can be used to extend spans around the globe.

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Figure 5. Trial fit-up at the precast concrete producer's yard using actual I-girder and haunch segments.

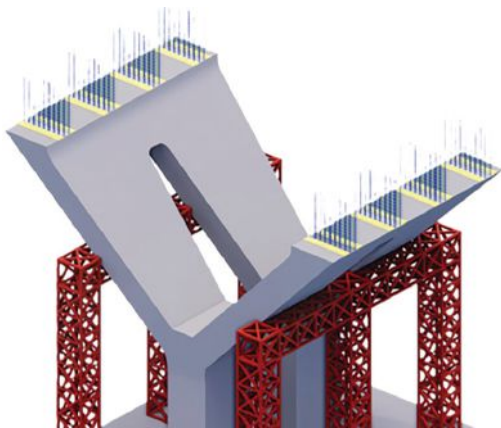


Figure 6. Rendering of the Y-shaped pier showing projecting dowels for the connection to the precast concrete U-shaped haunch segment..



Figure 7. Installation of the first and second precast concrete U-shaped haunch girders.

Shifting Sands: Unlocking Cost-Effective Ultra-High-Performance Concrete with Natural Sand Alternatives

by Dr. Sherif El-Tawil, Dr. Sukhoon Pyo, and Zhengye Tang, University of Michigan; Dewayne Rogers, Clare County Road Commission; and William Hazelton and Jon Winckler, St. Clair County Road Commission

Ultra-high-performance concrete (UHPC) is specified for demanding bridge and structural applications that require high compressive strength, low permeability, enhanced durability, and improved resilience. In many applications, the structural efficiency of UHPC delivers compounding savings. Lighter sections reduce handling and transportation costs; smaller foundations are required; and the extreme durability of UHPC dramatically lowers long-term maintenance and replacement expenditures. However, the broader adoption of UHPC has long been constrained by initial material cost. A significant driver of that cost is quartz (silica) sand, a precisely graded, processed aggregate that is expensive to source and transport. The research described in this article asked a straightforward question: Could widely available, inexpensive natural sands serve the same purpose? The answer, backed by both laboratory and field data, is a resounding yes!

The Problem with Quartz Sand

Finely graded quartz sand is used in traditional UHPC mixtures because of

its high purity and favorable particle characteristics. However, quartz sand typically requires energy-intensive processing and transportation over long-distance supply chains, and those requirements contribute significantly to material cost and environmental impact. As the use of UHPC transitions from niche applications to broader infrastructure deployment, the high cost of silica sand and the associated sustainability concerns have become increasingly important considerations.

Natural sands such as mason sand, golf course sand, and bunker sand are locally available across the United States, require minimal processing, and cost 95% less than quartz sand. This study, conducted at the University of Michigan using an open-source UHPC mixture developed by El-Tawil et al.,¹ systematically evaluated whether these sands could replace quartz sand without compromising performance.

The Sands Tested

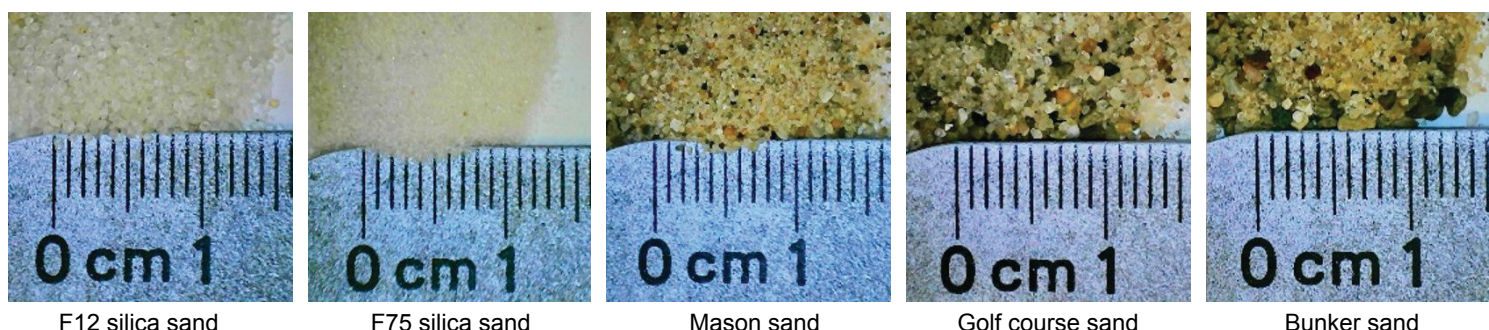
Four natural sands were evaluated alongside two reference quartz sands

(F12 and F75) that are conventionally combined in UHPC production (Fig. 1). Each natural sand candidate has distinct characteristics:

- Mason sand (ASTM C144² and ASTM C33³) is well graded, typically sourced from river deposits or crushed rock, and widely available from masonry suppliers.
- Golf course sand meets United States Golf Association (USGA) topdressing guidelines,⁴ with rounded to subangular particles and tight size control.
- Bunker sand meets the USGA characteristics outlined for golf bunker performance⁵ with a high proportion of medium-sized particles (0.25–1.0 mm [0.01–0.04 in.]) and an angular, gritty texture.
- 3CS (3 Coarse Sand designation), a fine sand found locally in Michigan, was tested only under field conditions in this study.

All UHPC mixtures used the same binder system: Type IL cement, Grade 100 ground-granulated blast-furnace

Figure 1. Particle shape of the sands (shown from left to right): F12 silica sand, F75 silica sand, mason sand, golf course sand, and bunker sand. All Figures and Photos: University of Michigan.



slag, and undensified silica fume with a polycarboxylate-based water-reducing admixture. Each mixture was reinforced with 13-mm-long (0.5-in.), 0.20-mm-diameter (0.01-in.) brass-coated striated steel fibers at 1.5% by volume. That fiber dosage, which is less than the typical U.S. dosage of 2%, is consistent with prior work on this fiber type.⁶ Figure 2 depicts the particle size distributions of the six different sand types used in the study.

Laboratory Results

Table 1 summarizes the compressive strength and flowability results achieved in the laboratory. All four mixtures that included natural sands far exceeded the 120 MPa (17.5 ksi) compressive threshold required for UHPC classification in the American Association of State Highway and Transportation Officials' *Guide Specifications for Structural Design with Ultra-High Performance Concrete*.⁷ The silica sand reference mixture (SS) reached 191.6 MPa (27.8 ksi), whereas the mason sand (MS), golf sand (GS), and bunker sand (BS) mixtures achieved compressive strengths of 183.8, 178.6, and 176.5 MPa (26.7, 25.9, and 25.6 ksi),

respectively. The lowest strength of the alternate sand mixtures is within 8% of the silica sand reference.

The tensile strength results were equally compelling, and, in some cases, the natural sands outperformed the SS reference (Fig. 3). The GS mixture achieved the highest peak tensile strength at 10.50 MPa (1.52 ksi), compared with 9.80 MPa (1.42 ksi) for the SS reference. The MS and BS mixtures reached tensile strengths of 10.19 and 10.35 MPa (1.48 and 1.50 ksi), respectively. All mixtures exhibited pronounced strain hardening after cracking, a hallmark of effective fiber bridging and a key characteristic of UHPC.

Tensile and compressive strength performance do not always trend together. The BS mixture ranked lowest in compressive strength yet delivered outstanding tensile performance. This difference in performance is because compressive strength depends primarily on particle packing efficiency, whereas tensile behavior is governed by matrix quality and fiber-matrix bonding. The angular texture of bunker sand seems to enhance interfacial bonding with the fiber surface, enabling superior post-cracking resistance.

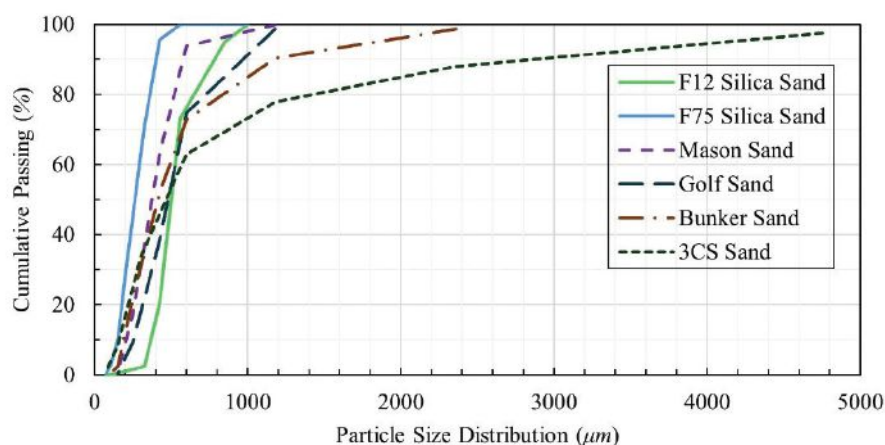


Figure 2. Particle size distributions of various sands.

Table 1. Compressive strength, tensile strength, and flowability of laboratory specimens

Series	Sand type	Flowability, mm	Compressive strength, MPa	Tensile strength, MPa
SS	Silica (F12/F75)	254	191.6 ± 4.36	9.80
MS	Mason	227	183.8 ± 4.40	10.19
GS	Golf	225	178.6 ± 5.19	10.50
BS	Bunker	232	176.5 ± 4.51	10.35

Note: Series 3 Coarse Sand (3CS) was not tested in the laboratory. 1 mm = 0.0394 in.; 1 MPa = 0.145 ksi.

Field Results

Although the laboratory results in this study were encouraging, field conditions are where materials face the real test. Field specimens were made using mason sand and 3CS sand, mixed by workers with no prior UHPC experience, using an IMER-750 pan mixer, a deliberately practical scenario (Fig. 4).

Two challenges emerged. First, the natural sands arrived wet from washing, so on-site moisture measurement was required to adjust water content in the mixtures. Because UHPC is sensitive to water dosing, accurate adjustment is essential. Second, the mixer lacked sufficient power and nearly stalled when the mixture turned over (became fluid). A slight increase in water content (from 0.24 to 0.25 of the cementitious weight) was needed to keep the mixer running. An uncalibrated moisture meter on site added further uncertainty.

Despite these complications, both field mixtures produced viable UHPC. The MS field mixture reached 152.4 MPa (22.1 ksi) in compression (Table 2). That result is well above the 120 MPa (17.5 ksi) minimum, although it is lower than the laboratory value of 183.8 MPa (26.65 ksi); the difference in results is primarily due to the higher effective water content used in the field. The peak tensile strength in the field was 8.81 MPa (1.28 ksi), about 13% below the laboratory value, whereas the localization strain was 10% higher in the field than in the laboratory, indicating good ductility.

