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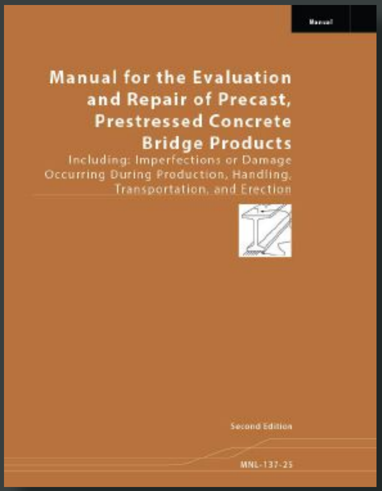

Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products, 2nd Edition (MNL 137-25)

Non-member price: $70
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Including: Imperfections or Damage Occurring During Production, Handling, Transportation, & Erection. This manual presents methods for evaluating and repairing damaged or nonconforming precast, prestressed concrete bridge products such as beams, piles, and deck panels. The various types of cracks, chips, voids, missing bars, and nonconformances that may occur from production through erection are defined and illustrated. Causes, prevention, engineering, and repair procedures are given for consideration for each type of damage or nonconformance. Chapters include "standard" repair procedures, which provide methods to repair damage and nonconformances, as well as patching and epoxy injection procedures. New to the second edition are the inclusion of piles among the product types and appendices on the evaluation of camber and sweep in beams. Precast concrete producers are encouraged to purchase copies to supply to their local agencies and specifiers.

This new manual is available from the PCI Bookstore.

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

PCI. Precast/Prestressed Concrete Institute

Photo: Nicholls Kovich Engineering.

Photo: Delaware Department of Transportation.

Photo: Delaware Department of Transportation.

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Photo: Nicholls Kovich Engineering.

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EDITORIAL

Photo: PCI.

A Journey

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

For most of my career, I have written and spoken about concrete, bridges, and the construction industry. Today, however, I want to talk about something far more personal—and far more important.

Two years ago, around Christmas 2024, I became sick with what seemed like a routine respiratory illness. Doctors treated it as a cold with inflammation in my lungs and prescribed antibiotics. The same thing happened again about a year later. After another urgent care visit and another round of medicine, I was told to follow up with my regular physician.

Then, suddenly, things changed.

While visiting family over the holidays, I reached a point where I could barely breathe. I had to lie down just to get air into my lungs. The day after Christmas, I went back to the doctor and was immediately sent to the emergency room. Doctors discovered that my lungs were nearly 60% filled with blood. What initially appeared to be recurring respiratory illness was actually something far more serious.

After several days of testing, specialists diagnosed me with a rare autoimmune disorder called granulomatosis with polyangiitis (GPA; formerly known as Wegener's disease). GPA can manifest at any age but is most commonly diagnosed in adults. Researchers have estimated that each year in the United States, approximately 13 cases are diagnosed for every 1 million people,[1] and it becomes an active illness in even fewer people. My case was likely triggered by occupational dust exposure over many years. As one doctor explained, I had been born with an immune disorder (although the condition is not hereditary) and repeated exposure to dust and airborne particles containing a mineral resulted in activation later in life.

I am telling this story not to gain sympathy, but to raise awareness. I hope my experience is shared in toolbox talks, company safety briefings, and leadership meetings because it offers lessons that every worker, supervisor, and company leader needs to hear.

Lesson 1: Wear Your PPE

My diagnosis forced me to reflect on my early years in construction. Like many young workers at the time, I rarely wore respiratory protection. Whether demolishing old materials, removing tile, or working in dusty environments, we simply "coughed it out" and kept going. Wearing masks was considered inconvenient, unnecessary, or even weak.

We were wrong.

Modern safety equipment exists for a reason. Respirators, masks, and other types of personal protective equipment (PPE) may feel uncomfortable in the moment, but they are far less uncomfortable than sitting in a hospital bed wondering whether your lungs, kidneys, or you will survive.

Lesson 2: Advocate for Your Care

By the time doctors identified my condition as GPA, it had already severely affected my lungs and kidneys. Treatment required high doses of steroids followed by advanced infusion treatments, commonly used with chemotherapy in cancer care. A challenge was that these additive treatments were extremely expensive and obtaining insurance approval was overly complicated.

Despite recommendations from multiple specialists, my insurance company initially denied approval of my treatment. I subsequently faced weeks of additional insurance denials, administrative reviews, and bureaucratic delays. During that time, my kidney function deteriorated to stage 3B failure. One physician bluntly warned me that waiting much longer for medical intervention could become life threatening.

Eventually, one pulmonologist took decisive action. He readmitted me to the hospital, restarted aggressive steroid treatment, and personally confronted the insurance company by challenging

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Workers set wide-flange prestressed concrete deck girders for the Arden Bridge no. 253 replacement project in Stevens County, Wash. Ultra-high-performance concrete joints were later field cast. Photo: Nicholls Kovich Engineering.

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the qualifications of those overriding the recommendations of multiple specialists and Mayo Clinic consultants. Within minutes, the treatment was finally approved.

That experience taught me a second critical lesson: Patients must actively advocate for their own healthcare. We cannot assume that the system will automatically work in our favor. Instead, we must ask questions, gather information, follow up constantly, challenge delays, and find doctors willing to advocate alongside us. Medicine continues to advance rapidly, and insurance systems and regulatory processes often struggle to keep pace. Without persistent advocacy from both patients and healthcare professionals, critical treatments can be delayed when time matters most.

Advocating for yourself includes making preparations and having conversations that many of us would like to avoid. This situation forced my family and I to have conversations we never expected to have so suddenly. We began organizing legal documents, updating medical directives, and discussing end-of-life wishes. As someone raised in a traditional Greek household, where the husband is often expected to simply "handle everything," I found those conversations to be uncomfortable. However, once we finally started talking openly, it became easier for our family to face the situation together.

Lesson 3: Stop "Toughing It Out"

Throughout my illness, one truth became increasingly clear: ignoring symptoms and "toughing it out" only make things worse. The construction industry, in particular, has long struggled with a culture that can reward silence over honesty. Too many workers pride themselves on pushing through pain, avoiding doctors, or hiding health concerns. We often treat vulnerability as weakness.

That mentality has to change.

The men and women in our industry are some of the hardest-working people in America. But real strength is not measured by how long someone ignores a problem. Real strength means paying attention to your health, speaking up when something feels wrong, and supporting coworkers who may be struggling physically or mentally.

Asking for help is okay. As one colleague told me, "You have to tell us what you need. We cannot take the medicine for you, but we can help with everything else." Once I asked for help, the support was immediate and generous.

As leaders, supervisors, and coworkers, we have a responsibility to look out for one another. Sometimes, the strongest thing you can say to a colleague is simply, "You don't look well. Go get checked out." Safety culture is not only about preventing falls or accidents but also creating an environment where people feel comfortable discussing their health before small problems become catastrophic ones.

Concluding Remarks

Many illnesses develop gradually over years before suddenly becoming severe. My engineering journey started outdoors. Although I later shifted to more of a desk job, my history of quiet exposures, untreated symptoms, and delayed action eventually had life-altering consequences.

If my experience encourages even one worker to wear proper protection, write and share an end-of-life plan, schedule a medical appointment, follow through on testing, or advocate for necessary care, then sharing this story will have been worthwhile.

Your health is not something to gamble with. Protect it early, protect it consistently, and never assume that you can simply "push through" forever.

Reference

1. Panupattanapong, S., D. L. Stwalley, A. J. White, M. A. Olsen, A. R. French, and M. E. Hartman. 2018. "Epidemiology and Outcomes of Granulomatosis with Polyangiitis in Pediatric and Working-Age Adult Populations in the United States: Analysis of a Large National Claims Database." *Arthritis & Rheumatology*. 70 (12): 2067–2076. https://doi.org/10.1002/art.40577.

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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.

June 28–July 2, 2026
AASHTO Committee on Bridges and Structures
Charlotte, N.C.

September 9–12, 2026
PCI Committee Days
San Antonio, Tex.

September 13–16, 2026
AREMA 2026 Annual Conference & Expo
Kansas City Convention Center
Kansas City, Mo.

September 30–October 2, 2026
PTI Committee Days
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
The 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 3–5, 2027
PCI Convention at The Precast Show
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.

September 21–25, 2027
PCI Committee Days
Loews O'Hare Chicago
Chicago, Ill.

Nicholls Kovich Engineering

A small firm fosters client relationships and focuses on real-world concrete bridge designs in the Pacific Northwest.

by Monica Schultes

Located in Spokane Valley, Wash., Nicholls Kovich Engineering is a small consulting firm that focuses on bridge design in the Pacific Northwest, with most of their work being performed in eastern Washington state and northern Idaho. The firm primarily works with cities and counties on local bridge infrastructure projects, but it also consults with precast concrete manufacturers and private entities. In addition to bridge design, Nicholls Kovich Engineering provides bridge inspection services, load ratings, and bridge rehabilitation designs. Although the firm occasionally provides subconsulting services on complex bridge projects, most of its projects involve small or medium-sized structures.

Client Relationships

The Nicholls Kovich design philosophy emphasizes efficient and streamlined bridge solutions, with a commitment to personalized, responsive service for each client. "Our biggest challenge is that we have to wear so many different hats as a small business," says Susan Kovich, owner and principal at Nicholls Kovich Engineering. "On any given day, we might conduct a design review, design a bridge girder, develop a project scope, prepare a cost estimate, or answer construction questions. It is a balancing act, but we have a talented team, and we face those challenges together."

The firm's work with local agencies tends to be 20% inspection, 60% design, and 20% rehabilitation and maintenance projects. Nicholls Kovich Engineering provides bridge inspection and evaluation services for several counties in Washington, including Adams County, Pend Oreille County, and Ferry County. The firm also consults with small municipal entities, often reviewing the jurisdiction's entire inventory of bridges. "That is valuable because we not only evaluate and monitor structures; we can also support agencies in securing funding for repair or replacement," says Kovich.

The firm understands that clients want to maximize their investments in all types of projects, and that all feasible options should therefore be explored. "When clients are looking for a 75-year minimum service life with limited funding, we really must look at bridge rehabilitation in different ways," says Kovich. "We look at replacement and rehabilitation options out in the field. The perspective of the entire structure is different from the underside of the bridge," she adds.

Background in Prestressed Concrete

Many of the structures designed and monitored by Nicholls Kovich Engineering are constructed with precast, prestressed concrete. Susan Kovich started her career as an engineer working for a PCI-certified producer. Thinking back on her earlier career, Kovich recalls, "I knew little about prestressed concrete and wondered why I didn't learn more in college!" Despite the initial learning curve, she embraced prestressed concrete and gravitated to the concrete bridge industry.

Kovich met Jerry Nicholls on a prestressed concrete girder project and decided to pivot to consulting work in 2002. Today, Nicholls Kovich Engineering maintains a connection with the precast concrete industry, with an estimated 70% of their consulting work involving precast concrete bridge replacements, inspections, and load ratings. In 2025, about 10% of the firm's work involved design and drafting services for local precast concrete producers.

The Hedlund Bridge over the Kettle River in Stevens County, Wash., has 95-in.-deep single-span, prestressed, post-tensioned concrete girders. The bridge spans 330 ft and is 28 ft wide. Maintaining the sag vertical curve and reusing the previous bridge's center pier were particularly challenging aspects of this project. All Photos: Nicholls Kovich Engineering.

This structure over the Little Pend Oreille River has a 45-degree skew. Nicholls Kovich Engineering provided structural design, hydraulic analysis, and construction management.

"We really understand the prestressed concrete side of things," says Kovich. "It provides valuable perspective to understand the fabrication process, identify potential production issues, maximize efficiencies, and move products efficiently from the casting bed to the jobsite. This inside knowledge leads to more practical designs."

Early in her career, Kovich grappled with the unique geometry of skewed, decked-girder bridges. The layout of the structure—skew, cross slope, and longitudinal slope—and the camber of the prestressed concrete girders could result in vertical misalignment of top-flange edges of adjacent decked girders in the field (also known as the "sawtooth" effect). To resolve this issue, the firm developed a simplified method to calculate bearing elevations of individual girders on a skewed bridge that takes into consideration camber, and cross and longitudinal slopes. In the Spring 2018 issue of *ASPIRE*®, Nicholls and Kovich described this method in an article titled "Practical Solution for Skewed Geometry on Decked-Girder Bridges," which is still relevant today.

Selected Projects

A favorite project in the firm's history was the conversion of an old Union Pacific Railroad bed into the Trail of the Coeur d'Alenes, a paved recreational trail that traverses 73 miles in scenic northern Idaho. The trail includes 36 pedestrian bridges, one of which is the Chatcolet Bridge, a 3100-ft-long elevated timber trestle structure that spans Chatcolet Lake at the south end of Lake Coeur d'Alene. The firm was tasked with designing the numerous approach spans of the rails-to-trails pathway, which was accomplished using precast concrete slabs at 8% grade to meet challenging safety and accessibility requirements.

Lessons learned in the field on projects like the Trail of the Coeur d'Alenes have bolstered the firm's expertise. "It's so valuable to see how these bridges are actually performing in service," says Kovich. "We can see where deterioration is occurring and anticipate challenges that maintenance crews will face. We bring those lessons into our design. For example, by inspecting expansion joints over time in the field, we can create details that will perform better over the long term."

In another notable project, Nicholls Kovich Engineering provided the design of the precast concrete bridge deck for the City of Spokane's Iron Bridge,

History of Nicholls Kovich Engineering

Jerry Nicholls started his career with the Washington Department of Transportation and then served as the bridge engineer for Spokane County. In 1992, he started his own firm, Nicholls Engineering, to provide bridge design services for local agencies in Washington state.

Susan Kovich joined the firm in 2002, and in 2014 the firm became Nicholls Kovich Engineering, with Kovich taking the reins as principal engineer. Nicholls retired in 2022, and Kovich is now principal and owner. The firm has retained the same four engineers since 2014, and those engineers have more than 85 years of combined experience in the bridge industry. "We are small but mighty," says Kovich. "It may sound cliché that we work well as a team, but it is true," she adds. "Our small business size works well. We get to decide which projects to pursue and focus on our passion for bridge design. I am honored to continue designing bridges, building on the knowledge and mentorship I received from Jerry Nicholls."

Nicholls Kovich Engineering designed a new precast concrete panel deck system supported by new precast concrete beams for the rehabilitation of the open-spandrel concrete arch Hatch Road Bridge in Spokane, Wash.

a 1911 steel latticed railroad truss that had been abandoned for nearly 40 years. Precast concrete hollow-core slabs spanning between the existing steel floor beams were selected to provide a durable, low-maintenance decking solution. The Iron Bridge is now a pedestrian bridge that crosses the Spokane River and connects to the popular 40-mile-long Centennial Trail.

Some projects have involved emergency situations. For example, in 2023, Grant County, Wash., administrators asked Nicholls Kovich Engineering to inspect large cracks in the abutment of Bridge 202, a 130-ft-long precast concrete I-girder bridge that was constructed in 1989. Kovich describes the findings: "One of the girders was suspended in the air, and we determined that severe scour had removed the spread footing support underneath." To preserve the superstructure, crews jacked up the existing girders and reconstructed the damaged abutment. The county expects the structure to last another 30 years. For more information on this project, see the Concrete Bridge Stewardship article on p. 31 in this issue of *ASPIRE*.

Nicholls Kovich Engineering worked with the Stevens County, Wash., Public Works Department on the Arden Bridge no. 253 replacement project in 2024. The firm provided civil and structural engineering design for the project from initial inspection of failed bearings on a steel bridge to construction of a new concrete bridge designed to a 45-degree skew. The replacement structure is a 109-ft-long single span that incorporates wide-flange prestressed concrete deck girders that are 39-in. deep. Ultra-high-performance concrete (UHPC) joints were field cast, delivering a durable solution capable of supporting traffic from agricultural and forestry activities.

The firm also led the design efforts for another Stevens County bridge replacement, the Hedlund Bridge over the Kettle River. The post-tensioned concrete bridge replaced a 60-year-old steel-truss bridge. This project featured spliced, precast post-tensioned concrete girders. One challenge involved maintaining the alignment along a sag vertical curve; its resolution required careful coordination of geometry and structural performance. A 200-ft-long post-tensioned concrete girder system was selected to retain the existing center pier. There were five total segments for the two-span bridge. Span 1 is 134-ft long and consists of two segments. Span 2 is 200-ft long and consists of three segments. The segments were spliced together with intentional negative offsets from the tangent span line to form the sag curve. Nicholls Kovich Engineering designed a solution with the spliced bridge girders on different vertical alignments to create the vertical sag curve. The bridge begins at a −10% grade and ends at a +2.7% grade. The low point is within the 200-ft-long span.

Nicholls Kovich Engineering served as the engineer of record for the Hatch Road Bridge project in the city of Spokane, Wash., which addressed deficiencies in an open-spandrel concrete arch structure built in 1919. The project replaced the aging metal deck with new precast concrete floor beams, 7-in.-thick, full-depth concrete deck panels with UHPC joints, and a modified concrete overlay. The new design improves safety, durability, and traffic flow of this major arterial, which connects the south side of Spokane to U.S. Route 195 and carries more than 8000 vehicles daily. The precast concrete deck elements were designed to minimize weight on the existing structure and support present-day design loads. The bridge was closed for three months and reopened ahead of schedule in July 2022.

The rehabilitated Hatch Road Bridge concrete arch structure is designed to accommodate higher volumes of traffic and improve long-term durability.

Looking Forward

Like many professionals, Kovich is cautiously optimistic about the future of artificial intelligence (AI) in the bridge design and construction industry. "As engineers, we need to use new tools and embrace innovation, but we also need to stay focused on the fundamentals of engineering," she says. "We need to pass that knowledge on to the next generation because those fundamentals are the basis for good decisions."

Kovich believes that the use of three-dimensional models to store quantities, materials, maintenance logs, and other information could transform the bridge construction industry. "I think the exciting part is being able to have a comprehensive record of a structure. We can review not only the design but also materials, methods, and the complete construction documents. It would be helpful to know what type of concrete was used and reference mixture proportions when problem-solving. That would be so beneficial," she says.

The future of Nicholls Kovich Engineering looks promising. "We are going to continue the work that that we have been doing. I am excited for the future and look forward to seeing local agencies awarded additional funding for bridges," Kovich says. The firm has a vested interest in maintaining the relationships with clients that they have nurtured for more than 25 years. Long-standing clients have ready access to the firm's engineers, and they do not plan to change the formula that has worked so well.

Near-Surface-Mounted Titanium Alloy Reinforcing Bars: A High-Performance Solution for Strengthening Concrete Bridges

by Dr. Christopher Higgins, Oregon State University

Thousands of reinforced concrete and prestressed concrete bridges built in the mid–20th century were designed to requirements that we now recognize have deficiencies. Many of these structures have inadequate shear reinforcement by current codes and poorly detailed flexural steel, and in many cases, designers did not properly account for the interactions of moment and shear on both the shear strength and flexural steel demands. When modern load-rating methods are applied and heavier trucks are considered, such bridges receive low ratings that seem to indicate a need for posting or replacement. However, in many cases, the deficiencies are localized, and much of the rest of the bridge structure is adequate. Rather than expend resources to replace or load post deficient bridges, a targeted strengthening method can sometimes be used to safely extend the life of the bridge. Many materials and techniques are available, and all have relative benefits and costs. Among the more promising recent developments are the use of titanium alloy bars (TiABs) as near-surface-mounted (NSM) reinforcement. Titanium alloy is well established in aerospace structural applications, and research has now demonstrated that NSM-TiABs are an effective and cost-competitive solution for both flexural and shear strengthening of existing concrete structures.

The development of TiABs for strengthening concrete structures began at Oregon State University (OSU) in 2012 through research sponsored by the Oregon Department of Transportation (ODOT).[1] Multiple research programs have since been completed to refine and validate the materials and methods through testing of full-scale bridge girders. That work has now moved from the laboratory to the field, with the first commercial field deployment completed in 2014. Since that time, 30 in-service structures across the United States and New Zealand have been strengthened with NSM-TiABs. The development of the American Association of State Highway and Transportation Officials' *Guide for Design and Construction of Near-Surface Mounted Titanium Alloy Bars for Strengthening Concrete Structures* (AASHTO NSMT-1)[2] and ASTM International's *Standard Specification for Titanium Alloy Bars for Near Surface Mounts in Civil Structures* (ASTM B1009-24)[3] now give engineers the tools to specify and use NSM-TiABs with confidence.

The NSM-TiAB Technique

NSM strengthening was originally developed for fiber-reinforced polymer materials and subsequently adapted for bonding TiABs into shallow saw-cut grooves in the concrete cover of members (**Fig. 1**). For flexural strengthening, grooves are cut longitudinally in the tension face of the girder over the region to be strengthened. Hammer-drilled holes are made at the ends of the grooves in which the hooked ends of the TiABs are anchored. For shear strengthening,

Figure 1. Installation of near-surface-mounted titanium alloy bars (TiABs): Saw-cut groove in cover concrete and hammer-drilled hook anchorage hole into core (left); bending of TiAB with 90-degree hooked ends (center); TiAB is installed and grooves filled with structural epoxy with exposed surfaces taped for a clean surface finish (right). Photo: C. Higgins, Oregon State University

grooves are cut vertically in the web faces and horizontally across the web soffit, and the TiABs are fabricated into U-shaped stirrups with 90-degree hooks that anchor into hammer-drilled holes at the ends of the web grooves beneath the deck soffit. In both applications, structural epoxy bonds the bar to the concrete substrate. The hooked ends provide mechanical anchorage so that the bond along the length is not the sole mechanism to develop the strength of the bar.

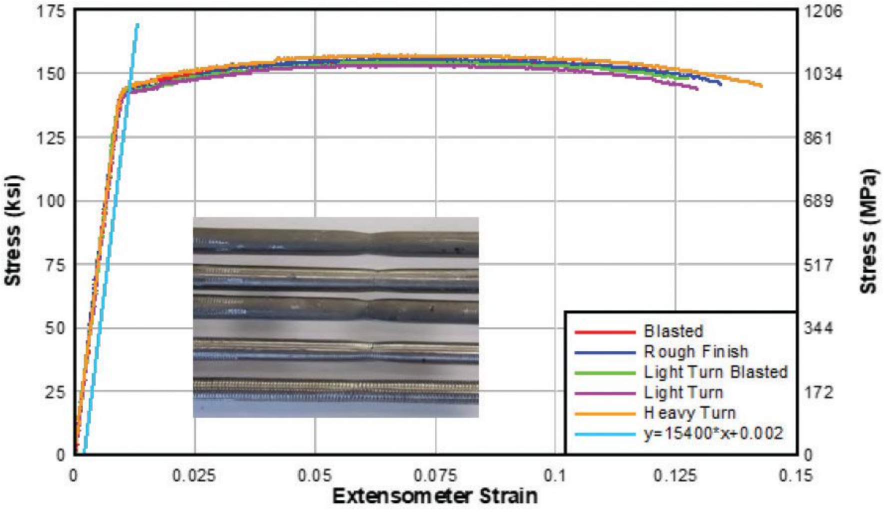
The hooked ends of the TiABs that provide the mechanical anchorage are a key feature of the system. Even if bond along the bar length is lost, the TiABs can maintain their load-carrying capacity through the hooks, producing a ductile and progressive response with visible warning signs before failure. Installation is straightforward and is typically done on in-service bridges while under traffic. Groove cutting, epoxy placement, bar installation, and curing can typically be completed during standard maintenance windows without shoring or lane closures.

Why Titanium Alloy?

Ti-6Al-4V, the titanium alloy bars described in ASTM B1009, is the same aerospace-grade alloy that aircraft manufacturers specify for structural airframe components. It is manufactured to strict aerospace quality standards with tightly controlled chemical composition and mechanical properties, resulting in very low variability in strength and elongation. That manufacturing precision, developed over decades, translates directly to consistent and predictable structural performance in bridge applications.

The properties of Ti-6Al-4V are well suited to NSM applications. The most commonly used yield strength is 130 ksi. This high yield strength means that designs can develop the required tensile force with fewer bars of smaller diameters than would be necessary when using other types of reinforcement. As a result, the number of grooves cut, the volume of epoxy required, and the installation labor on projects are all reduced. The unit weight of 276 lb/ft^3 is about half that of steel, which makes the bars easy to handle in the field. The material has an elastic modulus (15,500 ksi) in the same range as that of carbon-fiber-reinforced polymer (CFRP). Titanium is completely corrosion resistant in structural engineering environments, including marine and deicing-salt exposure. The minimum specified elongation of 10% provides high ductility, so the bars yield and sustain deformations under increasing load, which provides clear visual warning before failure. **Figure 2** shows typical uniaxial tension properties with the yield stress established based on 0.2% offset. These properties are for bars with surface deformations. The surface deformations enable bond between the TiABs and concrete through epoxy or other bonding material.

Figure 2. Example stress-strain responses for titanium alloy bars for a variety of different surface deformation patterns (inset) showing high strength with excellent ductility (>12%). Yield strength is determined with a 0.2% offset. Figure: C. Higgins, Oregon State University.



Flexural Strengthening

The OSU flexural-strengthening research program tested 11 full-scale tee and inverted tee girder specimens designed to replicate conventional mid–20th century reinforced concrete deck-girder construction materials and designs. Each specimen had intentional flexural steel anchorage deficiencies, with flexural bars extending only one-third of the required development length past a 45-degree diagonal crack. NSM-TiAB strengthening increased the load capacity by 31% to 44% and deformation by 85% to 174% compared with baseline control specimens that had not been strengthened.[4]

The failure mode also changed. The control specimens failed in brittle shear-tension at the cutoff locations of the flexural bars, with little warning. Strengthened specimens failed in ductile flexural modes with extensive distributed cracking, giving engineers and inspectors clear visual indication of distress well before capacity was reached.

Long-term durability of the NSM-TiAB flexural system was also investigated. A full-scale strengthened girder was simultaneously subjected to 1.6 million fatigue cycles and 200 cycles of freezing and thawing, representing more than 50 years of equivalent service, and then tested to failure. No meaningful change in stiffness or service-level response was observed during the exposure period. The postexposure failure load and ductility were within 1% of a companion specimen that had not been subjected to any fatigue or environmental exposure. The results confirm that the NSM-TiAB flexural strengthening system performs well under long-term in-service conditions.

Shear Strengthening

Shear strengthening with NSM-TiABs configured as either double-legged U-shaped or single-legged J-shaped stirrups was investigated for reinforced concrete bridge girders with insufficient transverse reinforcement. Seven full-scale tee and inverted tee specimens were strengthened with ¼-in.-diameter TiAB stirrups installed in vertical grooves

Figure 3. A crack with vertical offset at web soffit that was observed during a routine biennial inspection of Mosier Bridge no. 07626A in 2013 (left); flexural bar cutoff location superimposed over the interior girder of the bridge (right). The bridge was immediately shored pending analysis and strengthening. Photo: Oregon Department of Transportation.

on both web faces and hooks anchored beneath the deck soffit. Compared with the control specimens, the NSM-TiAB specimens demonstrated increases in shear strength ranging from 34% to 47%.[5]

One finding from this program is directly relevant to designers. Design provisions for some NSM materials limit the design strain to values below the actual material limit. TiAB stirrups with mechanical end anchorage develop their full yield strain. In addition, fractured TiABs were observed at the controlling diagonal cracks after failure. The experimental data support using the full yield strength of the TiABs in shear design calculations, which simplifies design and makes effective use of the material strength.