Experience from other projects has shown that truck mixers eliminate many of these field challenges, producing results closer to those obtained in the laboratory without the need for water adjustments during the mixing process.⁸

The Cost Case

The financial argument for UHPC made with natural sand is clear. When golf course sand (representative of all natural sand alternatives tested) is used, the total material cost is approximately \$753/yd³ (Table 3). The equivalent mixture using silica sand would cost \$1068/yd³, 42% more than the golf course sand mixture. In comparison, proprietary UHPC products can cost more than \$2000/yd³.⁹ The dominant cost driver in UHPC products is steel fiber. In this study,

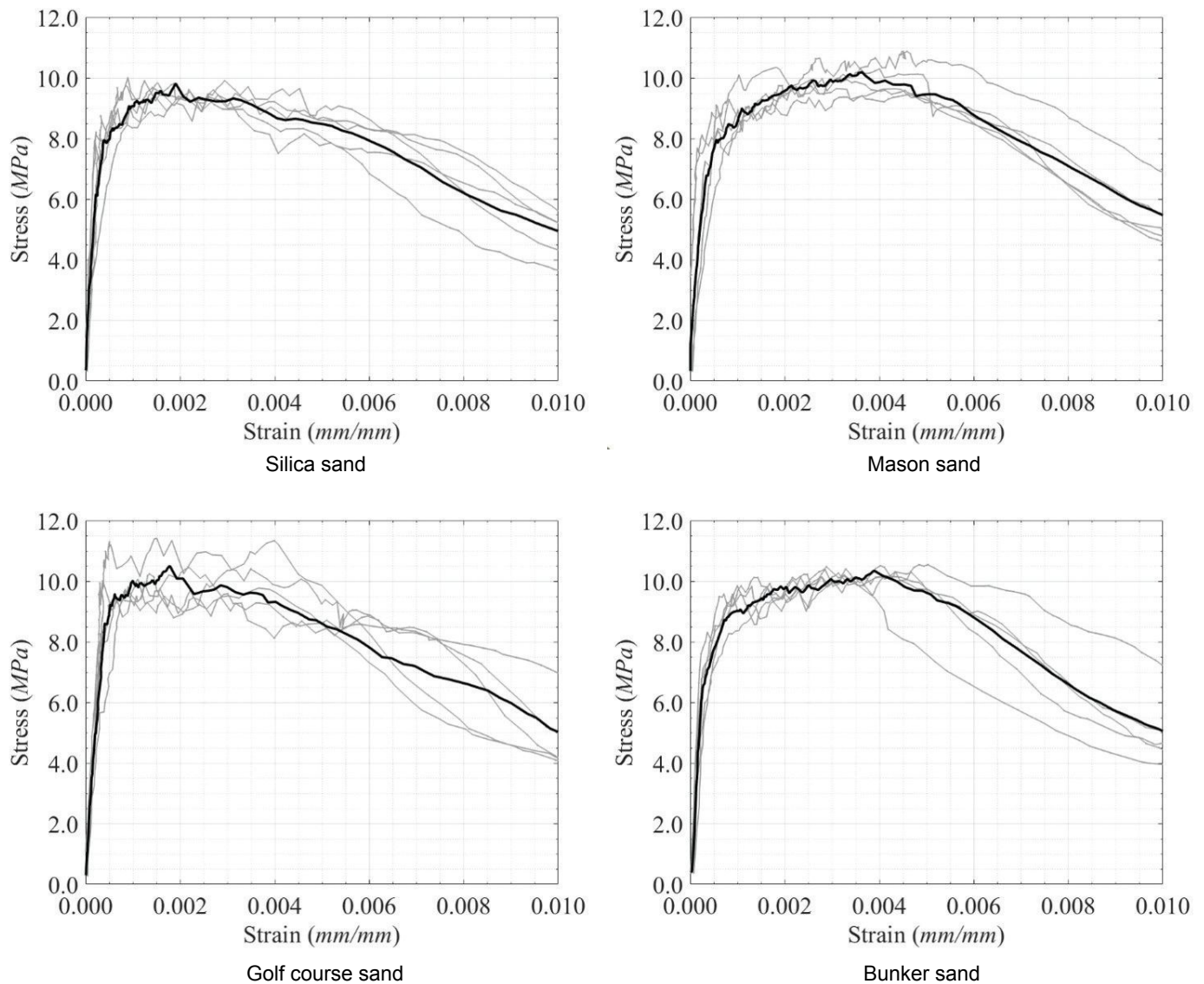


Figure 3. Tensile stress and strain curves of laboratory-mixed ultra-high-performance concrete using silica sand, mason sand, golf course sand, and bunker sand.
Note: 1 mm = 0.0394 in.; 1 MPa = 0.145 ksi.

Table 2. Compressive strength of field-mixed specimens

Series	Sand type	Flowability, mm	Compressive strength, MPa
MS	Mason	254	152.4 ± 5.17
3CS	3CS local	227	149.3 ± 3.00

Note: 3CS = 3 Coarse Sand. 1 mm = 0.0394 in.; 1 MPa = 0.145 ksi.

Table 3. Component unit prices and costs in 2023/2024 dollars

Component	Price	Amount per yd ³	Cost per yd ³
Quartz (silica) sand	\$0.165/lb	—	—
Golf course sand	\$0.005/lb	1971 lb	\$9.86
Cement	\$0.11/lb	653 lb	\$71.83
Slag cement	\$0.09/lb	653 lb	\$58.77
Water-reducing admixture	\$5.45/gal.	4.7 gal.	\$25.62
Silica fume	\$0.45/lb	326 lb	\$146.70
Steel fiber (1.5% by volume)	\$2.20/lb	200 lb	\$440.00
Total			\$752.78

the steel fiber cost \$440/yd³ (58% of total cost) at the 1.5% dosage used. Depending on fiber type and supplier, conventional steel fibers at the standard 2% dosage would cost in the range of \$500/yd³ to \$600/yd³, for a total mixture cost that would be far less than that of the silica sand alternative or proprietary products. At less than \$10/yd³, natural sand represents a fraction of overall UHPC cost regardless of fiber selection, making a compelling case for its use as a lower-cost alternative.

Conclusion

This study demonstrates that mason, golf, bunker, and 3CS sands can fully replace quartz sand in nonproprietary UHPC mixtures without reducing structural performance and may improve tensile behavior. At a small



Figure 4. Field-mixing process: (a) Worker loads steel fiber into IMER-750 mixer; (b) molds for making specimens in the field; (c) flow testing to ensure viability of the ultra-high-performance concrete mixtures.

fraction of the cost of silica sand, these locally available materials offer a practical path to making UHPC more accessible for bridge rehabilitation, accelerated construction, and demanding infrastructure applications.

Two practical lessons from field implementation deserve emphasis. First, moisture management of natural sands is critical and requires calibrated meters and an experienced site engineer. Second, mixer capacity must be sufficient to handle the UHPC mixing process without stalling. Truck mixers are preferred where feasible.

With properties matching those of commercial UHPC, nonproprietary natural sand UHPC offers comparable performance at substantially lower cost, supporting its potential for wider use in the industry. Further technical details about this research may be found in Tang et al.¹⁰ and Pyo et al.¹¹

Acknowledgments

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A Crack Is Not a Crack: Root Cause Investigation and Monitoring Strategies

by Dr. Oguzhan Bayrak, University of Texas at Austin

This article, which is the ninth in this series, focuses on root cause investigations, development and deployment of monitoring systems, and interpretation of monitoring data related to concrete cracking. It presents a high-level discussion and is not intended to cover all cases and situations that we may encounter when evaluating concrete bridges for cracking. To facilitate this discussion, let us use two distinctive examples.

Example 1: Large Footing Affected by Alkali Silica Reaction and Delayed Ettringite Formation

Let us consider a case in which a column supported on a footing shows signs of map cracking. As part of the root cause analysis, several cores are taken from the column, and those samples suggest that the damage may result from alkali-silica reaction (ASR). In many routine bridge inspections, investigators do not need to expose a footing and examine its condition. However, in this case, the investigative team considers the background information and findings for the columns and decides to expose the footing for examination. **Figure 1** shows the appearance of the exposed footing.

Because the general appearance of the crack pattern shown in Fig. 1 supports the possibility of ASR in the footing, additional coring is conducted. The petrographic examination of the cores from the footing reveals internal microcracking of concrete attributable to ASR, as well as evidence of delayed ettringite formation (DEF). In other words, the observed cracking is due to ASR and DEF expansions. While most cracking can be attributed to ASR, the

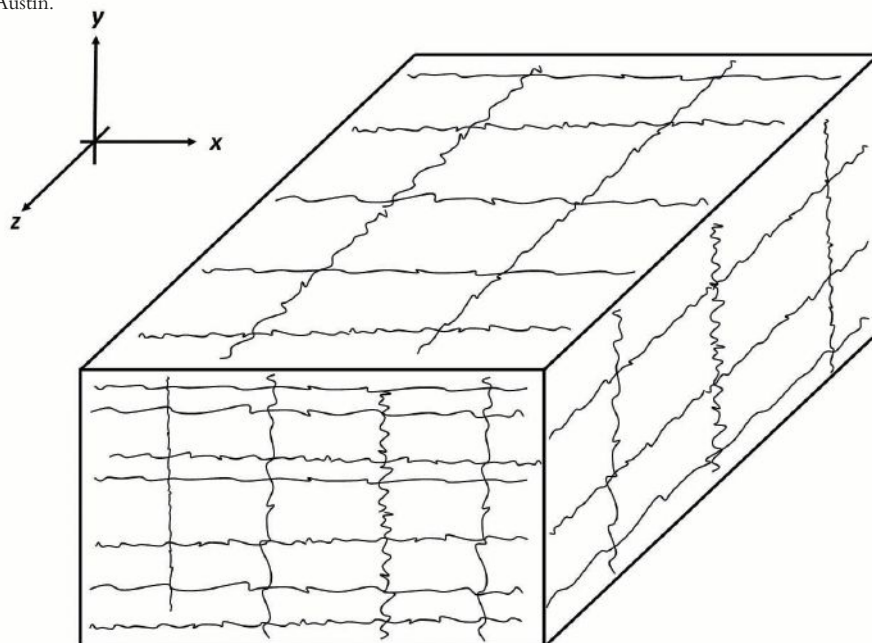
presence of DEF makes the situation a little more complicated because maximum expansion potentials of DEF are typically higher than ASR. (For more information on ASR and DEF, see articles in the Summer 2018 and Spring 2019 issues of *ASPIRE*®.)

With these observations serving as a backdrop, the team decides to use Concrete Works software^{1,2} to further investigate whether the damage was related to DEF. Thermal modeling of the concrete footing was conducted with incomplete information because the time of day at which the footing concrete was placed was unknown. This modeling led the investigative team to conclude that curing temperatures were likely in the 170°F to 175°F range. Because that temperature range is above the DEF-threshold of approximately 158°F, results from the

thermal modeling support the findings of the petrographic examination. In summary, the investigation that started out with map cracking readily observed in a column reveals that the column has ASR-related cracking and the footing supporting the column is affected by both ASR and DEF.

Given the findings from the root cause analysis, instrumentation of the footing with strain gauges to monitor further cracking is warranted (as is instrumentation of the column supported on the footing, but that is not the subject of this discussion). As we proceed, we need to remember the fundamental physics of the degradation mechanisms at play. Because ASR and DEF are bulk expansion mechanisms, it is appropriate to measure expansions in the x, y, and z directions (Fig. 1) and calculate the volumetric expansion due to ASR and

Figure 1. Footing with extensive cracking in all three directions. All Figures: University of Texas at Austin.



DEF. This information helps us determine the structural implications of the observed degradation of the footing.

Another key consideration is the expansions that occurred before the investigation. That is to say, while the instrumentation that we deploy can measure expansions that occur after the strain gauges are placed, we also need to use engineering principles to estimate how far ASR and DEF have progressed to date. To that end, the Damage Rating Index (DRI) may, at first glance, seem to be a useful tool.

DRI is a technique used by petrographers in which specific microscopic damage features are counted or quantified and then multiplied by weighting factors and summed to create a single DRI value. While tracking the DRI values and the change in those DRI values over time may potentially offer benefits, it is not possible at present to reliably convert a DRI value into the amount of expansion (the key indicator that relates to structural implications). Different concrete mixtures may produce different DRI values for the same expansion levels, and exact documentation of this type of information remains incomplete and therefore uncoded. Further, the DRI method is subjective, so DRI values calculated by one petrographer typically do not match the values calculated by another one.

Another technique to estimate the amount of expansion caused by DEF and ASR uses the gauge lengths along the x, y, and z axes. From these lengths, we can assume that expansions are concentrated in crack openings and any straining in the concrete between the cracks is much smaller in magnitude. In this way, the widths of the cracks can be measured by using crack comparators or crack microscopes, and an estimate for the expansions can be obtained. Estimating expansion values by this technique is based on several assumptions, which should be considered when evaluating the determined values. Those assumptions are as follows:

- The contribution of straining in between the cracks is omitted. This is omitted because these deformations are smaller in magnitude in relation to the deformation contribution of crack openings.

- The crack openings are assigned to ASR and DEF expansions only.
- Deformations due to creep- and shrinkage-related volumetric changes are omitted because they are smaller in magnitude in most cases and this omission leads to conservative estimates for the expansions assigned to ASR and DEF.

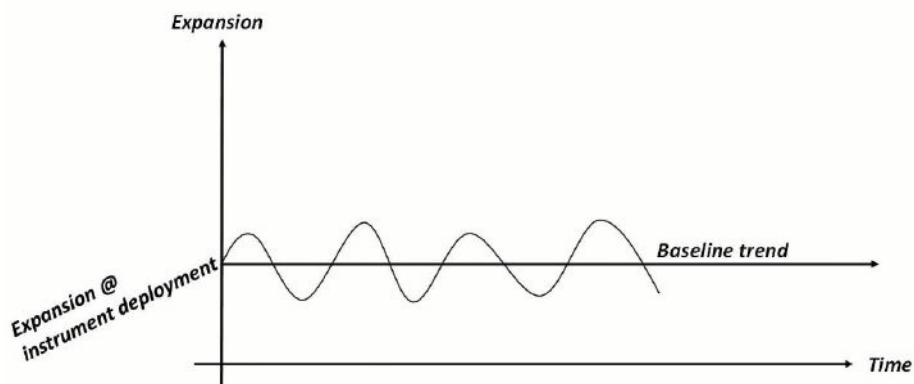
Although these assumptions are common in any engineering analysis on investigations that relate to ASR and DEF, the variables and assumptions involved in a specific investigation should always be explicitly stated in documents that report the findings.