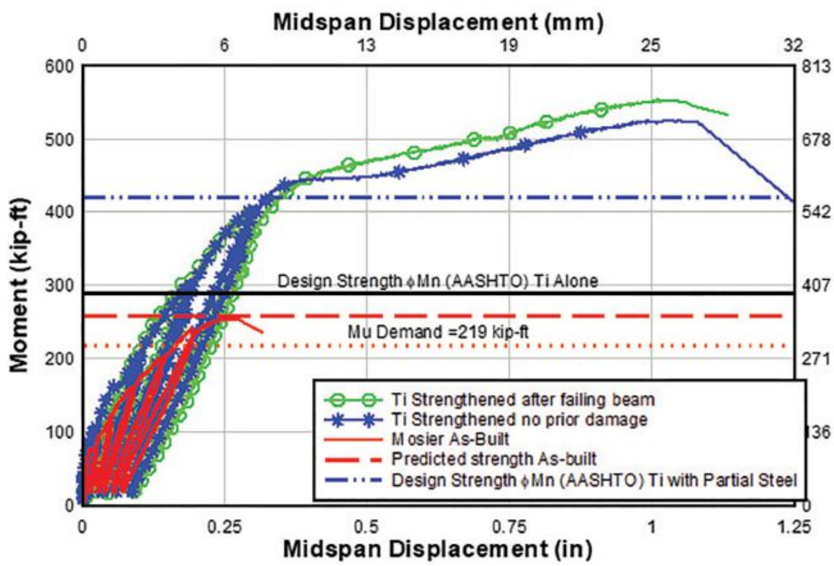


Figure 4. Moment versus midspan displacement for full-scale specimens representative of the Mosier Bridge interior girder. Near-surface-mounted titanium alloy reinforcing bar strengthening, even when applied to a previously failed specimen, more than doubled the load capacity and quadrupled the deformation as compared to the as-built control specimen. Figure: C. Higgins, Oregon State University.

Durability of the shear system was confirmed for specimens bonded with high-performance epoxy. After 2.4 million fatigue cycles combined with 120 cycles of freezing and thawing, representing more than 50 years of service in the Pacific Northwest, no reduction in strength or stiffness was observed. The *AASHTO LRFD Bridge Design Specifications*,[6] the American Concrete Institute's *Building Code Requirements for Structural Concrete and Commentary* (ACI 318),[7] and the *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures* (ACI 440.2R)[8] conservatively predict specimen strengths, with average experiment-to-predicted ratios of 1.15 to 1.17 for the durability-exposed specimens.

The Mosier Bridge: From the Laboratory to the Field

Mosier Bridge no. 07626A, a 1953 reinforced concrete deck girder (RCDG) overcrossing of Interstate 84 near Mosier, Ore., is a good example of how the OSU research applies to real bridges. This two-lane, three-span continuous RCDG bridge has four girder lines with girders spaced at 8 ft 8 in. and a 6.5-in.-thick deck. A routine biennial inspection in 2013 found a 0.03-in.-wide crack with a vertical offset in an interior girder, which is characteristic of flexural anchorage distress at a bar cutoff location (**Fig. 3**). The bridge was shored immediately. Analysis using the AASHTO LRFD specifications showed that the factored moment demand was 26.5% greater than the design strength from the AASHTO LRFD specifications, and NSM-TiABs were the most practical strengthening option given the geometry of the haunch at the critical section.

As a proof of concept, investigators used materials that reflected the vintage concrete and reinforcing steel to build and test full-scale specimens proportioned to match the Mosier Bridge interior girder.[1] The control specimen failed at a 64-kip point load placed at midspan with 0.26 in. of midspan displacement, and the observed cracking pattern matched what inspectors observed in the field, which validated the approach. Two NSM-TiAB-strengthened specimens reached 131- and 138-kip loadings with midspan displacements greater than 1.0 in., more than doubling the load capacity and quadrupling the deformation at the moment capacity (**Fig. 4**). Both specimens failed in a ductile flexural mode with extensive visual distress evident before ultimate was achieved (**Fig. 5**). Even the most

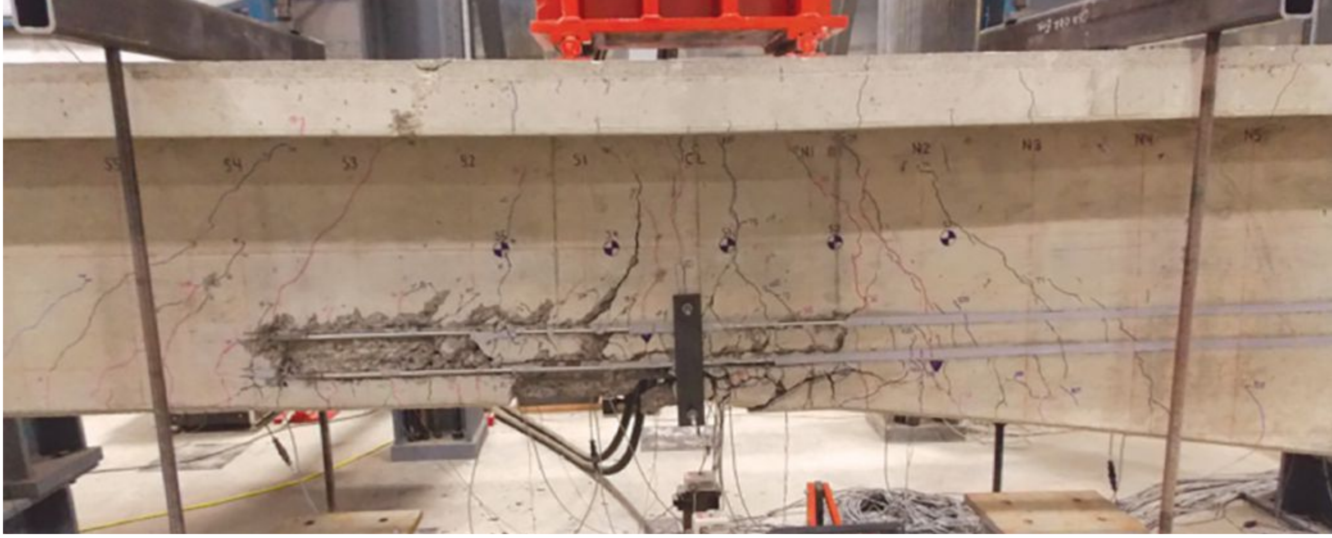


Figure 5. Condition of the replica of the Mosier bridge girder strengthened with near-surface-mounted titanium alloy reinforcing bars at failure in the Oregon State University laboratory. Large deformations and extensive distress give clear visual warning before ultimate failure. Photo: C. Higgins, Oregon State University.

conservative prediction—which used only the TiAB contribution for flexural strengthening and assumed that the original reinforcing steel provided no resistance at the critical section—exceeded the required factored demand. That lower-bound contribution provided by the TiABs acting alone is available to designers regardless of the residual condition of the in-place steel.

The bridge was strengthened in the field, and the installed cost was approximately 30% less than a competitively bid CFRP strengthening alternative. The savings came primarily from labor. The higher yield strength of titanium means fewer bars and fewer grooves to cut. Because labor accounts for a substantial part of the total cost of an NSM repair, a reduction in the number of bars, grooves, and structural epoxy can substantially reduce project costs. Subsequent field projects have consistently confirmed this cost differential. The structural benefits of a well-defined, highly ductile, inspectable, and corrosion-resistant material further support the use of NSM-TiABs for long-term bridge applications (**Fig. 6**).

In-Service Applications

Since the Mosier Bridge project, 29 other structures have been strengthened with NSM-TiABs in the United States and New Zealand. These projects span a wide range of structure types, repair conditions, and geographic locations, and illustrate how broadly the technique can be applied.

In Oregon, where the research originated, applications have included flexural strengthening of three coastal Highway 101 bridges (Devil's Lake, Schooner Creek, and Siltcoos River) where corrosion resistance was a primary requirement. Also in Oregon, the McKercher Bridge and SW 12th Avenue over Interstate 405 projects were emergency repairs completed under time constraints. The Abernethy Bridge, a major Willamette River crossing, required deck strengthening. Celilo Bridge required repair of hammerhead pier caps that were rated below requirements. Gold Beach Bridge is a historic landmark where 12,000 lb of TiABs were used in the rehabilitation. Oregon Route 217 and Camas Swale required crossbeam strengthening. Blowout Bridge was the first field application of the TiAB shear stirrup configuration developed in the OSU shear research program. Jefferson Bridge involved seismic reinforcement of columns. The Rogue River and Morrison bridges were historic preservation projects.

In Texas, NSM-TiABs have been used for pier cap repair on Interstate 10 over the San Jacinto River following tropical storm damage, cap and column repair on an Interstate 20 overpass, and slab bridge strengthening of U.S. Route 59 at Martin Creek and other structures. In New York, the technique was used to strengthen a salt shed at the Verrazano-Narrows Bridge complex, demonstrating that NSM-TiABs are applicable to any reinforced concrete member with accessible surfaces, not just bridge girders. In Pennsylvania, the Pennsylvania Deparatment of Transportation is using NSM-TiABs on State Road 286 over Humms Run for combined T-beam flexural and shear strengthening, and on the northbound and southbound Mill Hall bridges for pier cap negative-moment strengthening. In 2024, the first international application was completed in New Zealand with flexural strengthening of a beam bridge over Ngutuwera Stream.

Across these 30 projects, the structure types include T-girder bridges, box girders, slab bridges, pier caps, crossbeams, columns, and nonbridge concrete structures. The repair types cover flexural strengthening, shear strengthening, emergency repair, historic preservation, storm damage repair, and seismic retrofit. The range of geographic locations, from Pacific Coast marine environments to Gulf Coast hurricane zones and the Southern Hemisphere, confirms that the laboratory findings translate reliably to in-service conditions with more than a decade of successful in-service performance.

Design Tools for Practice

AASHTO NSMT-1,[2] which was published in 2020, provides design and construction recommendations for NSM-TiAB strengthening based on the AASHTO LRFD methodology. The guide covers both flexural and shear strengthening applications and includes guidance on groove geometry, bar sizing, development length, and hooked anchorage design. ASTM B1009 was first published in 2018 and updated in 2024. It covers chemical composition, mechanical property classes, dimensional tolerances, surface deformation requirements, and bend

Figure 6. Photo of the completed Mosier Bridge no. 07626A, a two-lane, three-span continuous reinforced concrete deck girder bridge that has four girder lines with girders spaced at 8 ft 8 in. The bridge was strengthened in the field, with near-surface-mounted titanium alloy bars proving to be the most practical strengthening option given the geometry of the haunch at the critical section. Photo: W. George.

test requirements for bar sizes no. 2 through no. 6. There are two strength classes, Class 120 and Class 130, with minimum yield strengths of 120 and 130 ksi, respectively. The term "class" rather than "grade" is used because grade is reserved for other ASTM titanium standards. Together, AASHTO NSMT-1 and ASTM B1009 provide engineers with a complete, code-referenced path from condition assessment and load-rating analysis through final design and material specification.

The design approach in AASHTO NSMT-1 is adapted from ACI 440.2R and calibrated to the AASHTO LRFD specifications. TiABs with hooked anchorages reach their full yield strength, so designers use standard yield strength–based calculations. For shear design, the full yield strength of the TiABs can be used without additional reduction factors when a specification-compliant epoxy is specified. The experimental data confirm that both the AASHTO LRFD and ACI methods produce conservative predictions for NSM-TiAB-strengthened members.

Conclusion

NSM-TiABs offer an effective, durable, and cost-competitive method for strengthening existing concrete structures deficient in flexure, shear, or both. The system increases load capacity and ductility; changes brittle failure modes to ductile ones; performs well under exposure to fatigue and freezing-and-thawing conditions that represent more than 50 years of service; and resists corrosion without ongoing maintenance. Installation is compatible with in-service bridge operations and conventional construction practices. Based on projects over the past decade, project costs are less than the costs for comparable CFRP strengthening. The technique is applicable to reinforced concrete and prestressed concrete bridge girders, pier caps, crossbeams, columns, decks, and nonbridge concrete structures. More than 12 years of field performance history and 30 successful projects across the United States and New Zealand demonstrate that the laboratory results apply to the conditions encountered in practice. With AASHTO NSMT-1 and ASTM B1009, engineers have the guidance they need to specify and design NSM-TiABs with confidence.

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Accelerated Bridge Construction of Two Bridges in Delaware

by Nicholas Dean, Delaware Department of Transportation

The Bridge 1-488N&S project involved the replacement of two bridges over Blackbird Creek along US 13 in northern Delaware. US 13 is a four-lane divided highway, classified as a minor arterial, with a 2050–projected average annual daily traffic (AADT) of 13,500 vehicles in each direction. Bridge 1-488S, which ranked first on Delaware's list of deficient bridges, was a 42-ft-long concrete arch bridge built in 1920. Bridge 1-488N was a 40-ft-long concrete rigid frame built in 1933. Bridge 1-488S was both structurally and hydraulically deficient. The concrete arch had significant spalling, cracks, and corrosion of exposed bar reinforcement. Bridges 1-488N&S both experienced scour issues related to the stream constriction created by their undersized span lengths. Due to their short timber piles, the bridges were considered scour critical. In addition to their structural and hydraulic deficiencies, both bridges were located at the bottom of a severe vertical sag curve. The roadway profile at the location of these bridges was substandard for stopping sight distance, leading to a high accident rate for vehicles trying to merge onto US 13.

Early in the design phase, several structure types were evaluated, including precast concrete buried arch structure, NEXT beams, and standard precast concrete economical fabrication girders with a cast-in-place (CIP) deck. Ultimately, because of the accelerated nature of construction, and the need to raise the profile of US 13 approximately 8 ft 6 in., the Delaware Department of Transportation (DelDOT) decided to replace the bridges with decked bulb-tee beams resting on stub abutments supported by prestressed concrete piles.

For years, DelDOT has demonstrated its commitment to implementing new and innovative bridge replacement techniques that decrease construction times, improve commuter and work zone safety, and minimize user costs. Given the high AADT and public exposure at this location, DelDOT opted to employ accelerated bridge construction (ABC) techniques to replace both bridges during a 45-day closure of US 13. The new bridges are composed entirely of precast concrete elements: decked

A precast concrete abutment segment is set into place over piles. Note the shiplap joint where two segments will be joined. All Photos: Delaware Department of Transportation.

profile

BRIDGES 1-488N&S / BLACKBIRD, DELAWARE

BRIDGE DESIGN ENGINEER: Delaware Department of Transportation Bridge Design, Dover, Del.

PRIME CONTRACTOR: Richard E. Pierson Construction Co. Inc., Pilesgrove, N.J.

CONCRETE SUPPLIERS: Cor-Tuf UHPC, Manassas, Va.; Heritage Concrete, Cheswold, Del.

PRECASTER: Precast Systems Inc., Allentown, N.J.—a PCI-certified producer

OTHER MATERIAL SUPPLIERS: Foamed glass aggregate: Aero Aggregates of North America LLC, Eddystone, Pa.

A 125-ft-long prestressed concrete decked bulb-tee beam with end diaphragms is set into place. Two cranes were needed to maneuver each 105-ton beam into its final position.

bulb-tee beams; stub abutments; prestressed concrete piles; approach, sleeper, and moment slabs; and T-walls. To accommodate the significantly accelerated construction timeline, the project team selected foamed glass aggregate (FGA) as a backfill. Additionally, the team chose ultra-high-performance concrete (UHPC) to transversely connect adjacent beams and approach slab segments.

Construction began with a full road closure of northbound US 13 on September 8, 2025. One week later, on September 15, southbound US 13 was fully closed. On October 20, 2025, after 44 days, northbound US 13 was opened to traffic. The next day, after 37 days of closure, the southbound direction was opened to traffic. By comparison, if standard construction techniques were used, bridge replacements of a similar size to Bridges 1-488N&S would require approximately a year to complete.

Precast Concrete Piles

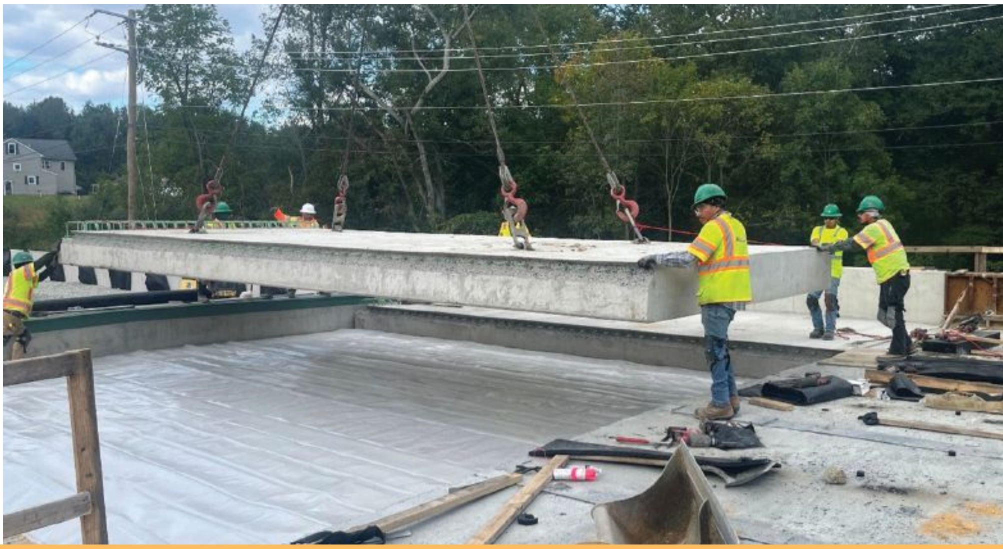
Soil conditions at the project site were well suited for the use of precast, prestressed concrete piles. A single row of eight 16 in. × 16 in., 50-ft-long piles was used to support the abutments on each side of the bridge. Because of the precast concrete abutments, tighter tolerances were required on the piles to ensure proper fit-up. To accommodate tolerances and achieve proper pile spacing, the contractor erected a robust steel template that could be disassembled and used on future projects. All 32 piles were driven to a depth between 27 and 31 ft into a dense sand layer and achieved a bearing capacity of more than 700 kip per pile. The pile driving took roughly one 10-hour shift per abutment to complete and was finished over the course of four days. Ensuring adequate capacity and proper placement of the precast, prestressed concrete piles was vital to the overall success of the project and allowed for the rapid erection of the precast concrete abutment sections.

Precast Concrete Abutments

Typical practice on previous DelDOT projects was to cast bridge abutments in place. That practice requires a large amount of time to place and tie reinforcing bars, build formwork, place the concrete, and allow the concrete to cure.

The substructures for Bridges 1-488N&S consist of two 55-ft-long, variable-height precast concrete stub abutments. The abutments were cast to accommodate the 2% cross slope of the roadway. Two separate sections were cast for each abutment with an overlapping (shiplap) joint in the center of the assembled abutment. The team decided to cast the abutment in smaller sections rather than one large piece to minimize the weight of the elements. Minimizing the weight and size of the abutment sections alleviated concerns about transportation and placement. Each precast concrete abutment section weighed roughly 40 tons.

A 1-ft 6-in.-thick precast concrete approach slab segment is placed. The approach slab segments were later connected with ultra-high-performance concrete closure pours.



DELAWARE DEPARTMENT OF TRANSPORTATION, OWNER

BRIDGE DESCRIPTION: Two 120-ft-long precast concrete decked bulb-tee beam bridges

STRUCTURAL COMPONENTS: Sixteen 66-in.-deep decked bulb-tee beams; thirty-two 16-in.-square precast, prestressed concrete piles; eight precast concrete abutment segments; four precast concrete sleeper slabs; sixteen precast concrete approach slab segments; fourteen precast concrete moment slab segments; one hundred fifty-four precast concrete T-wall segments

BRIDGE CONSTRUCTION COST: $12.1 million

Bridges 1-488N and 1-488S were replaced in 43 calendar days and 37 calendar days, respectively.

The precast concrete abutments were built with eight 30-in.-diameter full-depth voids to allow placement over the corresponding precast concrete piles. The voids, which were created by casting corrugated metal pipes into the abutments, were essential for proper connection between the piles and abutment sections. The 30-in.-diameter voids were designed to be oversized to account for the possibility that the piles would be improperly located or driven out of plumb. Extra care was exercised by the contractor to properly place the abutment sections over tops of the piles.

Each abutment was set in roughly an hour over four days. Once the abutment sections were set, the sections were doweled together and grouted to create a single composite unit. The 3-ft 4-in.-long dowels are no. 10 bars that are embedded 12 in. into the bottom abutment segment and 2 ft 3 in. into the top segment.

To connect the precast concrete piles with the abutments, a closure pour around the piles within the voids was cast. Because of the short duration of the construction project, it was important for the contractor to be able to load the abutments as quickly as possible. To facilitate this process, Type I 4500-psi concrete (DelDOT Class A) with a 2% high-early-strength admixture was used. DelDOT specifications for loading concrete substructures require that the concrete reach a minimum of 50% of the 28-day compressive strength. By using this mixture, the contractor was able to erect the decked bulb-tee beams four days after performing the closure pour. Successful installation of the precast concrete abutments set the tone for the installation of the precast concrete T-wall system and placement of the beams.

Precast Concrete T-Wall System

With the abutments in place, the contractor was able to begin placing the precast concrete T-wall segments and establish the limits of the proposed embankments. The decision to use the T-wall system was made to limit the impacts of the embankment side slopes on adjacent utilities and wetlands.

Efficient placement of these wall elements started with proper grading of the subbase and achieving the proper elevation for each wall. The front face panels of the T-wall rest on a precast concrete leveling pad, with the stem being supported on a layer of coarse aggregate. The walls are 10 ft

The precast concrete abutments were built with eight 30-in.-diameter full-depth voids, which were created by casting corrugated metal pipes. These voids were essential for proper connection between the precast concrete piles and abutment sections.

The faces of the precast concrete T-wall panels rest on a precast concrete leveling pad, with the stem being supported on a layer of foamed glass aggregate (FGA). FGA was selected for all backfill operations because of its low unit weight and other attributes.

high and are composed of 154 precast concrete elements stacked two units high. This project marks Delaware's first use of this wall type. Installation of the 540 ft length of proprietary wall took approximately eight days to complete.

Precast Concrete Decked Bulb-Tee Beams

The superstructures of Bridges 1-488N&S consist of eight 125-ft-long, 5-ft 6-in.-deep, 6-ft-wide prestressed, precast concrete decked bulb-tee beams. Using longer beams and significantly extending the spans of the bridges provided a cost-effective solution that helped limit the duration and quantity of the backfill work needed to raise the profile of the bridges approximately 8 ft. 6 in. Additionally, the use of the decked bulb-tee beams eliminated the need for drawn-out placement of the CIP deck.

Each of the precast concrete decked bulb-tee beams weighs roughly 105 tons. Given the size of these elements, placement of the beams required multiple steps using two cranes. Because a single crane was not large enough to place the beams in their final position, the contractor opted to build a temporary support system at midspan in the channel. On the south side of the channel, the contractor staged a 500-ton crane to pick the beams off the trailer and walk the beams out to the temporary support. The beams were landed on the support, and the crane's pick points were adjusted. A second crane, located on the north side of the channel, was attached to the beams, and together, the two cranes maneuvered each beam into its final position. This approach was efficient and well planned by the contractor, with each beam lift requiring skill and precision to ensure precise installation.

Moment slab segments were cast with the sleeper slabs to accommodate the transition from bridge barrier to guardrail.

Placement of the beams took two days per bridge. With the precast concrete beams in place, the UHPC connections could be placed, and the remaining precast concrete elements could be installed. (For more information on the use of UHPC on this project, see the Concrete Bridge Technology article on page 26.)

Precast Concrete Sleeper Slabs and Approach Slabs

The use of fully precast concrete sleeper slabs and approach slabs as part of the bridge replacement project marks a first for DelDOT. The use of sleeper slabs and approach slabs helps alleviate the issue of settlement behind the bridge and improves rideability. Additionally, the interface between these two elements is located where bridge movement is designed to occur. Moving the joints off the bridge helps extend the life of the bearings.

The sleeper slabs are 51 ft 6 in. wide and were cast as a single precast concrete element. In accordance with DelDOT's *Bridge Design Manual*,[1] moment slab segments were cast with the sleeper slabs to accommodate the transition from bridge barrier to guardrail. The sleeper slabs were the most complex element from a prefabrication standpoint. Each

AESTHETICS **COMMENTARY**

by Frederick Gottemoeller

In the last issue of *ASPIRE*® (Spring 2026), we considered the successful application of innovative precast concrete elements to the Sonoma–Marin Area Rail Transit (SMART) bridge in San Rafael, Calif., which created an attractive background bridge. In this issue, we are reflecting on the successful application of innovative precast concrete elements to accelerate the construction of US 13 over Blackbird Creek in Delaware, which has also created attractive background bridges.

The common feature of these bridges is that the visible precast concrete elements are themselves simple and go together in a visually simple and straightforward manner. (Of course, in practice, the requirements of ultra-high-performance concrete design are far from simple.)

The bulb-tee beams of the US 13 over Blackbird Creek bridges are slim and have an attractive horizontal shadow line, which makes them appear even thinner, and they rest neatly on the simple shapes of the precast concrete stub abutments. All the other precast concrete elements are hidden within the pavement or embankment.

Background bridges are not meant to attract attention, but they can still be features of urban or rural scenes. The use of attractive precast concrete elements offers a great aesthetic advantage, as it always results in the creation of an attractive bridge. Local residents and visitors alike are the beneficiaries of the care that went into the bridge design.

The 51-ft 6-in.-wide sleeper slabs were cast as a single precast concrete element and were connected using ultra-high-performance concrete closure joints.

approach slab consists of four 30-ft-long, 12-ft 10-in.-wide, 1-ft 6-in.-thick segments. Precast concrete barriers were cast with the exterior approach slab segments. Grading and placement of the precast concrete sleeper slabs and approach slab segments took place over the course of four days for each element type. The approach slab segments were then connected with UHPC.

Precast Concrete Moment Slabs

Because of the need to tighten the embankment limits, guardrail could not be used within the limits of the T-walls. Guardrail would have required much wider limits to accommodate the deflection zone needed behind the rail system. Each moment slab section is 24 ft 6 in. long and accommodates a precast concrete two-strand tube rail barrier system. In total, 14 moment slab segments were placed over three days.

Foamed Glass Aggregate

Given the large amount of backfill required to adequately raise the vertical profile of US 13, DelDOT needed to use a material that could safely and efficiently accommodate the expedited construction schedule. The work required to raise the profile approximately 8 ft 6 in. above the existing grade required roughly 17,000 yd^3 of backfill. Because the backfill work was a critical path for opening the roadways on schedule, DelDOT opted to use Foamed Glass Aggregate (FGA) to facilitate the work. FGA is produced from 100% recycled glass. Its permeability, high friction angle, and low unit weight made it an ideal backfill material for the Bridge 1-488N&S project. The free-draining characteristics of FGA mean that it can be placed in almost any weather condition, except in ponding water. The contractor was able to continue placing the material during rain events, which kept the project moving forward. The high friction angle and low unit weight improved stability of the slopes outside of the T-walls, reduced lateral earth pressure on the wall, and decreased the load on the existing soils. With typical backfill materials, settlement of both the existing soil and the proposed embankment are major concerns. However, because FGA is about 85% lighter than typical backfills with a dry unit weight of approximately 15 lb/ft^3, these concerns could be alleviated. Installation and geotextile wrapping of the FGA took place over 14 days and occurred concurrently with the installation of the bridge elements and UHPC.