Selection and installation of a reliable deformation monitoring system is also important to measure future expansion. Commonly, surface-mounted targets are used and relative movement of those targets is measured. The durability of the target installation is important because to trend data, measurements must, at a minimum, span over a period of 2 to 3 years. To increase durability, it is advisable to drill holes that reach into the structural core (that is, the holes penetrate behind the reinforcement near the exposed concrete face) and install stainless steel pins using an epoxy adhesive. Measuring from stainless steel pins more accurately represents the expansions experienced by the structural core than those measured from surface-mounted targets. Once the stainless steel pins are installed, initial readings can be taken in the x, y, and z directions and the datum of expansions in each of these three directions can be set. Importantly, future expansions can be measured by using these pins and their relative movements against each other.

Let us next discuss possible trends that may be observed in the data. By focusing our attention on the expansion readings taken in the x, y, and z directions, we can see an expansion-time history similar to that shown in Fig. 2.

In Fig. 2, the vertical line offset seen in the expansion axis is determined using the expansion-to-date estimation technique described previously. The uncertainty of this estimation is repeated here to emphasize that this uncertainty must be included in the subsequent engineering analysis in a conservative manner. Once this datum is established, the almost sinusoidal trend depicted in Fig. 2 can be examined. The expansion behavior shown in Fig. 2, which is one of the cases we will examine herein, is idealized (that is, near sinusoidal) to simplify the subsequent discussion. In actuality, the expansion behavior may be more “jagged” as it may be influenced by daily live-load application on the superstructure, daily temperature variations, and other factors. This subtlety notwithstanding, the overall behavior shown in Fig. 2 is largely influenced by seasonal temperatures, and the baseline trend in this figure seems to be constant. This baseline trend implies that the cracks depicted in Fig. 1 are opening and closing as the seasonal temperature is fluctuating. From these observations, it can be interpreted that both ASR- and DEF-related expansions are influenced by daily live-load application on the superstructure, daily temperature variations, and the exposure conditions that are present on site. In this case, the maximum expansion measurement can be taken from the monitoring systems and converted into structural implications of that expansion

Figure 2. Expansion in the x, y, or z directions shown in Fig. 1, case 1.



level by using guidance, such as those presented in the *fib* (International Federation for Structural Concrete) Model Code 2020.³ This evaluation can be done in all applicable directions; most importantly, it can be done in the direction and behavioral mode that controlled the design in the first place. For example, if shear strength of the footing was the controlling behavioral mode in the original design, we would be most interested in evaluating that mode. If bending about one of the axes of interest controlled the design, that would be our starting point/primary focus of interest.

Let us now consider another possibility in which the baseline trend is not as benign as the trend shown in Fig. 2. **Figure 3** depicts a case in which the seasonal temperature variations result in expansions and contractions that cycle around a baseline trend that shows a net positive expansion. This type of behavior indicates that ASR-related expansions, DEF-related expansions, or both ASR- and DEF-related expansions are still active. The volumetric (bulk) expansions that result from one or both of these chemical degradation mechanisms are ongoing. In other words, reactants that contribute to the reaction mechanisms have not been depleted, and the environmental exposure conditions are still conducive to promoting these reactions. In such a case, interim structural evaluations are desired for the maximum expansion levels documented. Alternatively, a structural evaluation can be made for expansion levels higher than those measured and may be used as limiting expansion levels for the structural evaluations. If the future expansion trends show the expansion has stopped and the trends look like alternative A in Fig. 3, the structural evaluations based on the expansion trends can be finalized. Provided that the maximum expansion levels support structural capacities that are larger than the demands imposed by the governing load cases, we can conclude that there is no structural load-carrying deficiency at play. In the case of alternative B in Fig. 3, the footing is still undergoing expansion, and continued monitoring and structural evaluation are essential. This type of trend does not mean the structure is unsafe. If the maximum expansion levels indicate a

structural capacity degradation that is large enough to create a challenge with load effects, structural retrofits and/or replacement may be needed.

Example 2: Three-Column Bent with Shear and Flexural Cracks

Let us next focus our attention on a three-column bent that is exhibiting a substantial amount of shear distress (Fig. 4). As a starting point, investigators visually inspect the bent to document the widths of the cracks and map the locations and lengths of the cracks so that subsequent inspections can benefit from this datum. The plan set available for this bent indicates that the crack-control reinforcement in the cap is 0.2% in the x and y directions (Fig. 4). Let us also assume the maximum crack width measured is 0.03 in. Using Table 1 from the Summer 2025 *ASPIRE* article in this Concrete Bridge Technology series, we can determine that the cap has been loaded to

30% to 50% of its capacity. This initial assessment is helpful in decision-making.

In addition to evaluating the margin to the capacity of the cap, the investigators in this case conduct a petrographic examination of three cores taken from the cap. This examination reveals no deleterious chemical degradation mechanisms. However, it points to a higher water-cement ratio of 0.55 than the maximum water-cement ratio of 0.45 specified in the construction documents. The higher-than-specified water-cement ratio will result in a greater drying shrinkage tendency. That tendency is a contributing factor to the observed cracking, but it does not fully explain the formation of shear cracks.

To further investigate the cracked condition of the cap, let us consult Section 5 provisions of the American Association of State Highway and Transportation

Figure 3. Expansion in x, y, or z directions in Fig. 1, case 2.

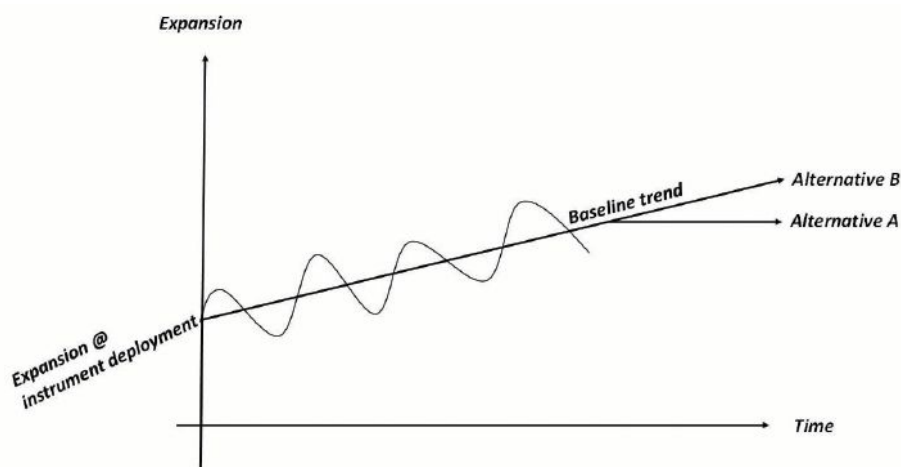
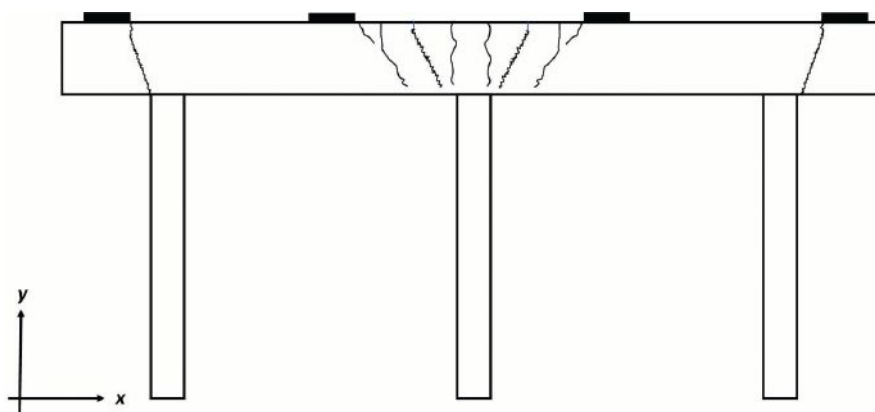


Figure 4. Three-column bent with shear cracking.



Officials' AASHTO LRFD Bridge Design Specifications.⁴ Commentary to Article 5.8.2.2 states the following:

The estimated resistance at which diagonal cracks begin to form, V_{cr} , is determined for the initial geometries of the D-Regions using the following expression (Birrcher et al., 2009^[5]):

$$V_{cr} = \left[0.2 - 0.1 \left(\frac{a}{d} \right) \right] \sqrt{f'_c} b_w d \quad (C5.8.2.2-1)$$

but not greater than $0.158 \sqrt{f'_c} b_w d$,

nor less than $0.0632 \sqrt{f'_c} b_w d$.

where:

a = shear span (in.)

d = effective depth of the member (in.)

f'_c = compressive strength of concrete for use in design (ksi)

b_w = width of member's web (in.)

Where the shear in service is less than V_{cr} , reasonable assurance is provided that shear cracks are unlikely to form.

By using Eq. C5.8.2.2-1 in our example 2, we can understand that shear cracking is expected for the three-column bent shown in Fig. 4. This observation is important in completing the root cause analysis. It is certainly possible to perform a more detailed analysis, consistent with the Levels of Approximation approach articulated in the *fib* Model Code 2020. However, the additional computational effort does not seem warranted in this case because we can explain the observed shear cracking readily, as explained earlier. In addition, restrained shrinkage effects that introduce axial tension forces in the bent cap further exacerbate the cracking potential.

With the apparent margin to failure loads, a minimal amount of instrumentation is needed to monitor the cracks. In this case, inspectors attach targets to a few representative cracks to measure the movement of those cracks. Instruments at three of the widest shear

crack locations will measure the stability of those cracks over time. In addition, the vertical (flexural cracks) over the interior column are also instrumented to complete the instrumentation plan.

Let us next examine a few potential data trends, to complete the picture. Figure 5 shows a potential crack-width time history. In that figure, the widths of the cracks monitored exhibit an opening and closing pattern due to temperature changes experienced by the bent cap. In this case, we have "active" cracks, but they are not "progressive" cracks. They are not progressive because the baseline trend shown in Fig. 5 is horizontal. This condition is quite stable. Therefore, in this case, a justifiable strategy is to perform a durability repair (for example, seal the cracks) and continue the crack-width monitoring for 2 to 3 years. After that monitoring period ends, the original inspection routine, consistent with the owner's inspection program, can be resumed.

Next, let us study the case in which the baseline trend is not horizontal and

the widths of the cracks that are being monitored are increasing over time. This condition signals cracks that are both active and progressive. The progression of cracking should be examined carefully. Specifically, applied loading on the structure, the condition of the foundations, and other potential significant contributors to cracking should be considered.

As the crack widths increase, two scenarios are possible. In the best-case scenario (alternative A in Fig. 6), the baseline trend stabilizes. That is to say, after a few years of progression, the behavior can be deemed stable. In the other alternative (alternative B in Fig. 6), the progression continues and the baseline trend is not stable. In this case, structural retrofit options alongside the replacement option should be evaluated, with due consideration given to safety, durability, and cost. The use of advanced tools, such as the one described by Zaborac et al.,⁶ may also be appropriate.

Conclusion

Although the examples discussed in this article do not cover all of

Figure 5. Shear crack width versus time: case 1.