Conclusion

The replacement of Bridge 1-488N and Bridge 1-488S in 43 calendar days and 37 calendar days, respectively, was deemed a monumental success, and DelDOT expects these new bridges to have a 100-year service life. In many ABC applications, it is common to trade a shorter construction duration for an increased replacement cost. However, given the complexities of normal construction in this location, the ABC replacement of Bridges 1-488N&S slightly lowered costs, while the road still reopened in a fraction of the time that would have been required for a conventional project.

The project provided valuable insight and experience and demonstrates that DelDOT's commitment to the use of ABC methods has helped stabilize and sometimes diminish costs as better details are developed, ABC practices become more mainstream, and contractors gain more experience. Although ABC techniques in Delaware can be further improved, the success of this project shows the merit of using such techniques, especially for high-volume roadways. Given the high-profile nature of this project, DelDOT closely monitored the progress of construction. Some of the major lessons learned from the replacement of Bridges 1-488N&S include the following:

- Proper detailing of precast concrete elements is essential. When errors in the construction of bridge elements occur, in-field changes to precast concrete components cannot be easily accommodated without cost and scheduling impacts.
- While Delaware is at the forefront with ABC techniques, DelDOT recognizes the benefits of seeking assistance from other states and federal agencies. While developing details and specifications during the design phase, the design team reached out to multiple states. This outreach allowed DelDOT to learn about previously implemented techniques and improve on them in this project.
- An open line of communication with the precast concrete producer and contractor is important. The fabricator and the contractor for Bridges 1-488N&S were able to provide valued input on what did and did not work well, as well as possible changes that could be incorporated into future projects.

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Demystifying Concrete Segmental Design

by Gregg A. Freeby, American Segmental Bridge Institute

The development of post-tensioned concrete and its application to concrete segmental bridges began in Europe after World War II to address war damage and steel shortages. The firms of Freyssinet (France) and Dyckerhoff & Widmann (Germany) were among the primary proponents of post-tensioned concrete segmental design. The technology rapidly spread and was introduced to the United States in the early 1970s. Meanwhile, as the technology evolved, so did the need for reference documents for designers of these unique structures.

Origins in the United States

The first concrete segmental bridge built in the United States was the JFK Memorial Causeway on Texas Park Road 22 in Corpus Christi, Tex. (**Fig. 1**). This bridge was constructed for $2.5 million using the precast concrete balanced-cantilever method and opened to traffic in 1973. Segments for this bridge were cast off site and joined using epoxy resin and post-tensioning tendons. (See the Summer 2021 issue of *ASPIRE*® for information on the long-term performance of this first-of-a-kind structure.)

The bridge was designed referencing the prior knowledge and experience gained in Europe, as well as research conducted by Dr. John E. "Jack" Breen and colleagues at the University of Texas at Austin. A summary report derived from this research, *Design Procedures for Long-Span Prestressed Concrete Bridges of Segmental Construction*, was funded by the Texas Highway Department (TxDOT) and published in 1969.[1]

Figure 1. JFK Memorial Causeway in Texas is recognized as the first precast, post-tensioned concrete segmental bridge built in the United States. Built in 1973, the structure connects Corpus Christi to North Padre Island. All Figures and Photos: American Segmental Bridge Institute.

Less than a year after the completion of the JFK Memorial Causeway, the Pine Valley Creek Bridge in San Diego County, Calif., opened to traffic in 1974. This structure consists of twin bridges constructed using the balanced-cantilever method, with constant-depth, cast-in-place segments. At that time, creep and shrinkage were not well understood. To accommodate creep and shrinkage, constructors used a system of hydraulic jacks to induce a bending moment into the segmental box girder before placement of the closure pours to create a stress condition that emulated a comparable structure built on falsework. This strategy proved highly successful—the structure has been in service for more than 50 years with no perceivable additional deflection. See **Fig. 2** for a contemporary photo of the bridge.

The American Segmental Bridge Institute's *Design Manual for Concrete Segmental Bridges* is a free downloadable resource.

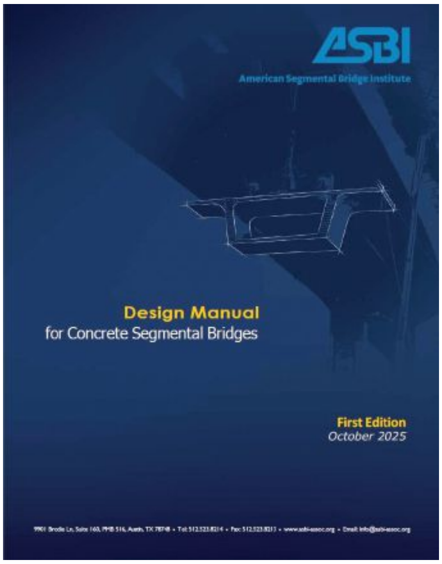

Figure 2. Pine Valley Creek Bridge in Pine Valley, Calif., is the first long-span concrete segmental box-girder bridge in the United States built using the cast-in-place balanced-cantilever method.

With these two projects completed within a year of each other, the concrete segmental method of construction was established in the United States. Nearly 500 additional concrete segmental bridges carrying vehicular traffic—along with countless others supporting light rail, people movers, heavy rail, and other transportation modes—have since been erected in the United States. Among the most notable examples is a concrete segmental bridge supporting a skyscraper for the Manhattan West project in New York City, a project featured in the Winter 2015 issue of *ASPIRE*.

Guidance on the design and construction of concrete segmental bridges in the United States was initially very limited. In 1978, the Post-Tensioning Institute (PTI) and the Precast/Prestressed Concrete Institute (PCI) copublished two documents, the *Precast Segmental Box Girder Bridge Manual*[2] and the *Post-Tensioned Box Girder Bridge Manual*,[3] which provided guidance on the state of the practice at the time. In 1982, John Wiley and Sons published *Construction and Design of Prestressed Concrete Segmental Bridges*[4] by Walter Podolny and Jean Muller. This book summarized many of the existing, and planned, concrete segmental structures at the time, and provided readers with insight into their designs. It is much revered to this day for the historical information it contains. All three of these references are now out of print.

In 2016, the Federal Highway Administration (FHWA) published the *Post-Tensioned Box Girder Design Manual*.[5] This manual provides information on the analysis and design of cast-in-place concrete box-girder bridges. However, it does not specifically address concrete segmental bridges and is only current through the sixth edition of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*,[6] published in 2012.

Current Practice

One current reference document for concrete segmental design is Chapter 14 of the fourth edition of PCI's *Bridge Design Manual*, which was published in 2023.[7] The chapter is entirely dedicated to precast concrete segmental bridges and includes example calculations. The manual is available as a free download from the PCI website (https://doi.org/10.15554/MNL-133-23).

In 2025, the American Segmental Bridge Institute (ASBI) published the *Design Manual for Concrete Segmental Bridges*.[8] This manual, which is available as a free download from the ASBI website (https://asbi-assoc.org/resources), provides updated information on the current state of the art in concrete segmental bridge technology and design. It addresses both precast and cast-in-place concrete segmental bridges and therefore supplements the information found in PCI's *Bridge Design Manual*. Both the PCI and ASBI manuals are current through the ninth edition of the *AASHTO LRFD Bridge Design Specifications*.[9]

The ASBI manual is organized to follow the chronological sequence from concept development through final design and detailing. Additional resources are included in the appendices, including references, sample plans, and a design example for a cast-in-place balanced-cantilever superstructure. The first chapter presents examples of completed concrete segmental bridge projects, highlighting a wide range of solutions and the adaptability of concrete segmental bridges to project-specific needs and environments. Intermediate chapters address the various stages of design development for concrete segmental bridges. The concluding chapter covers sustainability, durability, and serviceability, and introduces emerging technologies related to both design and materials.

The intended audience for the ASBI manual includes owners, engineers, and contractors with little or no experience in concrete segmental bridges. This manual also serves as a compendium of design practices for the two most common methods of constructing concrete segmental bridges—span-by-span and balanced-cantilever structures.

Throughout their history, concrete segmental bridges have undergone continual improvement. Early design assumptions related to prestress losses, creep and shrinkage of concrete, shear design, and geometry control have been significantly refined. This is not to suggest that older designs are deficient, but rather that modern designs are more precise.

Advancements in software have also enhanced designers' ability to efficiently analyze time-dependent effects, which are integral to concrete segmental bridge behavior. Preliminary design requires analysis tools capable of modeling erection sequences as well as creep, shrinkage, and post-tensioning relaxation. While software has streamlined calculations, it remains essential for designers to understand the underlying mechanics. For this reason, the ASBI *Design Manual for Concrete Segmental Bridges* includes a design example of a three-span, cast-in-place balanced-cantilever bridge constructed using the form traveler method.

Conclusion

While no single resource can make someone an expert, the new ASBI *Design Manual for Concrete Segmental Bridges* offers a strong foundation in concrete segmental methods and can serve as a gateway to a deep understanding of concrete segmental bridge design and construction.

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A Crack Is Not a Crack: Thermal Cracking

by Dr. Oguzhan Bayrak, University of Texas at Austin

This article, which is the eighth article in this series, is focused on thermal cracking. To set the stage for our discussion, let us cover the basics first. Hydration of cement is an exothermic chemical reaction. Understanding how heat is generated and dissipated during the curing of concrete is essential to improve our ability to design and maintain durable concrete bridges.

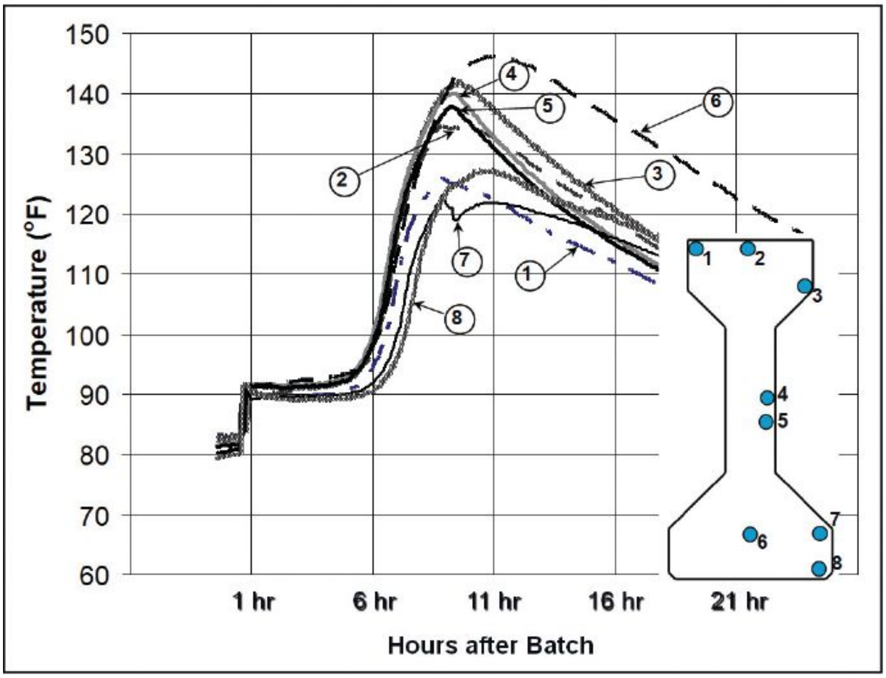
While this article is not solely directed toward exploring issues associated with mass concrete placement, let us start our discussion with a few values that relate to mass concrete. For most typical concrete mixtures, component dimensions larger than 36 to 48 in. would be classified as mass concrete. Project specifications and owners may stipulate requirements for the maximum temperature of the concrete core during curing, and the maximum temperature differential between the core of the concrete and its exposed surface in an effort to control and eliminate early-age cracking.

The maximum temperature of the concrete core used in mass concrete placement in Texas is typically 160°F.[1] The Texas Department of Transportation (TxDOT) specifies this limit to control thermal issues and delayed ettringite formation (DEF). DEF is an expansive mechanism that causes cracking similar to that caused by alkali silica reaction (ASR) (that is, map cracking). The difference between them is that ASR is associated with reactive aggregates, high-alkali pore solution, and presence of moisture in concrete while DEF is caused by aluminates in the cement. More specifically, the presence of excessive sulfates and alkalis within the cement, clinker, or aggregates may create a condition in which the sulfate is gradually released and bonds with aluminates. Excessive heat (temperatures in excess of ~160°F) creates a condition in which DEF potential is amplified. Conversely, controlling heat helps us mitigate/minimize the potential for DEF. With the description of ASR and DEF in place, let us now proceed to understanding the expansions associated with ASR and DEF. The maximum expansion potential and cracking severity associated with DEF are typically higher than those associated with ASR. The maximum temperature limit used in other states varies and can be as high as 180°F in some project specifications. Variations in the specified maximum temperature limits are rooted in the potential for DEF, and the presence or absence of ASR-reactive aggregates in the location under consideration. TxDOT specifies a maximum temperature differential between the core of the concrete placement and the exposed concrete surface of 35°F to minimize early-age thermal cracking. I will use the threshold values of 160°F and 35°F for maximum temperature and temperature differential, respectively, in the following discussion.

Precast Concrete Products

During his master's research at the University of Texas (UT), Tuchscherer[2] investigated the root causes of top flange cracking in AASHTO Type IV girders. While his research covered many topics and potential root causes, I will focus on one aspect of his work relevant to our discussion. In **Fig. 1**, the eight locations marked with blue dots within the cross section signify the locations where thermocouples were installed. As can be observed in the figure, the ambient

Figure 1. This plot shows the early-age temperature variation across a section that was part of an investigation at the University of Texas at Austin, into the root causes of top-flange cracking in AASHTO Type IV girders.[2]



conditions within the UT laboratory were such that the eight thermocouples initially measured temperatures ranging between 80°F and 84°F. The temperature of the fresh concrete mixture was around 90°F, and the temperature rose to a peak of 147°F around 11 hours after concrete was batched. This temperature is less than 160°F; therefore, DEF-related damage should not be an issue for this beam. The maximum temperature differential between the hottest and coolest location within the section is about 20°F to 25°F, which is less than the maximum temperature differential threshold value of 35°F used by TxDOT. The temperature-time history shown in Fig. 1 does not point to any temperature-related concerns. For all the test specimens in this study, we did not observe maximum temperatures or a temperature differential that would be a cause for concern.