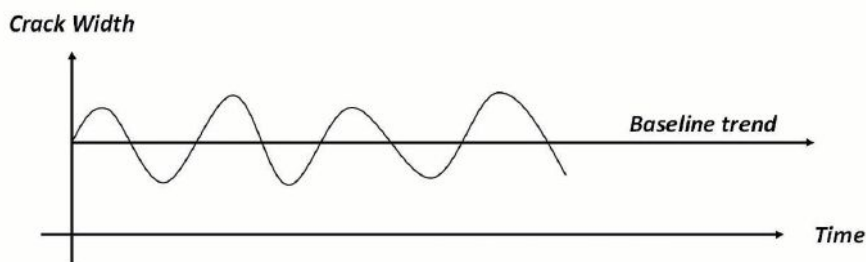
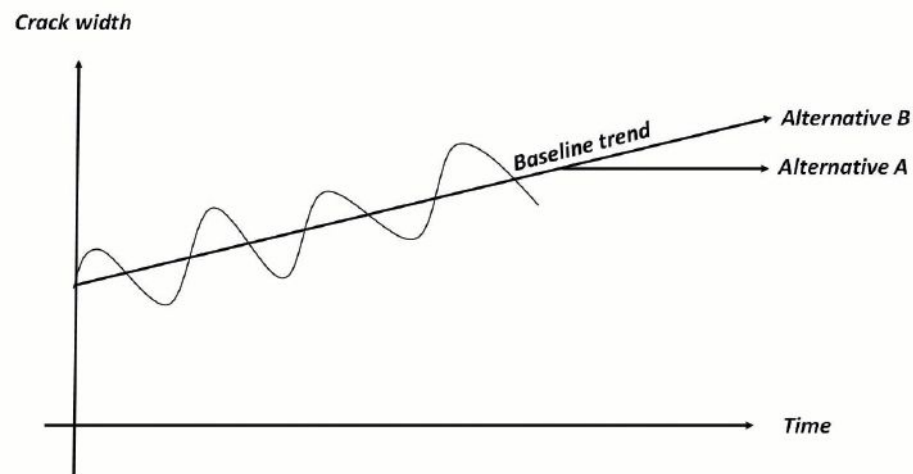


Figure 6. Shear crack width versus time: case 2.



the cases we may encounter, they are notable examples of physical phenomena that can cause distress in structural components. Both examples highlight the need to identify the root cause of cracking before developing any instrumentation or structural monitoring system. Engineers must clearly understand the fundamentals at play before they intervene. In some cases, structural evaluation is reasonably straightforward because there is sufficient guidance available in the bridge industry's design and structural evaluation specifications. In other cases, in-depth linear and nonlinear finite element analyses are necessary.

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

IN THIS ISSUE

<https://deldot.gov/Programs/BridgeDesignComp/index.shtml>

The Perspective article on page 5 highlights the success of the Delaware Bridge Design Competition, which is made possible by the support of Delaware Department of Transportation volunteers, partnering consultant volunteers, and educators. The goal of the competition is to teach science, technology, engineering, and math concepts to students. The website at this link provides details about the competition.

<https://www.aspirebridge.com/magazine/2018Summer/Perspective-OverviewOfDelayedEttringiteFormation.pdf>

The Concrete Bridge Technology article on page 36 discusses the deterioration mechanisms of alkali-silica reaction (ASR) and delayed ettringite formation (DEF). This is a link to an article in the Summer 2018 issue of *ASPIRE*® that provides more information about ASR and DEF.

<https://www.aspirebridge.com/magazine/2024Winter/Project-FirstUSBridgeWith100PercentUHPCSuperstructure.pdf>

One of the authors of the Concrete Bridge Technology article about ultra-high-performance concrete (UHPC) on page 32, Dr. Sherif El-Tawil, also coauthored “First U.S. Bridge with a 100% UHPC Superstructure: A Small Bridge with Big Implications,” a Project article in the Winter 2024 issue of *ASPIRE*. That article, which is available at this link, discusses a superstructure made of an open-recipe (nonproprietary) type of UHPC.

<https://www.aspirebridge.com/magazine/2022Summer/County-ClareCountyRoadCommision.pdf>

This is a link to a Concrete Bridge Technology article in the Summer 2022 issue of *ASPIRE* titled “Clare County Road Commission Seeks Higher Performance at Lower Cost with Open-Recipe UHPC Formula.” Some of the authors of the Concrete Bridge Technology article on UHPC on page 32 of this issue were also involved in the Clare County project.

https://www.fhwa.dot.gov/bridge/concrete/hif19067_Nov2021.pdf

This link provides access to a Federal Highway Administration report, *Lightweight Concrete Bridge Design Primer*, which is referenced in the NCBC Member Spotlight article on page 44.

<https://acmanet.org>

This is the link to the website of the American Composite Manufacturers Association (ACMA). ACMA is a new member of the National Concrete Bridge Council (NCBC) and a focus of the NCBC Member Spotlight article on page 42.

<https://www.stainlessrebarassociation.org>

This is a link to the website of the Stainless Steel Interest Group (SSIG) of the Concrete Reinforcing Steel Institute. SSIG is a new member of the NCBC and a subject of the NCBC Member Spotlight article on page 42.

<https://www.nps.gov/samo/learn/nature/pumapage.htm>

This is a link to a National Park Service (NPS) web page about the mountain lions that live in the urban landscape of Los Angeles, Calif. NPS research on these mountain lions has informed the design of the Wallis Annenberg Wildlife Crossing, which is the subject of a Project article on page 16.

<https://doi.org/10.1016/j.conbuildmat.2024.139001>

The Concrete Bridge Technology article on page 32 reports on recent research investigating the substitution of natural sand for quartz sand in ultra-high-performance concrete (UHPC) mixtures. This is a link to a journal article about the performance of UHPC prepared with manufactured sand.

<https://highways.dot.gov/federal-lands/wildlife-crossings>

This is a link to a web page for the Federal Highway Administration's Wildlife Crossings Program, which offers funding and resources for wildlife crossing projects. Two wildlife crossing projects in California are the subjects of Project articles on pages 16 and 22.

<https://www.aspirebridge.com/magazine/2026Winter/ProfessorsPerspective.pdf>

To supplement the Perspective on page 8, additional information about teaching bridge design-related skills to students can be found in the Winter 2026 issue of *ASPIRE* at this link. The author, Dr. George Okere of the University of Cincinnati, explains the value of teaching students in construction-related programs to read and interpret construction specifications and drawings so they are adequately prepared for industry careers.

<https://aspirebridge.com/magazine/2023Spring/ProfessorsPerspective-PrecastBridgeStudioAdvancesAcademia.pdf>

This link is to a Spring 2023 Professor's Perspective by Dr. Eric Matsumoto at California State University, Sacramento, about how precast concrete bridge studios prepare the next generation of bridge professionals. Like the Perspective on page 8, Matsumoto's article advocates for partnerships between industry and academia to educate students.

<https://www.aspirebridge.com/magazine/2019Summer/AProfessorsPerspective.pdf>

Dr. Richard Miller, the author of this issue's Perspective on page 8, also makes the case for improving the way that engineering is taught in the Summer 2019 issue of *ASPIRE*. In the 2019 article which is available at this link, Dr. Miller argues that engineering should be taught holistically, in the same manner that baseball is taught.

NCBC Welcomes Two New Members: The American Composite Manufacturers Association and the Stainless Steel Interest Group

by Tim Christle, Post-Tensioning Institute

The American Composite Manufacturers Association (ACMA) and the Stainless Steel Interest Group (SSIG) of the Concrete Reinforcing Steel Institute recently joined the National Concrete Bridge Council (NCBC). Their memberships bring fresh perspectives and reinforce NCBC's mission to improve concrete bridges nationwide.

ACMA represents the North American supply chain for the fiber-reinforced polymer (FRP) composites industry, from manufacturers and distributors to the end users. The association has been actively participating in the development of codes and standards in this sector for more than 30 years, working with the American Concrete Institute (ACI), ASTM International, and the American Association of State Highway and Transportation Officials (AASHTO).

"ACMA's membership in NCBC will help increase awareness of FRP composites technology and support the concrete bridge market. Working with NCBC, we can advance knowledge and innovation while addressing the specific needs of concrete bridge construction. We will collaborate with agencies to provide education and support demonstration projects that utilize FRP composites for the repair or replacement of transportation structures," says John P. Busel, senior vice president of the Composites Growth Initiative at ACMA.

SSIG is a newly formed interest group committed to raising awareness and promoting the use of stainless steel reinforcing bars in reinforced concrete applications. The group shares

independent, science-based information about stainless steel's properties and benefits, including its strong corrosion resistance, superior ductility, extended uniform elongation, exceptional toughness, and ability to maintain strength at cryogenic temperatures.

"SSIG has partnered with NCBC to advocate for the improved performance, durability, and sustainability of concrete bridges, a key market for the stainless steel reinforcing bar industry," says Richard Huza, PE, executive director of SSIG.

Industry Emphasis on Repair

The addition of these two new members to NCBC is timely as the transportation industry pushes forward

on expanded repair strategies. The steady growth of rehabilitation projects is fueled by limited funding for new construction, bridge safety updates, and a massive backlog of maintenance projects. As owners adopt data-driven tools such as digital-twin modeling during initial construction, they will likely transition into more planned, long-term rehabilitation projects, rather than total reconstruction.

Many highways and bridges are reaching the end of their design lives, requiring reconstruction. A wave of legacy projects to rebuild aging interstate bridges and upgrade urban and rural routes will join the pipeline. NCBC and its members are prepared to develop programs to address bridges in poor condition and improve highway capacity.

The Gordie Howe International Bridge that connects Detroit, Mich., and Windsor, Ontario, includes 6000 tons of stainless steel reinforcing bar. Photo: Stainless Steel Interest Group of the Concrete Reinforcing Steel Institute.





ACMA Programs

ACMA companies serve a diverse set of construction and infrastructure markets. One of ACMA's committees, the FRP Rebar Manufacturers Council, promotes the use of FRP reinforcement through education; development of quality procedures; industry specifications, codes, standards, and guidelines for field application; and legislative advocacy. ACMA is an accredited ANSI standards developing organization, and it creates standards and specifications that inform best practices and ensure quality in the use of composites in many markets.

ACMA has successfully collaborated with the AASHTO Committee on

Bridges' former T-6 technical committee, which focused on FRP composites, and is now part of the Concrete Structures Committee to develop and implement FRP standard product and design guidelines. The *AASHTO LRFD Guide Specifications for the Design of FRP Pedestrian Bridges*, second edition (2025), and *AASHTO LRFD Bridge Design Guide Specifications for GFRP Reinforced Concrete*, second edition (2018), are examples. In a similar initiative, ACMA and member companies collaborated with ACI to develop codes, standards, and specifications. That collaboration established mandatory language and criteria for designing structural concrete reinforced with glass and carbon FRP bars in ACI CODE-440.11-22, *Building Code Requirements for Structural Concrete Reinforced with Glass Fiber-Reinforced Polymer (GFRP) Bars—Code and Commentary*.

SSIG Initiatives

"Our decision to join NCBC was influenced by its established relationships with public agencies, especially FHWA and AASHTO," says Huza. Industry associations such as NCBC coordinate stakeholder input to government bodies developing standards and guidelines. Ongoing collaboration with these agencies provides opportunities for SSIG to

expand the use of stainless steel reinforcing bars in public projects.

Because stainless steel reinforcement is corrosion resistant, it offers bridge and infrastructure designers durable options for steel reinforcement in harsh environmental conditions and projects that stipulate an extended service-life design without repairs. Such efforts help foster the growth of concrete bridges, which supports the NCBC vision.

NCBC's Future

ACMA and SSIG have similar goals as they work to reduce life-cycle costs relative to traditional concrete bridge design options. Both organizations are raising awareness about corrosion-resistant materials for aggressive environments.

The use of stainless steel reinforcement and FRP composite reinforcement is becoming widely accepted in the concrete bridge industry. By joining NCBC, ACMA and SSIG are helping the coalition improve the durability and performance of the nation's bridge infrastructure. 

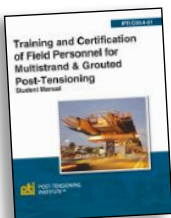
Tim Christle is executive vice president of the Post-Tensioning Institute in Farmington Hills, Mich., and currently serves as the chair of the National Concrete Bridge Council.



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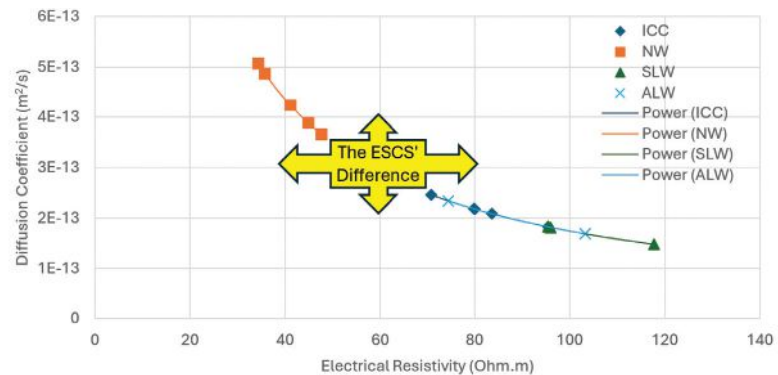
Lightweight Innovations Drive Durable, Sustainable Bridges

by Dr. Fariborz M. Tehrani, Expanded Shale, Clay and Slate Institute

As bridge owners and designers face intensifying demands for durability, resilience, and sustainability, the Expanded Shale, Clay and Slate Institute (ESCSI) continues to lead the advancement of lightweight-aggregate concrete (LWC) as a high-performance solution. The density of lightweight and sand lightweight concrete typically varies from 90 to 135 lb/ft³ (1440 to 2160 kg/m³), while the density of internally cured concrete using lightweight aggregate is comparable with that of normalweight concrete (NWC). Through decades of research, guidance, and collaboration, ESCSI and its members are helping shape a new generation of concrete bridge systems, engineered for long-term performance, not just initial cost.

A Smarter Approach to Concrete

Traditional NWC has served the bridge industry well, but its density contributes to larger dead loads and increased structural demands compared with LWC. LWC produced with expanded shale, clay, and slate (ESCS) offers a more efficient alternative to NWC. By reducing density while maintaining



Concrete containing expanded shale, clay, and slate (ESCS) has increased resistivity and reduced diffusion. Note: ALW = all-lightweight concrete; ICC = internally cured concrete; NW = normalweight concrete; SLW = sand lightweight concrete. Figure: Dr. Fariborz M. Tehrani.

required strength, LWC reduces the weight of superstructures, improves seismic performance, and facilitates design flexibility. The key contribution of LWC to seismic performance is reduced dead weight of the structure, which cuts seismic inertial forces, lowers lateral base shear demand, and decreases bending moments during an earthquake. Equally important, because LWC has a lower modulus of elasticity than NWC, it can better accommodate strain, temperature gradients, and traffic loads while reducing cracking caused by shrinkage. A lower modulus of elasticity provides higher flexibility and reduced strain-induced stresses caused by thermal and impact loads, and other sources of length and volume changes. These characteristics make LWC particularly well suited for bridge decks and pavement components where long-term durability is critical.

Built-In Durability Through Internal Curing

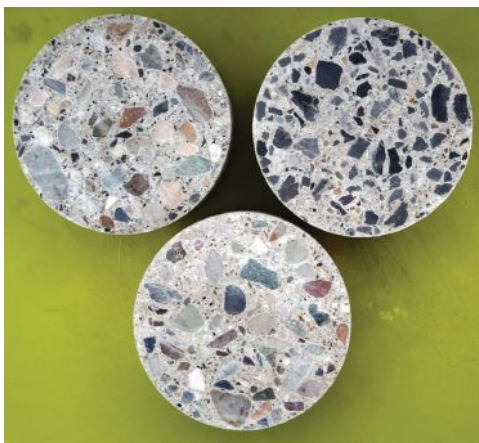
Cracking and chloride penetration remain leading causes of bridge deterioration. ESCSI-supported innovations directly target these issues through lightweight-aggregate technology and internal curing.

ESCS aggregates possess a unique internal pore structure that can

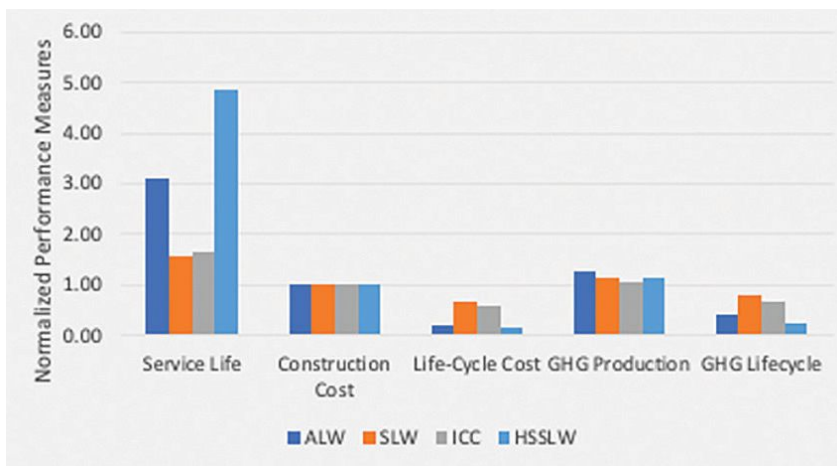
absorb and later release water into the surrounding cement paste. Prewetting lightweight aggregates enables this characteristic to convert pores to reservoirs for internal curing. This internal curing process improves hydration in concrete mixtures, reduces concrete shrinkage, and enhances the interfacial transition zone between aggregate and paste. The enhancement of the interfacial transition zone is due to the pozzolanic nature of ESCS surfaces and stronger hydrated cement infiltrating surficial pores to form stronger bonds. The result is a concrete matrix with lower permeability and improved resistance to chloride ingress, key factors in delaying corrosion of reinforcing steel and extending service life. (For details of the internal curing mechanism, see the FHWA article in the Summer 2024 issue of *ASPIRE*®.) For bridge owners, the characteristics of LWC translate into improved durability in aggressive environments, including those with marine exposure and cold regions where the use of deicing salts is prevalent.

Extending Service Life and Value

Perhaps the most compelling advantage of LWC is its impact on life-cycle performance. Recent research demonstrates that compared with



Cut surfaces of concrete samples: internally cured concrete (top left); sand-lightweight concrete (top right); normalweight concrete (bottom). Photo: Mineta Transportation Institute.



Sustainability performance measures of lightweight concrete. Note: ALW = all-lightweight concrete; GHG = greenhouse gas; HSSLW = high-strength sand lightweight concrete; ICC = internally cured concrete; SLW = sand lightweight concrete.

Figure: Dr. Fariborz M. Tehrani. Adapted from Mineta Transportation Institute.

conventional concrete, LWC and internally cured concrete can extend the service life of transportation structures by nearly five times in certain applications.¹ This extension dramatically reduces the maintenance, repair, and rehabilitation required for typical structures. In this same research, life-cycle cost analyses show associated savings of up to 85%, driven by reduced maintenance demands and longer replacement intervals.¹ At a time when transportation agencies are under pressure to do more with limited funding, these life-cycle benefits are driving an increased interest in LWC solutions.

Delivering Measurable Sustainability Gains

Sustainability is no longer optional in infrastructure projects; it is a core requirement. By enhancing durability and long-term performance, lightweight aggregates support meaningful reductions in the environmental impact of concrete structures.

Life-cycle assessments indicate that compared with conventional concrete, LWC can reduce greenhouse gas emissions by up to 76%. These assessments are based on predicted service life of concrete exposed to corrosive environments, such as areas where deicing agents are used or marine zones. Lower chloride diffusion within the body of the concrete containing lightweight aggregates is the key factor extending the service life of concrete, and hence reducing cost and environmental impact over the extended life-cycle of concrete. Results apply to concrete mixtures with normal- and high-strength concrete typically applied

for bridge decks exposed to various climate zones in northern America. Life-cycle results are presented per unit volume of concrete, but they can be extended to cover the entire bridge system, including substructure, safety barriers, joints, and other elements.² These greenhouse gas emission reductions are achieved through the following mechanisms:

- Lower material volume and structural demands due to reduced weight
- Decreased transportation energy for aggregate and concrete delivery
- Fewer repair cycles over the structure's lifespan

These attributes align directly with Federal Highway Administration initiatives such as Every Day Counts (EDC-7),³ which promote internally cured concrete and life-cycle-based sustainability practices.

Advancing Design and Practice

While LWC has been used successfully for decades, recent research supported by ESCSI is helping refine design approaches and expand adoption. Studies have shown that existing design provisions often underestimate LWC performance, particularly in areas such as modulus of elasticity and durability behavior.⁴ By developing improved predictive models and design recommendations, ESCSI is helping engineers accurately capture the benefits of lightweight aggregate and apply them with confidence in bridge applications.

Leading Toward the Future

As the industry shifts toward performance-based design and life-

cycle-focused decision-making, ESCSI continues to provide leadership at the intersection of innovation and implementation. LWC represents more than an incremental improvement. It offers a comprehensive solution, combining reduced structural demands, enhanced durability, extended service life, and lower environmental impact.

For bridge owners, designers, and contractors, the message is clear: concrete made with lightweight aggregates is stronger, performs better, lasts longer, and is more sustainable than conventional concrete. Through its ongoing work, ESCSI is ensuring that these benefits are realized across the next generation of concrete bridge infrastructure.

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Dr. Fariborz M. Tehrani is a professor at the Lyles College of Engineering at California State University, Fresno, founding scientific director of Pontis Research Group, and the director of the Expanded Shale, Clay and Slate Institute.

Cold Joints in Bridge Structures: From Design Practicality to Structural Behavior

by Dr. Brandon Li, Dr. Oguzhan Bayrak, and Doug Beer, Concrete Bridge Engineering Institute

As the concrete bridge industry continues to evolve, practitioners are increasingly called upon to make decisions that affect the structural performance, durability, constructability, and long-term costs of bridges. Research plays a critical role in informing those decisions, but its value is realized only when findings can be translated into practical applications. As a resource for bridge owners, engineers, contractors, and students, the Concrete Bridge Engineering Institute is committed to connecting the industry with reliable, evidence-based information and educational opportunities that advance the state of practice. In this article, we highlight findings from recent research^{1,2} on cold joints, and, more importantly, the practical lessons that they offer. The investigation, which was conducted at the University of Texas at Austin Ferguson Structural Engineering Laboratory, demonstrates how research can be applied to support informed decision-making and improved outcomes for concrete bridge projects.

Concrete placement is rarely as simple as casting an entire structure in one continuous operation. In bridge engineering, sequential concrete placement is often unavoidable in projects that involve staged construction, massive structural members, complex structural geometries, repair operations, or rehabilitation. **Figure 1** illustrates a common example with a cast-in-place bridge deck placed over a precast concrete panel, which creates an interface between an existing concrete member and a freshly cast component.³ This interface is commonly referred to as a cold joint.

Although cold joints are a familiar part of bridge construction, their behavior



Figure 1. After the deck has been thoroughly wetted and prepared, workers use a screed to level cast-in-place concrete on top of a precast concrete panel.³

can be complex and difficult to quantify. In many cases, the structural integrity of the member depends not only on the strength of the concrete on either side of the cold joint but also on the force transferred across it. Therefore, to design safe and efficient bridges, designers must understand the behavior and load-carrying capacity of cold joints, and the corresponding evaluation approaches.

Previous Research on Cold Joints

Decades of research have contributed to the current understanding of interface shear behavior. Early studies of composite concrete members led to the development of the shear-friction theory,⁴ which considers the resistance provided by friction, externally applied normal forces, and normal forces

generated by reinforcement crossing the interface. Later experimental studies⁵⁻⁷ further refined this concept by considering the contribution of chemical bonding and the role of interface preparation. Those efforts formed the foundation for many of the interface shear provisions used in current design specifications.⁸⁻¹²

Much of the research was on horizontal shear strength of the concrete-to-concrete interface of composite concrete beams.¹³⁻¹⁵ These studies focused on structural members with relatively uniform stress distributions and intentionally avoided regions that cannot be evaluated using sectional analysis. However, as discussed in the Concrete Bridge Technology article in the Fall 2020 issue of *ASPIRE*®, cold joints in actual bridge structures are not

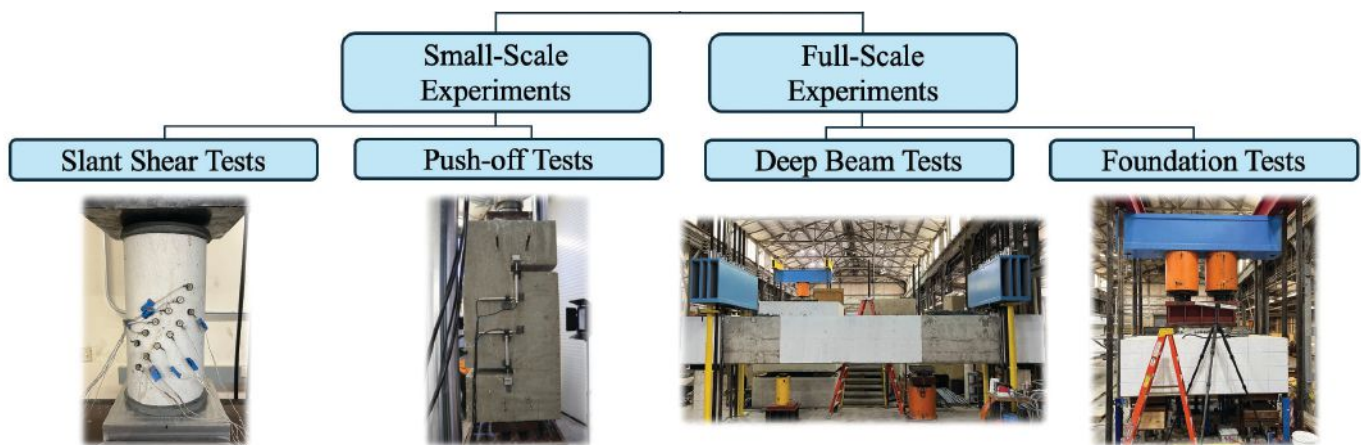


Figure 2. Research road map for Texas Department of Transportation Project 0-7117. Figure: Concrete Bridge Engineering Institute.

always located in such simple regions. Cold joints may occur near the supports, other regions where concentrated loads may be applied, the anchorage zones, or the connections. These locations, defined as the disturbed regions (D-regions), are characterized by the highly nonlinear and localized stress field, rendering the plane-sections-remain-plane assumption underlying sectional analysis invalid.