As an aside, and perhaps an important one, we also tracked the mechanical properties of cylinders that were cured in a temperature regimen that matched the reference points shown in Fig. 1. Because cracking occurs when the tensile strength of concrete is reached, we were interested in seeing whether strengths differed between hotter and cooler locations. In short, the differences we saw in mechanical properties of concrete in hot and cool locations were comparable or less than the accepted strength variability values in the relevant ASTM specifications. While this observation is true for the AASHTO Type IV beams and for the concrete used in this study, it should not be extrapolated to all concrete components, geometries, and mixtures.

To improve our understanding of key factors at play, let us work on a few thought exercises. If the ambient conditions were different and if the constituent materials were warmer than the ones used in our study, the overall time history of the temperatures measured in eight locations would be different. For example, if the fresh concrete temperature were 10°F to 20°F warmer, we would expect to see the maximum temperature reached at each point to be higher, although the increases might not be exactly proportional for all points. This increase in fresh concrete temperature may have an impact on the temperature gradient. If warmer conditions were present and the maximum temperature limit of the project specifications were challenged, we would have to consider cooling down the fresh concrete mixture with ice or liquid nitrogen injection. If we could bring down the placement temperature of the fresh concrete mixture, that would go a long way toward controlling the temperatures experienced by the structural component.

The properties of the concrete mixture also play a key role in the temperature-time history. In our study, Type III cement was used in the concrete, as is typical in some precast concrete applications. The high early strength needed in precast, prestressed concrete applications is coupled with the elevated heats of hydration in the early stages. The use of supplementary cementitious materials (SCMs) such as fly ash and slag can offer an overall temperature-controlling benefit but will also slow strength gain. This delay in strength gain may present a significant challenge for precast, prestressed concrete applications. In cast-in-place concrete applications, particularly in substructure construction, the use of SCMs poses a lesser challenge because early strength gain is typically not as important as it would be in precast concrete production.

Bridge owners and the literature on mass concrete placement typically stay away from a strict definition for mass concrete. Technical reports[3–5] and project specifications typically define mass concrete as a large volume of concrete placement that will require management of the heat of hydration generated during curing. This broad definition is intended to recognize that component dimensions less than 36 to 48 in. may also need temperature management in cases where the proportion of cementitious material content is large. To aid in this technical challenge, TxDOT funded research at UT.[5,6] That research effort resulted in a publicly available software package (ConcreteWorks), which allows users to make multiple adjustments to manage heat generation and dissipation to minimize the risk of thermal cracking.

Case Studies

Large substructure elements such as drilled shafts, footings, and piers may fall into a category of mass concrete, which will require heat of hydration management. To better understand the underlying mechanics, let us consider a few case studies.

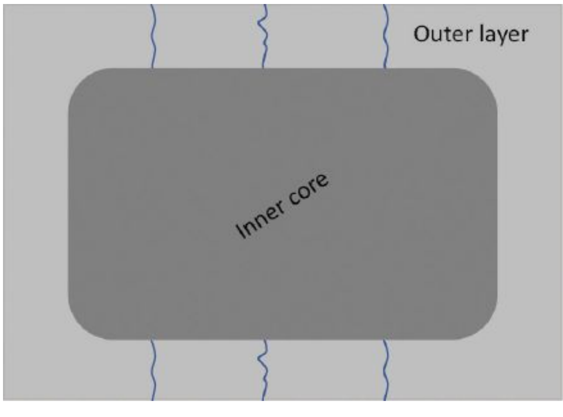
Large Pier

Let us consider a scenario in which the formwork for a hammerhead pier (**Fig. 2**) is removed early or a similar situation where the ambient temperature suddenly drops after the concrete for such a pier is placed. In this scenario, as the temperature of the concrete in the inner core continues to rise (or is maintained), the outer surface may suddenly cool down. The restraint provided by the inner core on the outer surface will result in the development of tensile stresses in the outer layer. When the tensile stresses in concrete reach the tensile capacity of the concrete, cracking in the outer layer will occur. The severity of this cracking will be a function of many factors, including the temperature gradient between the outer surface and inner core of the pier, the overall heat of hydration generated by the concrete mixture (which may or may not include SCMs), and environmental exposure conditions such as rapidly dropping or increasing ambient temperatures, just to name a few. In the event that such cracking occurs, the structural and durability implications of such cracks need to be determined. Remedial measures, crack repairs, installation of protective membranes, and other interventions may or may not be necessary.

Large Footing

Next, let us consider a large concrete footing supported on drilled shafts (**Fig. 3**). Suppose that a rain event rapidly cools the top side of the footing soon after the concrete is placed and the initial set occurs. With this cooling effect, let us assume that the temperature gradient becomes severe to the point where the

Figure 2. Schematic cross section of a column section in a hammerhead pier. Figure: University of Texas at Austin.



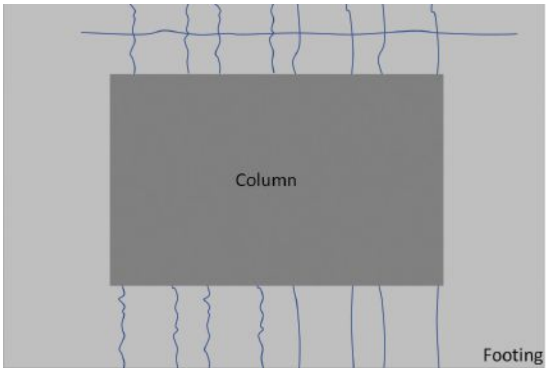


Figure 3. Schematic plan view of a single column–to–footing connection showing thermal cracking in the footing. Figure: University of Texas at Austin.

Figure 4. Cracking due to temperature gradients can be problematic with mass concrete placement. Figure: University of Texas at Austin.

difference between the hottest and coolest points in the footing becomes about 50°F. Due to this temperature differential, the cracks shown in Fig. 3 occur. Interestingly, this cracking resembles some of the cracking patterns we have discussed earlier in this series; similar map cracking may stem from ASR. However, cracking due to ASR will not be observed for days or weeks after the concrete is placed. If early-age map cracking is observed, it is probably not related to ASR. DEF cracks and restrained shrinkage cracks may also take the form of map cracking. Let us remember, a crack is not a crack. We must understand the root cause.

An examination of temperature-time histories of the footing can help us determine what may have happened. In addition, petrographic examination of a few cores taken from the footing could provide additional information to identify the root cause of the cracking. When cracking is due to thermal effects, an engineer must evaluate the structural and durability implications for the concrete component and invoke the use of remedial measures to restore the intended service life and structural capacity.

Figure 4 shows an example of thermal cracking in a real-world scenario. If conditions are not well managed, this type of cracking can be quite common; therefore, owners typically require management plans for mass concrete placement. One of the key takeaways from this discussion is that thermal cracking– and temperature gradient–related cracking issues cannot be directly tied to a single dimensional value such as 3, 4, or 5 ft. Depending on all the factors that surround heat generation during hydration and heat dissipation during curing, thermal cracking issues may be associated with dimensions that are different than the dimensions given in general guidance for mass concrete. ConcreteWorks is a commonly used software resource for those who are working on such thermal cracking issues and developing management plans. Management of heat of hydration, curing temperature, and dissipation is likely the most effective way of dealing with thermal cracking–related issues in concrete. Prevention, as always, is worth its weight in gold. Once the thermal cracking occurs, owners may incur additional expenses to rout and seal the cracks, apply coating on the structural component, invoke additional inspections, and so on. All these items can be costly and difficult to perform, and they may directly influence the service life of the concrete component and therefore the bridge.

Predicting the service life of a concrete component with thermal cracks is not a simple endeavor. Most service-life prediction models are based on diffusion of chemical solutions in porous media. Once cracks form in the concrete, the fundamental assumptions that are inherent to service-life prediction models come under question. Depending on their location, width, and depth, cracks in the concrete may provide direct conduits for chlorides to reach the reinforcing steel. Action plans for addressing thermal cracks in concrete will relate to the reinforcement type (for example, coated or uncoated steel, galvanized or pure stainless steel), as well as the environmental conditions. Once again, not all cracks are created equal. Understanding the root cause and the overall structural and durability context is key to making decisions that will be effective.

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Rethinking Connections to Accelerate the Construction of Delaware Bridges 1-488 North and South

by Nicholas Dean, Delaware Department of Transportation

For years, the standard Delaware Department of Transportation (DelDOT) practice for connecting adjacent-member bridge superstructures was to use grouted keyways, welded shear connector plates, and post-tensioned tie rods to connect the individual beams in a single unit once they were in place. However, this method of construction caused reflective longitudinal cracking in the decks of many bridges, which led DelDOT to experiment with the use of ultra-high-performance concrete (UHPC) in a revised shear-key shape developed by the Federal Highway Administration (FHWA). DelDOT first used UHPC as part of a bridge replacement project in 2015. Since then, the use of UHPC for the connection of adjacent superstructure elements has been standard DelDOT practice, and numerous projects using the material have been completed.

DelDOT specifications for the UHPC joint mixture require a minimum compressive strength of 17.5 ksi. This minimum-compressive-strength standard is newly adopted and based on guidance from FHWA's 2023 report *Structural Design with Ultra-High Performance Concrete*.[1] Because of DelDOT's continued commitment to the use of UHPC, precast concrete producers in the surrounding region have invested in reusable formwork for the superstructure shear key (longitudinal joints). While reusable formwork represents a significant upfront cost for the fabricator, it is also a huge step toward normalizing the use of UHPC.

The replacement of two bridges (1-488N&S) over Blackbird Creek along US 13 in northern Delaware was an opportunity to further refine UHPC practices. The successful use of the decked bulb-tee superstructure and precast concrete approach slabs for Bridges 1-488N&S was heavily predicated on the use of UHPC for the joints. Without UHPC, the closure pours and formwork for the joints would have been much larger and involved much more labor. Additionally, reflective cracking would have likely occurred if UHPC were not used. (For more information on the Bridges 1-488N&S over Blackbird Creek project, see the Project article on page 14.)

UHPC Decked Bulb-Tee End-Diaphragm Joints

The precast, prestressed concrete decked bulb-tee beams used for the Bridges 1-488N&S over Blackbird Creek project have large precast concrete end diaphragms that function as the backwall and seats for the precast concrete approach slabs. Because of the complexity of these types of diaphragms, it is common to field-cast them.

Crew members work on the top forms for the longitudinal ultra-high-performance concrete (UHPC) shear key joints. The system of top forms and buckets filled with UHPC creates a pressure head and forces UHPC into all of the voids within the joint. All Photos: Delaware Department of Transportation.

A close-up view of the approach slab joint showing the noncontact lap splice between the precast concrete elements. Backer rod is used to create a watertight seal and prevent ultra-high-performance concrete leakage.

However, given the time constraints of the project, the end diaphragms were precast integral with the beams and connected transversely with UHPC at the jobsite. In Delaware, placement of the entire joint in a single operation is preferred, but the contractor expressed concerns that the time needed to obtain appropriate UHPC strengths would affect the timing of other operations. Therefore, instead of holding up other operations while the contractor prepared formwork for the longitudinal shear key joints of the beams, the UHPC for the end diaphragms was placed separately, before that of the longitudinal joints.

The contractor also collaborated with the precast concrete producer during the shop drawing phase to address challenges related to the complexity of the joint and the limited space around the end diaphragms. The team decided to have threaded inserts installed on the underside of the end-diaphragm joint and back of the end diaphragms to facilitate placement of the formwork for the UHPC. In advance of the UHPC placement, the contractor's carpenters prepared the forms. After the beams were set, the threaded inserts and precut forms allowed the contractor to quickly prepare for the UHPC placement. The end-diaphragm joints required vertical placement of the UHPC, so the watertight integrity of the joints was even more important than usual. The formwork for the end-diaphragm joints consisted of ¾-in.-thick, 10-in.-wide lengths of plywood to cover the 6 in. joint width. On each side of the plywood formwork, ½-in.-thick foam expansion material was attached for the entire length of the joint. Tie rods were then threaded into the inserts and used to secure the plywood to the end diaphragms. By sufficiently tightening the nuts on the tie rods, the crews were able to compress the strips of foam expansion material and create a watertight seal. Because the UHPC in the longitudinal beam joints would be placed later, bulkheads were installed between the end diaphragm and the longitudinal beam joint.

Placing the UHPC for the joints of the end diaphragms required roughly 5 yd^3 per bridge. All the work associated with the UHPC end-diaphragm joint placement was performed over four days: one day per bridge to prepare the surface and install formwork, and approximately 1.5 hours of UHPC placement per bridge. To achieve a saturated surface dry (SSD) condition for the surface, the contractor used soaker hoses and wet burlap in the joints the day before the UHPC was placed. This preparatory step allowed the internal pores of the precast concrete joint to absorb the moisture from the soaker hoses and wet burlap, and kept the precast concrete from pulling the water out of the UHPC.

Workers place ultra-high-performance concrete (UHPC) from a concrete hopper into a longitudinal joint between adjacent decked bulb-tee beams. After the UHPC is placed, the joint is top formed and a pressure head system is created.

The timing of the backfilling operations for the foamed glass aggregate (FGA), as well as construction of the temporary walkways outside of the bridge parapets, was dependent on achieving adequate strength in the UHPC. Backfilling against the abutments and attaching the overhang brackets and walkways to the beams required that the UHPC reach a compressive strength of 10 ksi. It took the UHPC approximately two to three days to gain the required strength. By sequencing the UHPC placement operations, the contractor was able to give the UHPC in the end-diaphragm joints a head start on gaining strength while crews worked to assemble the formwork for the longitudinal joints. This sequencing allowed the other construction operations to continue alongside the UHPC placement for the longitudinal shear keys.

UHPC Decked Bulb-Tee Shear Key Joints

For the longitudinal joint formwork, lengths of 2× lumber were placed across the 6 in. joint width. As was done for the end-diaphragm joints, ½-in.-thick foam expansion material was attached to each side of the lumber, and tie rods

Watertight formwork on the backside of the beam for the ultra-high-performance concrete (UHPC) placement in the end diaphragm joints. A series of threaded inserts, ½-in.-thick foam, tie rods, and plywood are used to ensure that no UHPC is lost due to leakage.

End diaphragms are typically cast-in-place because of their complexity. However, given the time constraints of this project, the end diaphragms were precast integral with the beams and connected transversely with ultra-high-performance concrete at the jobsite.

were used to pull the bottom form tight against the underside of the beams and create a watertight joint. This approach with the formwork is something that the contractor developed and fine-tuned over the course of two other DelDOT bridge construction projects involving UHPC. By paying close attention to detail with the formwork, the contractor was able to avoid losing UHPC from the longitudinal joints.

Approximately 16.5 yd^3 of UHPC per bridge was placed for the longitudinal joints. All the work associated with the longitudinal UHPC joint placement was performed over six days: two days per bridge to prepare the surface and install formwork, and approximately four hours per bridge to place the UHPC.

During the UHPC installation in the longitudinal joints, the contractor elected to try two different techniques. The first followed common DelDOT practice for a 6 in. joint with a UHPC closure pour. UHPC was placed into troughs to help guide the materials into the joint. These troughs were moved along the beams from low to high elevation. As the UHPC was placed in the joint, a top form composed of ¾-in.-thick, 10-in.-wide lengths of plywood was applied. As was done for the formwork for the end diaphragm, ½-in.-thick foam expansion material was attached for the entire length of the joint. Concrete screws were drilled into the top edges of the precast concrete decked bulb-tee to cinch the plywood and expansion material down tight. At tenth-points along the beam, crews attached buckets with holes drilled into the bottom to the joint. These buckets were filled with UHPC and created a pressure head system, forcing the viscous material into any open void within the joint. As the UHPC placement progressed, the buckets were continuously topped off to maintain the pressure head.

This tried-and-true technique has worked on many DelDOT bridge construction projects in the past. When compressed, the foam expansion material maintains approximately ¼ in. of UHPC overpour in the joint. This hardened overpour is then ground flush with the top of the precast concrete element. The advantage of using this approach is that it helps ensure that UHPC fills every void and limits the potential for low spots in the joint. The drawback is that the work to top-form the longitudinal joint requires a significant amount of time, personnel, and effort.

For a portion of the concrete placement for the longitudinal joint on Bridge 1-488N, the contractor used a different UHPC placement technique: installing the product without top forming the joints. This option was suggested by a representative for the UHPC supplier, and the contractor decided to try it because it could potentially save time during the UHPC placement process; however, they were aware that there could be a greater likelihood of low spots developing in the joint because of underpouring.

In practice, this method did not yield the desired advantages. Because of the slope of the beams, the self-consolidating UHPC gravitated toward the low point and overflowed the joint. Notably, the contractor expressed the belief that this method could be feasible under the right conditions. However, given the time frame for completing the project, the contractor returned to the more conventional UHPC placement technique.

UHPC Approach Slab Shear Key Joints

After the UHPC was placed for the decked bulb-tee beams, the contractor placed the precast concrete approach slab segments. The use of fully precast concrete approach slabs was made possible because of the strength and durability that the UHPC connection can provide. For this project, the joints between the approach slab segments were designed to match FHWA's shear key for adjacent box beams. Because this shear key is only $^3/_8$-in. wide, the contractor was able to use backer rod in lieu of wooden formwork.

Adjacent decked bulb-tee beams are staged in the precaster's yard. The backsides of the beams show the threaded inserts for the ultra-high-performance concrete (UHPC) formwork. The reinforcement configuration creates a noncontact lap splice in the UHPC joint between beams.

An adjacent decked bulb-tee beam is staged in the precaster's yard. The precast concrete interface that will form the side of the longitudinal joint has a roughened concrete surface and reinforcement that will project into the joint to help the UHPC bond to the precast concrete beam.

The slope of the approach slabs on the north side of the bridges is ±0.5%, while the slope is ±2.5% on the south side. Learning from the experience with the UHPC placement on the longitudinal beam joints, the contractor implemented a hybrid technique of top forming the southside approach slab joints and open placement of the northside approach slab joints. Approximately 1.5 yd^3 of UHPC per bridge was placed in the joints of the approach slabs. All work associated with placement of the UHPC approach slab joints was performed over three days: one day per bridge to prepare the surface and install formwork, and approximately 1.5 hours of UHPC placement per bridge.

UHPC Mixing and Delivery Method

The UHPC for Bridges 1-488N&S was mixed off site, at a concrete plant. This was a new mixing and delivery method for UHPC in Delaware.

On previous UHPC projects, the material was mixed on site with high-energy, high-shear mixers. That on-site process adds several layers of complexity to bridge projects, such as the following:

- Many bridge replacement/rehabilitation projects have relatively small footprints. As such, staging of the mixers, batch materials, and UHPC delivery systems can become a major concern.
- When UHPC is mixed on site, the number of personnel on site increases signifcantly. Depending on the size of the UHPC placement, it is not uncommon for the contractor to allocate 20 to 25 people for mixing UHPC, delivering material from the mixers to the joints, assembling and installing top forms, topping off pressure head buckets, ensuring joint integrity, and general quality assurance/quality control processes.
- The success of UHPC placement depends on efficient delivery and installation of materials, whereas an interruption to on-site mixing operations can result in loss of material, the need for remedial actions, and the derailment of tight construction schedules. On UHPC projects in Delaware, it is common for mixers to malfunction. Therefore, DelDOT requires a minimum of two high-energy, high-shear mixers for every UHPC project, regardless of the size of the placement project.

With so many moving parts created by on-site mixing of UHPC, the risk of inefficiency and waste increases significantly.

For the Bridges 1-488N&S project, the UHPC material supplier trained and certified members of the local concrete supplier to mix the material using standard mixers. Using typical mixing machines allows for a level of control that cannot easily be achieved in the field. Additionally, training concrete plant staff in the nuances of UHPC helps them understand the material and troubleshoot any issues that could arise in production. Also, with this off-site mixing process, larger quantities of UHPC can be mixed in a single batch.

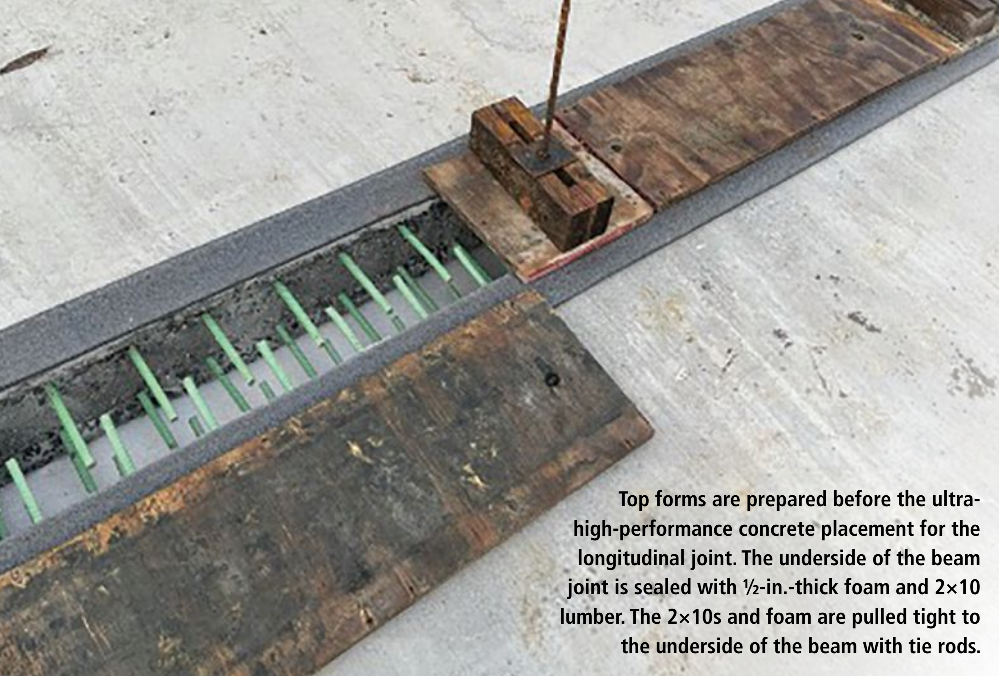
For this project, representatives from the UHPC supplier and DelDOT's Materials & Research section oversaw the mixing process at the plant. The mixing process took approximately 1.5 hours to complete. Once the UHPC was mixed, testing samples were taken, and the UHPC was loaded into a standard ready-mix truck. It then took another 30 minutes to deliver the material to the construction site. The mixing barrel on the concrete truck slowly rotated, and as a result, the UHPC remained workable for approximately 4 hours.

Once the UHPC was on site, it was loaded into a concrete hopper, which was lifted by crane and smoothly guided along the joints, quickly filling them with UHPC as it went. The concrete hopper, which had a capacity of 2 yd^3, had a spring gate that allowed the trained workers to easily control the flow of UHPC. On-site mixing procedures typically use wheelbarrows or concrete buggy machines during placement. That equipment offers a capacity of between 6 and 14 ft^3, which is only about 10% to 25% of the capacity of the concrete hopper. Use of the concrete hopper eliminated the need for wheelbarrows and buggies, and a smaller crew was easily able to place the UHPC, install top forms, maintain the pressure head system, and ensure joint integrity. The decreased site congestion increased the speed and efficiency of the UHPC placement process. It also improved the cleanliness of the construction site compared with an on-site UHPC mixing operation.

Conclusion

Some of the UHPC lessons learned from the replacement of Bridges 1-488 N&S are as follows:

- It is vital to have a contractor that is experienced in and open minded about the use of UHPC. The contractor for this job is one of Delaware's more knowledgeable UHPC installers. They continue to look for ways to improve delivery and implementation of the material, and they are open to suggestions throughout the construction process.
- Care should be taken when sealing the beams and building the formwork for UHPC placement. The contractor's formwork approach helped accelerate placement and eliminate UHPC leakage.
- Mixing UHPC off site eliminates the need for large mixers within the work zone, allows for larger UHPC batches, and provides an added level of quality control. It is important for



Top forms are prepared before the ultra-high-performance concrete placement for the longitudinal joint. The underside of the beam joint is sealed with ½-in.-thick foam and 2×10 lumber. The 2×10s and foam are pulled tight to the underside of the beam with tie rods.

concrete plant staff to be trained and certified in the UHPC mixing process, and it is also recommended that a supplier's representative be present for oversight.

- As noted by the contractor, material waste can be a concern during the off-site mixing of UHPC. The concrete plant that mixed the UHPC was only set up to batch the material in increments of 1 yd³. If smaller increments were achievable, that could help to reduce material waste. This issue might be resolved as this process becomes more mainstream and concrete plants adjust their procedures.
- Eliminating top forms may help speed up UHPC installation, but it is important that slopes are considered. The contractor found success when no top forms were used on flatter joints but encountered difficulties with this technique on slopes greater than 0.5%. With more time and planning, steps could be taken to mitigate issues on joints with steeper slopes.
- Using a concrete hopper improved the speed and efficiency of UHPC installation. It also decreased the number of personnel required for the operation and minimized site congestion.
- Off-site UHPC mixing and delivery of UHPC in ready-mix trucks improve the efficency of UHPC installation. Given the merit to this approach, it is likely that more UHPC suppliers will modify their mixture proportions to suit this method, leading to more concrete plants becoming proficient in mixing UHPC and further advancement of the industry.

By combining precast concrete elements with refined shear key joints and improved UHPC delivery methods, DelDOT expects that the new bridges over Blackbird Creek will have a 100-year design life with minimal maintenance.

Reference

1. Graybeal, B., and R. El-Helou. 2023. *Structural Design with Ultra-High Performance Concrete*. FHWA-HRT-23-077. Washington, DC: Federal Highway Administration. https://rosap.ntl.bts.gov/view/dot/72525.

Successful Collaboration and Stewardship Preserve Prestressed Concrete Bridge

by Susan M. Kovich, Nicholls Kovich Engineering PLLC

In August 2023, following a heavy rainstorm in central Washington state, a truck driver reported that Bridge 202 in Grant County "wobbled" during crossing, which was the first hint that something was amiss.[1] The bridge was immediately closed, and an emergency inspection revealed a critical substructure failure that resulted in one prestressed concrete girder without bearing support. While such damage could necessitate a full bridge replacement, this project instead became an example of concrete bridge stewardship—demonstrating how careful evaluation and innovative design preserved a prestressed concrete bridge and extended the life of local agency infrastructure.

In August 2023, a truck driver reported that Grant County (Washington) Bridge 202 "wobbled" and a subsequent emergency inspection revealed that significant erosion had occurred around the north abutment after heavy rains. Photo: Nicholls Kovich Engineering.

Background

Bridge 202 is a 130-ft-long, single-span prestressed concrete bridge constructed in 1989. The structure crosses the Rocky Coulee Wasteway Canal with a 34-degree skew. The canal, built by the U.S. Bureau of Reclamation, is an integral part of the Columbia Basin Project, which serves about 671,000 acres in east central Washington.

The bridge's superstructure consists of five prestressed concrete I-girders supporting an 8-in.-thick cast-in-place composite deck. Each girder weighs 105,000 lb and is supported on shallow concrete spread-footing abutments.

Failure Mechanism and Structural Assessment

The emergency inspection identified severe erosion around the northwest corner of the bridge, which had resulted in a large void beneath the north abutment footing. The abutment wall exhibited significant vertical cracking and spalling, and one exterior girder had completely lost bearing support. The void also extended below the concrete-lined canal.