This distinction raises important questions for bridge engineers: How does a cold joint behave when it is intersected by localized compressive or tensile stress fields? How should the effective resisting area of the interface be defined when forces are not uniformly distributed? Are conventional interface shear equations applicable when the cold joint is part of a complicated load transfer system?

Recent Advancements

Texas Department of Transportation Project 0-7117¹ was developed to address the questions posed in the previous paragraph, and to advance the understanding of cold joints in D-regions of reinforced concrete members. The project combined a nationwide survey, small-scale component experiments, full-scale structural experiments, analytical modeling, and numerical simulation (Fig. 2). Rather than treating cold joints only as isolated material interfaces, the investigators examined how the cold joints interact with the overall load-transfer mechanism.

The first step was to understand standardized current practice. The

research team conducted a nationwide survey of representatives from departments of transportation about construction practices, interface treatment, reinforcement detailing, and design concerns related to cold joints. The survey responses indicate that although cold joints are common in bridge construction, guidance on design details for cold joints in D-regions is limited and agencies take a variety of approaches to construction when cold joints are encountered in D-regions.

The research then moved from practice-based observations to laboratory testing. Investigators conducted a series of slant shear tests and push-off tests to quantify the influence of key variables on cold-joint behavior. The results showed that interface roughness is one of the most influential parameters for cold-joint behavior. Roughened interfaces (Fig. 3 [left]) generally provided greater resistance than smooth interfaces (Fig. 3 [right]), a finding that confirms the importance of surface preparation in construction. From a practical construction perspective, the experimental observations and the wide variety of roughening methods identified in the survey raise several important questions that need to be investigated further: How should roughness amplitude be measured? How should interface condition be specified and standardized? What roughening procedures are appropriate for various interface geometries and structural demands? Future efforts to standardize roughening procedures and measure reliable roughness



Figure 3. Different levels of interface roughness: intentionally roughened interface on the left and form-finished interface on the right.¹

characteristics are expected to improve both design consistency and construction quality.

After the small-scale component experiments, the project expanded to full-scale structural experiments. Deep beams and drilled-shaft footing caps were tested to investigate how cold joints interact within members governed by nonlinear and nonuniform stress distribution. The full-scale tests demonstrated that the location and orientation of a cold joint relative to the internal stress field can substantially influence the structural response. When the cold joint intersects the critical load-transfer path, interface slip or opening may influence the redistribution of forces and the capacity of the components, potentially resulting in cold-joint

controlled failure before the strength of the remaining portion of the structure is fully mobilized (**Fig. 4**). These experiments provided direct evidence that cold joints should not be evaluated only as isolated interfaces. Instead, their behavior must be considered together with the load path and adjacent components within the structural system.

Based on the small-scale and full-scale experimental results, the researchers proposed a design approach that incorporates cold-joint behavior into the strut-and-tie method. The approach evaluates the interface shear demand along the portion of the cold joint engaged by the internal load path (**Fig. 5**). By combining interface shear calculations with the compressive and tensile stress fields represented in the strut-and-tie model, the proposed method provides a practical way to account for the interaction between cold joints and other structural components in D-regions.

For practicing engineers, this step is important. The strut-and-tie method is already widely used for the design of D-regions in concrete bridge structures, and integrating cold-joint evaluation into this framework allows engineers to analyze the interface based on the load-transfer mechanism, rather than calculating the interface capacity as a separate design check. This approach is particularly valuable for bridge components with construction joints that may be subjected to localized force transfer, such as precast concrete segments in balanced-cantilever segmental bridges and composite members with cast-in-place decks

placed over precast, prestressed concrete girders. The approach also provides an experimentally validated tool for evaluating structural components that contain unintentionally created cold joints. This tool can help engineers assess structural capacity without resorting to costly demolition and reconstruction. Beyond new construction, the framework supports the assessment of rehabilitation projects in which new concrete is added to existing structures, such as footing cap enlargements or structure widening (**Fig. 6**), where forces must be transferred across newly created interfaces. Even when the contribution of interface resistance is not explicitly considered in design, it is essential that designers understand the load-transfer mechanism across cold joints so they can evaluate the long-term safety, serviceability, and performance of repaired or expanded structures (see the “Future Provisions” sidebar).

Closing Remarks

Cold joints are an unavoidable part of bridge construction, repair, and rehabilitation. As bridge systems become more complex and as existing infrastructure continues to age, engineers will increasingly encounter situations where new and existing concrete must work together as one structural system. A better understanding of cold-joint behavior will help ensure that these interfaces are designed and constructed with the same level of confidence as the rest of the structure.

The research project presented herein reminds us that structural performance depends not only on individual materials or isolated design checks but also

Future Provisions

A standardized design approach and procedure were included in the final project report for Texas Department of Transportation Project 0-7117,¹ providing a practical template for design applications. The authors are currently developing and preparing the proposed recommendations for submission to the American Association of State Highway and Transportation Officials (AASHTO) Committee on Bridges and Structures for consideration in future provisions in the *AASHTO LRFD Bridge Design Specifications*. Meanwhile, the authors are actively applying the proposed approach to practical design examples to demonstrate its implementation in three-dimensional and multipanel strut-and-tie models.

on the systematic response and the relationship of various components. By studying these interfaces through experiments, design models, and advanced analysis, the bridge engineering community can continue to improve the safety, efficiency, and resilience of concrete bridge structures.

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Figure 4. Failure of deep beams with cold joints intersecting the load transfer path.¹



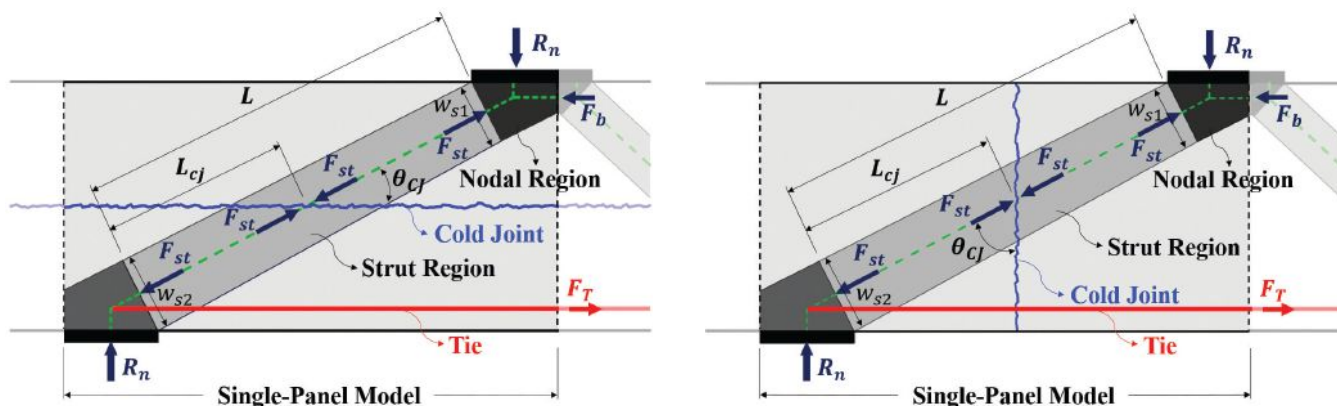


Figure 5. Strut-and-tie model (STM) with cold joints intersecting the load path in different orientations. On the left, the cold joint is not intersected by STM ties. On the right, the cold joint is intersected by STM ties.¹

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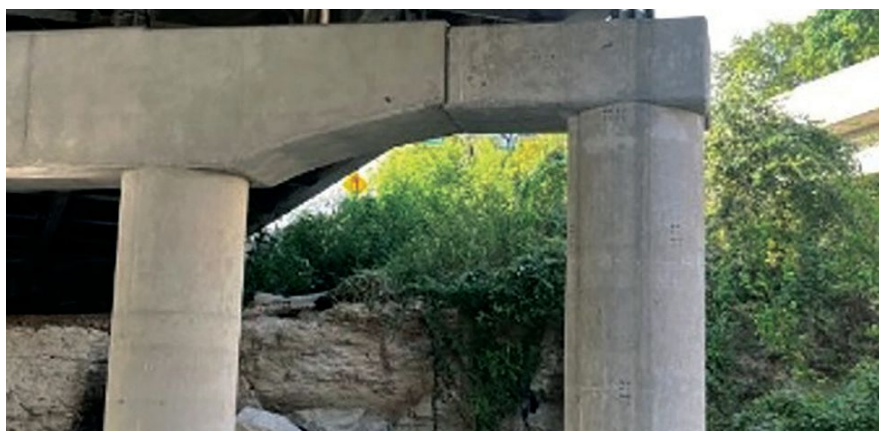


Figure 6. Example of bridge widening where forces are transferred across newly created interfaces. Photo: Concrete Bridge Engineering Institute.

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Longitudinal Reinforcement: Understanding the Demands Imposed on It and Its Capacity as Presented in the *AASHTO LRFD Bridge Design Specifications*

by Dr. Oguzhan Bayrak, University of Texas at Austin

Over the decades, bridge design specifications have evolved to reflect findings from research and development efforts. The first edition of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*, published in 1994,¹ based shear design on modified compression field theory (MCFT) which was a significant change in bridge design and load rating practices. Since then, the AASHTO LRFD specifications have been updated several times, and the 11th edition² will be published soon.

This article reviews the load effects as well as the resistance provided by the longitudinal reinforcement as presented in the AASHTO LRFD specifications. Article 5.7.3 is devoted to sectional design. To set the stage for our discussion, let us consider Commentary C5.7.3.3, in which the traditional shear design is explained:

The traditional approach to proportioning transverse reinforcement involves the determination of the required stirrup spacing at discrete sections along the member. The stirrups are then detailed such that this spacing is not exceeded over a length of the beam extending from the design section to the next design section out into the span. In such an approach, the shear demand and resistance provided is as shown in Figure C5.7.3.3-1.

In situations where a significant amount of the load is applied below the mid-depth of the member, such as inverted T-beam pier caps, and the section model is used to design for shear, it is more appropriate to use the traditional approach to the design of transverse reinforcement shown in Figure C5.7.3.3-1.

For typical cases where the applied load acts at or above the mid-depth of the member, it is practical to take the traditional approach as shown in Figure C5.7.3.3-1 or a more liberal yet conservative approach as shown in Figure C5.7.3.3-2 which has the effect of extending the required stirrup spacing for a distance of $0.5d_v \cot \theta$ toward the bearing.

where

- d_v = effective shear depth, as determined in Article 5.7.2.8 (in.)
- θ = angle of inclination of diagonal compressive stresses, as determined in Article 5.7.3.4 (degrees)

Figures C5.7.3.3-1 and C5.7.3.3-2 are reproduced herein as **Fig. 1** and **2**, respectively. They clearly demonstrate that by adjusting the detailing (for example, the number of stirrup legs) and the spacing of the stirrups, we can provide shear resistance that is consistent with the shear demand. The number of sections where we change the stirrup detailing can also be controlled by the changes in flexural element geometry. A designer or load rater must identify critical sections along the length of the component when evaluating spliced girder and segmental bridges. Points where the beam depth, web width, or flange thickness vary along the span are common for this type of construction.

Article 5.7.3.5 addresses the demands imposed on the longitudinal reinforcement by a variety of different load effects. Equation (5.7.3.5-1) shows the capacity of flexural tension reinforcement as well as the demands imposed on that reinforcement as follows:

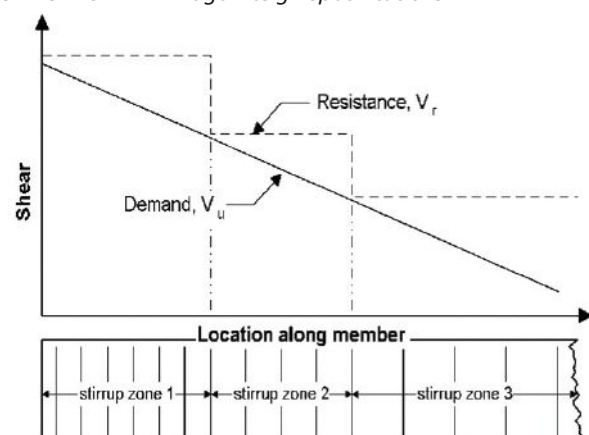
$$A_{ps}f_{ps} + A_s f_y \geq \frac{|M_u|}{d_v \phi_f} + 0.5 \frac{N_u}{\phi_c} + \left(\left| \frac{V_u}{\phi_v} - V_p \right| - 0.5V_s \right) \cot \theta$$

AASHTO LRFD Eq. (5.7.3.5-1)

where

- ϕ_f, ϕ_v, ϕ_c = factors taken from Article 5.5.4.2 as appropriate for moment, shear, and axial resistance
- V_s = shear resistance (kip) provided by transverse reinforcement at the section under investigation as given by AASHTO LRFD Eq. (5.7.3.3-4), except V_s shall not be taken as greater than V_u/ϕ_v in Eq. (5.7.3.5-1) and (5.7.3.5-2)

Figure 1. Traditional shear design. Source: Figure C5.7.3.3 -1 from the *AASHTO LRFD Bridge Design Specifications*.²



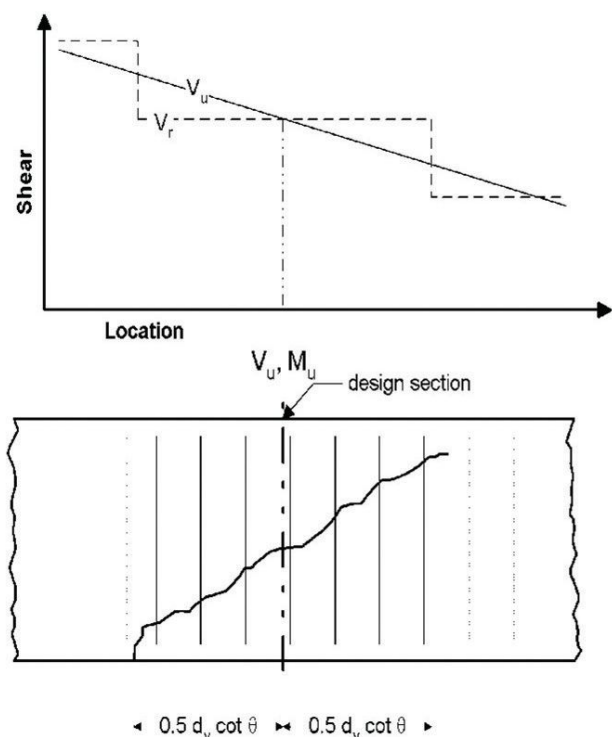


Figure 2. Simplified design section for loads applied at or above the middepth of the member. Source: Figure C5.7.3.3-2 from the AASHTO LRFD Bridge Design Specifications.²

It is important for the designer (or a load rater) to recognize that the left-hand side (that is, the capacity side) of the expression may be governed by the available development length of the reinforcing bars and prestressing strands. Additionally, the presence or absence of debonded strands will also influence the capacity. To better understand the demands imposed on the longitudinal reinforcement (the right-hand side) of Eq. (5.7.3.5-1), let us consider the commentary for Article 5.7.3.5:

Shear causes tension in the longitudinal reinforcement. For a given shear, this tension becomes larger as θ becomes smaller and as V_c becomes larger. The tension in the longitudinal reinforcement caused by the shear force can be visualized from a free-body diagram such as that shown in Figure C5.7.3.5-1. Taking moments about the compression force C in Figure C5.7.3.5-1, assuming that the aggregate interlock force on the crack, which contributes to V_c , has a negligible moment about the compression force C , and neglecting the small difference in location of V_u and V_p leads to the requirement for the tension force in the longitudinal reinforcement caused by shear.

Figure C5.7.3.5-1 is reproduced herein as **Fig. 3**. As explained in the commentary, moment, shear, and axial load will all influence the force in the tension tie. In cases in which torsional forces act on the element, those forces will also introduce additional demands and should be accounted for by using the provisions included in Article 5.7.3.6.3. Importantly, Article 5.7.3.5 states that “at each section the tensile capacity of the longitudinal reinforcement on the flexural tension side of the member shall be proportioned to satisfy” Eq. (5.7.3.5-1). The mandatory nature of the specification language is

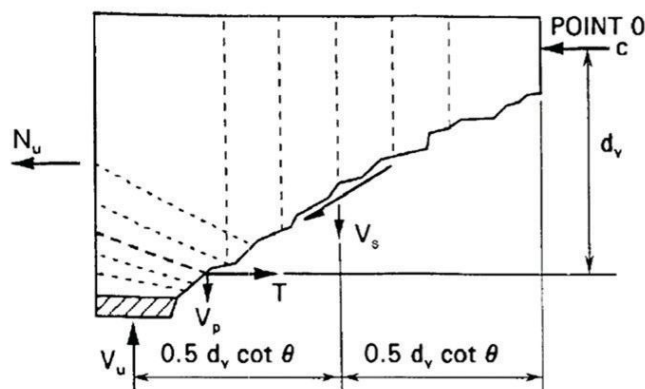


Figure 3. Forces assumed in resistance model caused by moment and shear. Source: Figure C5.7.3.5-1 from the AASHTO LRFD Bridge Design Specifications.²

significant. It is also important that Eq. (5.7.3.5-1) is *not* limited to the ends of simply supported beams. However, it is true that beam ends become particularly important. To that end, Article 5.7.3.5 states the following:

At the inside edge of the bearing area of simple end supports to the section of critical shear, the longitudinal reinforcement on the flexural tension side of the member shall satisfy:

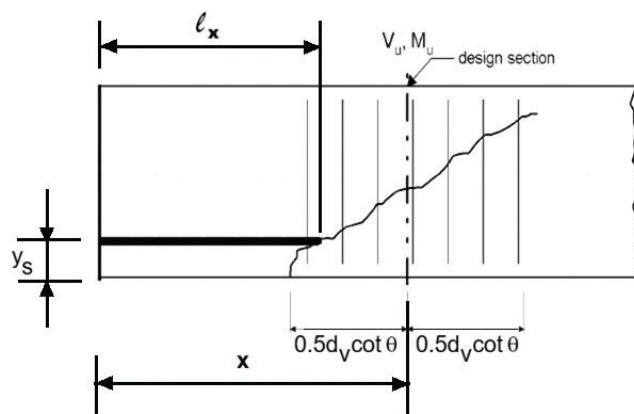
$$A_s f_y + A_{ps} f_{ps} \geq \left(\frac{V_u}{\phi_v} - 0.5 V_s - V_p \right) \cot \theta$$

[AASHTO LRFD Eq. (5.7.3.5-2)]

Equation (5.7.3.5-2) is a simplified version of Equation (5.7.3.5-1) where the bending moment and axial loads are equal to zero at the simply supported ends of a beam. In other words, Eq. (5.7.3.5-1) is a more general expression that covers all locations, including the beam ends with due recognition given to boundary conditions.

There are many design examples that illustrate the calculation of strength of and demands on the longitudinal steel at the face of support, but few examples demonstrate how to evaluate the strength of and demands on the longitudinal steel at an arbitrary section. **Figure 4** is a

Figure 4. Embedment of longitudinal steel for a design section located at a distance x from the end of a beam.



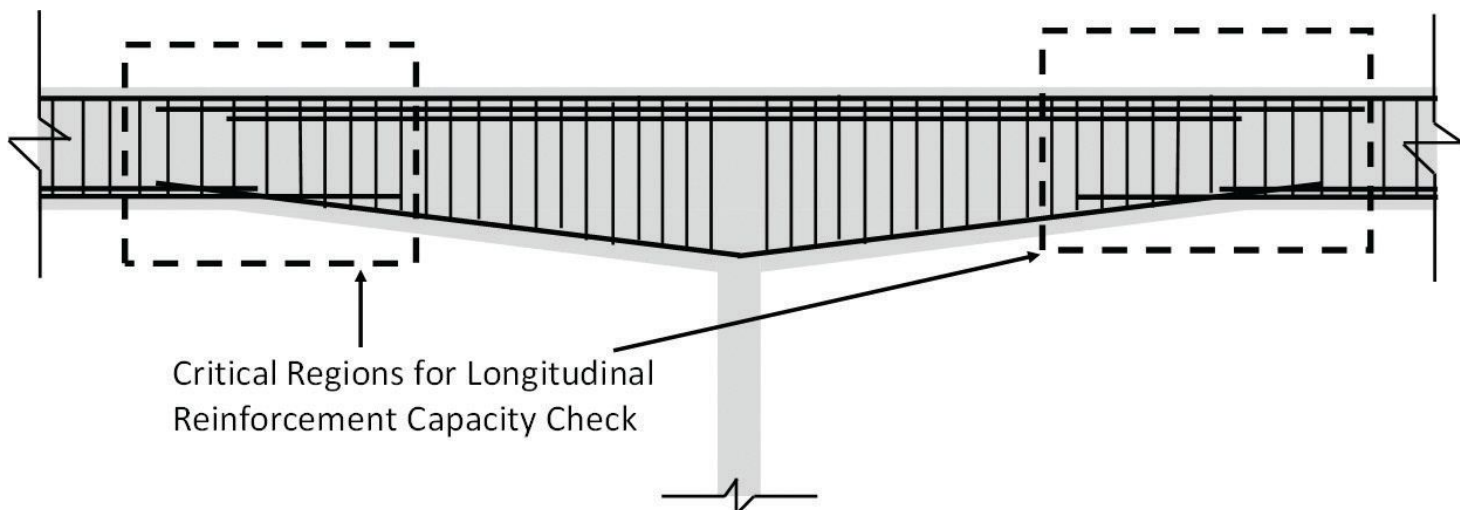


Figure 5. Possible locations where a longitudinal reinforcement check may control the capacity. Source: Based on the Jasper Bridge

version of Fig. C5.7.3.3-2 modified by the author. It shows that, for a given design section, the embedment length of the longitudinal steel must be evaluated at the point where the longitudinal steel crosses the crack. The embedment length is given as

$$\ell_x = x - (0.5d_v - y_s) \cot \theta$$

where

ℓ_x = embedment length of the longitudinal steel
 x = distance from the end of the beam to the design section
 y_s = distance from the bottom of the beam to the centroid of longitudinal steel

In this equation, d_v and θ are as previously defined.

If there is only one or two layers of steel, ℓ_x can be calculated based on the centroid of all the steel. However, if there are more than two layers of steel, it is advisable to find ℓ_x for each individual layer.

Figure 5 shows an example of details near a pier segment where flexural tension reinforcement terminations and the variable depth of the concrete section make it critical for a designer (or a load-rating engineer) to check the adequacy of longitudinal reinforcement with due consideration given to all load effects.^{3,4}

It is important to recognize that Eq. (5.7.3.5-1) and (5.7.3.5-2) are intended for use with concurrent forces. In other words, bending moment, shear force, and axial force envelope diagrams may produce maximum values for a section, but those maxima do not concurrently exist. The use of values from envelope diagrams would be conservative. However, this level of conservatism may be, and in most cases will be, unnecessarily high.

The Federal Highway Administration's *Modified Compression Field Theory (MCFT) for Shear Load Rating—Pretensioned*

Example (FHWA-RC-24-0005)⁴ provides a comprehensive discussion, with sample calculations, about how to use MCFT to calculate shear capacity and demonstrates how the longitudinal reinforcement check can be performed along the length of flexural members.

Concluding Remarks

This article emphasizes the importance of the longitudinal reinforcement adequacy check in accordance with the AASHTO LRFD specifications. Bending moments, shear forces, axial loads, and torsional moments all influence the demands imposed on flexural reinforcement. The adequacy of longitudinal reinforcement should be checked with due consideration given to reinforcing bar or prestressing strand development. For existing bridges where the interaction between bending moments and shears were not explicitly considered in the original designs, this check may present a challenge. Nevertheless, it is important to take this interaction into account in our new designs as well as load ratings.

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Upcoming Changes to the *AASHTO LRFD Bridge Design Specifications* and Other Items Presented by the Concrete Committee

by Dr. Oguzhan Bayrak, University of Texas at Austin

When the 2026 meeting of the American Association of State Highway and Transportation Officials' (AASHTO) Committee on Bridges and Structures (COBS) took place in June in Charlotte, N.C., COBS approved all 10 agenda items presented by the AASHTO Concrete Committee. This article provides an overview of those working agenda items (WAIs). I will discuss each agenda item in greater detail in future issues of *ASPIRE*.

Ballot Item 4 (WAI 232): AASHTO Guide Specifications for Structural Design with Ultra-High-Performance Concrete, Section 2. Material Conformance Guidance

This agenda item presents the second edition of the *AASHTO Guide Specifications for Structural Design with Ultra-High-Performance Concrete*. This edition will include material conformance specifications that cover requirements for qualification and acceptance testing of ultra-high-performance concrete (UHPC) materials.

The first edition of these guide specifications,¹ which was published in March 2024, is based on extensive research carried out by the Turner-Fairbank Highway Research Center of the Federal Highway Administration (FHWA), the Precast/Prestressed Concrete Institute (PCI), and other researchers from around the world. The material behavior of UHPC is directly related to its constituent materials, mixture proportions, mixing procedures, curing regimens, and placement methods. Therefore, during the development of the first edition, it became clear that, in addition to structural design specifications, the industry also needs UHPC material conformance specifications.

Ballot Item 5 (WAI 231b): Bridge Construction Specifications for Adjacent Members

Concrete box-beam bridges can be constructed by placing box beams side by side and connecting them with shear keys. Cracking of these shear keys has been a long-term problem for adjacent box-beam bridges. Research conducted as part of National Cooperative Highway Research Program (NCHRP) Project 12-95A² at the University of Cincinnati showed that cracking can be caused by temperature-induced deformations and movements. More specifically, solar energy causes the top slabs of the adjacent members to heat up and expand. The shear keys are usually filled with a grout material when the adjacent members are expanded, and cracking is caused by subsequent contraction when the members cool. Poor bond between the shear key material and the sides of the girders contributes to cracking. The results of NCHRP Project 12-95A showed that not filling the top 4 in. of the shear key with grout keeps the shear key material out of the zone of greatest thermal movement. Using a deeper shear key, roughening and prewetting the girder surfaces, and using a high-bond-strength shear key material improves the bond between the adjacent members and the shear keys, thereby mitigating cracking. An article in the Summer 2025 issue of *ASPIRE* summarized the NCHRP project. The specific changes to the *AASHTO LRFD Bridge Construction Specifications*³ addressed in this ballot item include requiring a Concrete Surface Profile of 4 (CSP4) or rougher, as defined by the International Concrete Repair Institute,⁴ for the vertical surfaces of the shear keys of adjacent members or an exposed aggregate surface, provided that this surface does not adversely affect the cover or create a situation where aggregates can be dislodged.

Ballot Item 6 (WAI 255): Guide Specification for Conducting Environmental Life-Cycle Assessments of New Bridges

This agenda item introduces a new guide specification to establish the minimum requirements for conducting environmental life-cycle assessments (LCAs) of new bridges. This guide specification is not intended for asset management decisions for in-service bridges. Instead, the intent is to provide uniform practices that bridge designers and owners can adopt to conduct environmental LCAs of new bridges. This guide specification provides criteria that bridge design teams can use to assess and compare the environmental LCA results associated with a reference design and a final design of an identified bridge, or the comparison of design options (preliminary designs) based on the same primary material selection being considered during the design of a bridge. The guide specification deals specifically with the environmental impacts of materials selected—including those due to construction, associated maintenance and replacement cycles, and end of life—over a target service life. The agenda item reflects collaborative efforts of the National Concrete Bridge Council and National Steel Bridge Alliance and benefited from the review comments provided by several AASHTO technical committees.

Ballot Item 7 (WAI 241): AASHTO Guide Specifications for GFRP-Reinforced Concrete, 2nd Edition

This ballot item corrects an error in the *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete*.⁵ Specifically, it corrects the upper-bound limit for the shear resistance of concrete reinforced with

glass-fiber-reinforced polymer (GFRP), making that limit consistent with Section 5 of the *AASHTO LRFD Bridge Design Specifications*.⁶ This change improves the design efficiency of GFRP-reinforced concrete; as a result, longer span lengths can be achieved before the conservative web-crushing limit is reached, without increasing the member size. The upper limit of V_n given by Eq. 2.7.2.5-1 in the GFRP guide specifications, is intended to ensure that the concrete in the web of the beam will not crush before the average maximum design strain limit (currently 0.004) of the transverse reinforcement across the critical shear crack plane is reached, as described in Commentary C2.7.2.5 of the GFRP guide specifications. With that stated, the maximum shear stress limit has been revised in the GFRP guide specifications to be 25% of the compressive strength of concrete rather than square root of the compressive strength of concrete, consistent with modified compression field theory based shear designs.

Ballot Item 8 (WAI 243): Grade 80 Reinforcing Bars

The American Concrete Institute removed Grade 75 reinforcing bars from the 2011 edition of *Building Code Requirements for Structural Concrete (ACI 318) and Commentary (ACI 318R)*.⁷ In the 2019 edition of ACI 318,⁸ Table 20.2.2.4(a), Nonprestressed Deformed Reinforcement, shows maximum values of yield strength f_y permitted for design calculations for ASTM A615, A706, A955, and A996 deformed bars. ASTM International moved Grade 75 reinforcing bar specifications to Annex A2 in the 2018 revision of ASTM A615, *Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement*.⁹ Grade 80 reinforcing bar specifications are listed in the main body of both ASTM A615 and A706, *Standard Specification for Deformed and Plain Low-Alloy Steel Bars for Concrete Reinforcement*.¹⁰ The change in these specifications seems to consolidate Grade 75 into Grade 80 for the design and material specifications. The small difference in the yield strength of 5 ksi allowed the two bar grades to be considered interchangeable from the standpoint of steel reinforcing bar manufacturing. Current design provisions in the AASHTO LRFD specifications stipulate a yield strength limit of 75 ksi

in various places, such as Articles 5.6.4.6, 5.7.2.7, 5.8.2.1 (just to name a few). As a result, Grade 80 reinforcing bars have been precluded from use in the design. To be consistent with the aforementioned design and material specifications moving away from specifying 75-ksi yield strength, revisions to change the specified minimum yield strength from 75 ksi to 80 ksi, where applicable, are made to the AASHTO LRFD specifications.

Ballot Item 9 (WAI 239): GFRP Axial Resistance

This ballot item makes revisions to Article 2.6.4.2 and its commentary in the *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete*, second edition. The intent of the revisions is to better align the guide specifications and recent research on the axial compressive resistance of GFRP. In short, the proposed revisions credit GFRP axial stiffness and strength in an amount similar to concrete that surrounds the GFRP bars.

Ballot Item 10 (WAI 240): GFRP Material Specifications

The *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete*, second edition, only refers to ASTM D7957, *Standard Specification for Solid Round Glass Fiber Reinforced Polymer Bars for Concrete Reinforcement*,¹¹ for material specifications and does not include ASTM D8505, *Standard Specification for Basalt and Glass Fiber Reinforced Polymer (FRP) Bars for Concrete Reinforcement*,¹² which covers higher-strength, straight GFRP bars. This ballot item, which has numerous subsections and additional references, addresses this oversight and aligns the guide specifications with the materials available in the marketplace.

Ballot Item 11 (WAI 242): Distribution Reinforcement

In the current AASHTO LRFD specifications, the provisions for distribution reinforcement are provided in Article 5.12.2.1 for slab bridges and Article 9.7.3.2 for deck slabs. Both articles provide equations for calculating the distribution reinforcement area based on a percentage of the primary reinforcement area. Use of the calculations is challenging when the primary reinforcement is high-strength

steel or prestressing steel and the distribution reinforcement is normal-strength mild steel. If the percentage of the area of the high-strength primary reinforcement is used to calculate the area of normal-strength steel, the result can be a very small amount of distribution reinforcement. For example, with partial-depth prestressed concrete deck panels, applying the AASHTO percentage in Article 9.7.3.2 to the prestressing steel area, using the primary prestressing strand area to size the mild distribution steel, produces a very small amount of distribution reinforcement. In such cases, excessive cracking in the panels perpendicular to the distribution steel could occur. If the same panel is designed using mild steel reinforcement in both directions, the amount of distribution reinforcement would be almost three times more.

The original provisions for distribution reinforcement were likely developed with members with equal strength reinforcement. In contrast, in this recommended revision to the AASHTO LRFD specifications, the amount of distribution reinforcement is based mostly on the strength of the section, not necessarily the area of reinforcement. Adjustment factors are proposed to account for different strengths of reinforcing steels. The concept of adjusting the percentage of primary reinforcement for differing steels is not new. In the AASHTO LRFD specifications, Eq. 5.12.2.1-2 uses this type of ratio; however, the equation is based on Grade 60 distribution reinforcement. The new provisions account for any combination of reinforcing steels using the same approach as Eq. 5.12.2.1-2. If the intent of these provisions is to have distribution reinforcement generally based on a percentage of strength, the proposed ratio should be applied to the percentage calculation and not be limited to the 50% and 67% values.

Ballot Item 12 (WAI 205): Definition of the Flexural Tension Side of Members

The changes detailed in this ballot item clarify the term A_{ps} that is currently used in various articles of Section 5 of the AASHTO LRFD specifications. A_{ps} is reserved for defining the area of prestressed reinforcement on the flexural tension side of a flexural member. This

agenda item offers clarifications for pretensioned members with harped strands or post-tensioned members with draped and straight tendons. It is aimed at achieving uniform assumptions by all designers. Importantly, the proposed clarifications were checked against an extensive database to ensure that the recommendations are conservative and accurate.

Ballot Item 13 (WAI 231a): Bridge Design Specifications for Adjacent Members

This ballot item, which has 15 subitems and many references, implements the design recommendations that were developed in NCHRP Project 12-95A.² It improves the AASHTO LRFD design provisions that apply to adjacent member use in superstructure construction and complements the construction provisions covered in Ballot Item 5, which was previously discussed.

Concluding Remarks

The AASHTO Concrete Committee has been very active in developing these ballot items, as well as a comparable number of working agenda items slated to be finished for the next cycle. I look forward to sharing the details of the 10 agenda items outlined in this article with our concrete bridge community in future issues of *ASPIRE*.

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Topics:

- History
- Table of Contents
- Chapter 1 – General Requirements
- Chapter 2 – Materials
- Chapter 3 – General Design and Testing
- Chapter 4 – Structural Analysis, Design, and Detailing

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Instructors



Richard Miller, PhD, PE, FPCI, is Professor Emeritus and former head of the Department of Civil and Architectural Engineering and Construction Management at the University of Cincinnati, where he taught for 36 years. Dr. Miller's research focuses on concrete materials and prestressed concrete bridges. He has been principal or co-principal investigator on seven projects for the National Cooperative Highway Research Program. Work performed by Dr. Miller and his colleagues has resulted in numerous changes to the *AASHTO LRFD Bridge Design Specifications*, including incorporation of high-strength reinforcing bar and provisions on debonded strands and continuous for live-load bridges. Dr. Miller has also completed numerous projects for the Ohio Department of Transportation and the Federal Highway Administration related to concrete bridges. He has served on and chaired several PCI councils and committees and currently serves on the PCI Board of Directors as the chair of the Technical Activities Council. He is a Fellow of PCI, and in 2024 he was named a PCI Titan of the Industry.



Clay Naito, PhD, PE, FPCI, is a professor of structural engineering at Lehigh University in Bethlehem, Pa., where he has taught for 22 years. Dr. Naito's research focuses on experimental and analytical evaluation of reinforced and prestressed concrete structures subjected to extreme events, including earthquakes, tsunamis, and intentional blasts. He has also conducted research studies for the Pennsylvania Department of Transportation, the Federal Highway Administration, and the Precast/Prestressed Concrete Institute on the performance of concrete bridge structures. Research topics include the performance of adjacent box-beam bridges, integration of electrically isolated tendons, use of self-consolidating concrete and ultra-high-performance concrete in bridges, and strand bond. He received the Distinguished Educator Award from PCI in 2015 and was elected Fellow of PCI in 2019.

Resources

PTI/ASBI Specification for Multistrand and Grouted Post-Tensioning

PTI Specification for Grouting of Post-Tensioned Structures, 4th Edition

FHWA Replaceable Grouted External Post-Tensioned Tendons

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