Despite this loss of support, the prestressed concrete girders and deck remained structurally intact, with no visible signs of distress-related cracking. This performance is consistent with the inherent redundancy and load-distribution characteristics of prestressed concrete girder systems. Load redistribution through the deck, girders, and diaphragms likely limited demand increases in the unsupported girder, preventing structural distress.

Subsurface exploration confirmed that the original spread footing was founded on erodible silty sand, with competent basalt located 3 ft below the footing elevation. The abutment failure was attributed to a combination of surface runoff, groundwater seepage, and soil erodibility—conditions that led to catastrophic undermining of the abutment.

Evaluating Preservation Versus Replacement

With canal irrigation flows scheduled to resume in March 2024, less than seven months after discovering the damage,

To preserve the bridge superstructure, all five prestressed concrete girders at the north abutment were simultaneously jacked and temporarily supported by cast-in-place concrete shoring columns founded on micropiles. Photo: N. A. Degerstrom Inc.

the project team faced a constrained construction window. A multidisciplinary team—including Nicholls Kovich Engineering, Budinger & Associates Inc., N. A. Degerstrom Inc., and Grant County Public Works—evaluated repair alternatives. Nicholls Kovich Engineering, which provides on-call services to the county, was able to subcontract with the geotechnical engineer, and the contractor was retained by the county through an emergency contracting process.

A key decision point was whether the prestressed concrete girders remained viable for continued service. Field observations and engineering judgment confirmed the following:

- No evidence of overstress or excessive deformation in the girders
- No strand exposure or loss of prestress capacity
- No deck cracking indicative of redistribution-induced distress

Based on these findings, the superstructure was deemed salvageable. Keeping the girders avoided significant material costs, shortened the duration of construction and road closures, and minimized disruption to canal operations.

Stabilization and Temporary Support

The repair strategy involved complete removal and replacement of the north abutment while the bridge girders were temporarily shored. Initial stabilization included construction of a 5-ft-wide access pathway beneath the bridge and placement of approximately 80 ft^3 of lean concrete to fill undermining voids and stabilize the existing footing. The unsupported exterior girder was also shimmed to restore temporary bearing.

Extensive excavation was not possible close to the canal. Therefore, to temporarily support the bridge during reconstruction, a micropile-supported shoring system was designed and installed. Workers used a limited-access drill rig to install thirty 3.5-in.-diameter production micropiles and one test pile into the underlying basalt. The micropiles supported a continuous footing, which in turn supported five cast-in-place concrete shoring columns and steel jacking beams—one positioned beneath each girder. This system temporarily transferred superstructure loads directly to competent bedrock, bypassing the weaker soil zone and facilitating the safe removal of the damaged abutment.

The temporary supports were 10 ft from the end of the girder. The girders and deck were checked to ensure that there was no overstress in the temporary condition.

Abutment Reconstruction and Long-Term Mitigation

Following the successful transfer of load to the temporary support system, crews carefully removed and reconstructed the damaged abutment. A concrete leveling pad was placed directly on basalt to support a new spread footing, improving foundation reliability relative to the original design. The new substructure includes the following:

- 8-ft-wide concrete spread footings with a 6-ft-tall, cantilevered concrete abutment wall
- 13-ft-tall reinforced concrete wingwalls
- A structural earth retaining wall to support the approach embankment

A drainage gallery was incorporated into the design to intercept and convey groundwater and stormwater. This feature reduces the potential for future erosion and hydrostatic pressure buildup behind the abutment, thereby addressing the root cause of the abutment failure. Grant County further supplemented the repair with a stormwater management plan addressing runoff from adjacent roadways and agricultural lands.

The damaged north abutment was carefully removed, and a new concrete abutment and wingwalls were constructed on basalt bedrock. Photo: Nicholls Kovich Engineering.

The completed Bridge 202 project illustrates successful concrete bridge stewardship, in which a critical substructure failure offers an opportunity to preserve the existing prestressed concrete girders and extend the bridge's service life. The bridge's projected service life is 50 years from the time of the repair. Photo: Nicholls Kovich Engineering.

Design and Construction Challenges

The project team adapted the original design approach during construction to address several key challenges. Temporary support of the superstructure using shallow footings was initially planned. However, concerns related to excavation stability and worker safety prompted a shift to a micropile-supported system. This change allowed loads to be transferred directly to underlying basalt, without extensive excavations.

Vertical clearance beneath the bridge was limited to approximately 5 ft, which constrained construction means and methods. As a result, specialized equipment and careful sequencing of micropile installation and shoring system assembly adjacent to the canal were required.

During operations before jacking, field measurements confirmed that the exterior girder had deflected more than 1 in. lower than the interior girders. This finding proved that some load redistribution had occurred. Therefore, the jacking system was carefully synchronized to apply uniform pressure across all girders. This controlled approach minimized the risk of inducing additional stresses in the superstructure and allowed the girders to be reset onto new elastomeric bearing pads at consistent elevations.

Stewardship in Practice

This project illustrates the value of engineering stewardship in concrete bridge rehabilitation. Rather than defaulting to replacement, the project team evaluated the true condition and performance of the prestressed concrete girders and identified an opportunity to preserve them.

By maintaining the existing superstructure and focusing on intervention to address the failed substructure, the team

- extended the service life of the bridge;
- minimized environmental impact through material reuse; and
- maintained structural reliability through improved foundation and drainage design.

Additionally, the team was able to contain the budget while accelerating the construction timeline. The project was completed in just 5 months with a construction cost of $1.23 million, compared with 3 years and an estimated $5 million cost to replace the bridge.

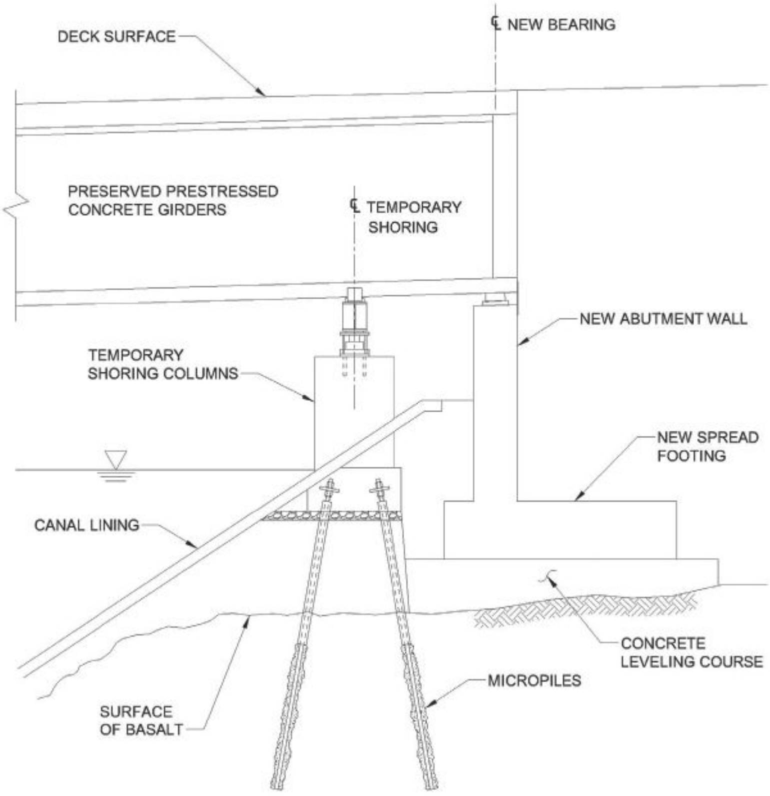
The success of the project depended on close coordination among the owner, engineer, geotechnical consultant, and contractor, as well as the team's willingness to pursue a technically rigorous rehabilitation approach under tight constraints. The project's achievements were recognized when the contractor, N.A. Degerstrom Inc., received the 2024 Build Northwest Award from the Inland Northwest Associated General Contractors in the Highway and Transportation Renovation category.

In an era of aging infrastructure and limited resources, this project demonstrates that preserving a prestressed concrete girder bridge can be both a practical and responsible solution when the strategy is supported by sound engineering evaluation.

Reference

1. Schweizer, S. 2024. "Grant County Bridge Work Under Way." *Columbia Basin Herald*, March 12, 2024. https://columbiabasinherald.com/news/2024/mar/12/grant-county-bridge-work-under-way.

The temporary shoring system and new abutment cross section show the tight physical constraints of the reconstruction adjacent to the irrigation canal. Figure: Nicholls Kovich Engineering.



Making the Concrete-Built World Last Longer: The International Concrete Repair Institute

By Eric Hauth, ICRI

Now in its 38th year, the International Concrete Repair Institute (ICRI) was born from the desire of its founding members to advance the concrete repair industry and the companies and professionals in that industry. Serving as both a technical institute and trade association, ICRI's mission is to make the concrete-built world safer and longer lasting. To fulfill this mission, ICRI is guided by four operational pillars: industry leadership, professional development, organization strength, and organization credibility. At present, ICRI counts nearly 2400 individuals and companies as members, including 47 supporting member companies.

ICRI creates industry resources such as *The Technical Guidelines for Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, Polymer Overlays, and Concrete Repair* (310.2R-2013). Concrete Surface Profile chips convey the proper concrete surface roughness before the application of coatings or sealers, or the start of repairs. Figure: ICRI.

ICRI is a proud associate member of the National Concrete Bridge Council (NCBC). Joining NCBC three years ago was a natural extension of the institute's mission, and ICRI's concrete repair knowledge and expertise have strengthened NCBC's focus on improving and sustaining concrete bridge infrastructure.

ICRI membership includes roughly equal proportions of material manufacturers/suppliers, contractors/applicators, and design professionals/engineers. As a result, the technical guidance developed by ICRI represents perspectives from a wide range of industry professionals.

Over its relatively brief history, ICRI has published 28 technical guidelines addressing important technical issues to inform better approaches and practices in concrete repair and service-life extension. These guidelines include ICRI 310.2R, *Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, Polymer Overlays, and Concrete Repair*.[1] A related product, Concrete Surface Profile (CSP) chips, has become an industry-standard tool for manufacturers, contractors, and design professionals in coatings and concrete repair. The CSP chips are available for purchase as a set of 10 tactile rubber reference cards designed to physically measure concrete roughness for proper surface preparation before coatings, sealers, and repairs are applied.

ICRI runs two certification programs: the Concrete Surface Repair Technician (CSRT) program and the Concrete Slab Moisture Testing Technician (CSMT) program. The institute is also developing a new strategic focus on applicator-based training programs to further advance the quality of concrete repair training in the field (see the "ICRI Resources" sidebar). In addition to the work of ICRI's technical institute, ICRI serves the concrete repair industry by hosting two conventions per year, supporting a wide network of chapters throughout North America (including Mexico), and celebrating leaders and notable projects through individual and project recognition awards.

ICRI Resources

In the area of training with applicability to bridge preservation, International Concrete Repair Institute (ICRI) runs two certification programs: the Concrete Surface Repair Technician (CSRT) program and the Concrete Slab Moisture Testing Technician (CSMT) program. These programs address fundamental considerations in assessing and repairing concrete structures. The institute is also developing a new applicator-based training program to further advance the quality of concrete repair training in the field.

Bridge repair professionals can also take advantage of ICRI's first ever digital app, available for free from the Apple App Store and Google Play, to assess reinforcing bar cleanliness in accordance with ICRI guideline 210.5, *Guide for Selecting and Specifying Reinforcing Bar Cleaning Levels*. ICRI encourages bridge engineers, field technicians, and inspectors to download the app and become familiar with this new tool.

For individuals looking to keep pace with the latest in concrete repair technical developments and case studies, ICRI publishes the *Concrete Repair Bulletin*, which is publicly available via ICRI's website: www.icri.org.

The upstream side of the historic Third Avenue Bridge in Minneapolis, Minn., is shown before rehabilitation was undertaken. Note the concrete deterioration in the pier bases and arches. Photo: Wiss, Janney, Elstner Associates.

ICRI's 2024 Project of the Year Award winner, the Third Avenue Bridge in Minneapolis, Minn., illustrates the vital importance of extending the service lives of concrete bridges.[2] The bridge, which was built in 1918, is on the National Register of Historic Places and is an icon in the Minneapolis skyline. Its preservation yielded significant sustainability impacts. According to David Whitmore, one of the key contractors involved in this project, the preservation effort achieved the following:

- The project kept 8000 yd^3 of concrete in service.
- The production of 16,000 tons of rubble was avoided.
- Carbon dioxide emissions were reduced by 4100 tons.

ICRI looks forward to future collaborations with federal and state partners, as well as cutting-edge training partners such as the Concrete Bridge Engineering Institute at the University of Texas, Austin. Through such partnerships, ICRI is prepared to further the respective missions of NCBC and ICRI.

References

1. International Concrete Repair Institute (ICRI). 2008. *Guideline for Surface Preparation for Repair of Deteriorated Concrete Resulting from Reinforcing Steel Corrosion*. ICRI 310.1R-2008. Minneapolis, MN: ICRI.
2. ICRI. 2024. "2024 Winner: 3rd Avenue Bridge Rehabilitation, Project of the Year." *Concrete Repair Bulletin* 37 (6): 14–19. https://www.icri.org/past-issues/concrete-repair-bulletin-november-december-2024.

Eric Hauth is the executive director of the International Concrete Repair Institute.

The project to preserve the historic Third Avenue Bridge in Minneapolis, Minn., received the International Concrete Repair Institute's 2024 Project of the Year Award. Photo: Trey Cambern Photography.[2]

What's Happening at CBEI: Education and Collaboration on Concrete Bridges

by Dr. Oguzhan Bayrak, Doug Beer, Dr. Thano Drimalas, Dr. Kevin Folliard, Gregory Hunsicker, and Deanna Mueller, Concrete Bridge Engineering Institute

Articles in past issues of *ASPIRE*® have highlighted the launch and progress of the Concrete Bridge Engineering Institute (CBEI) and its efforts to provide an immersive educational experience for workforce development. This article addresses recent and upcoming CBEI events and activities.

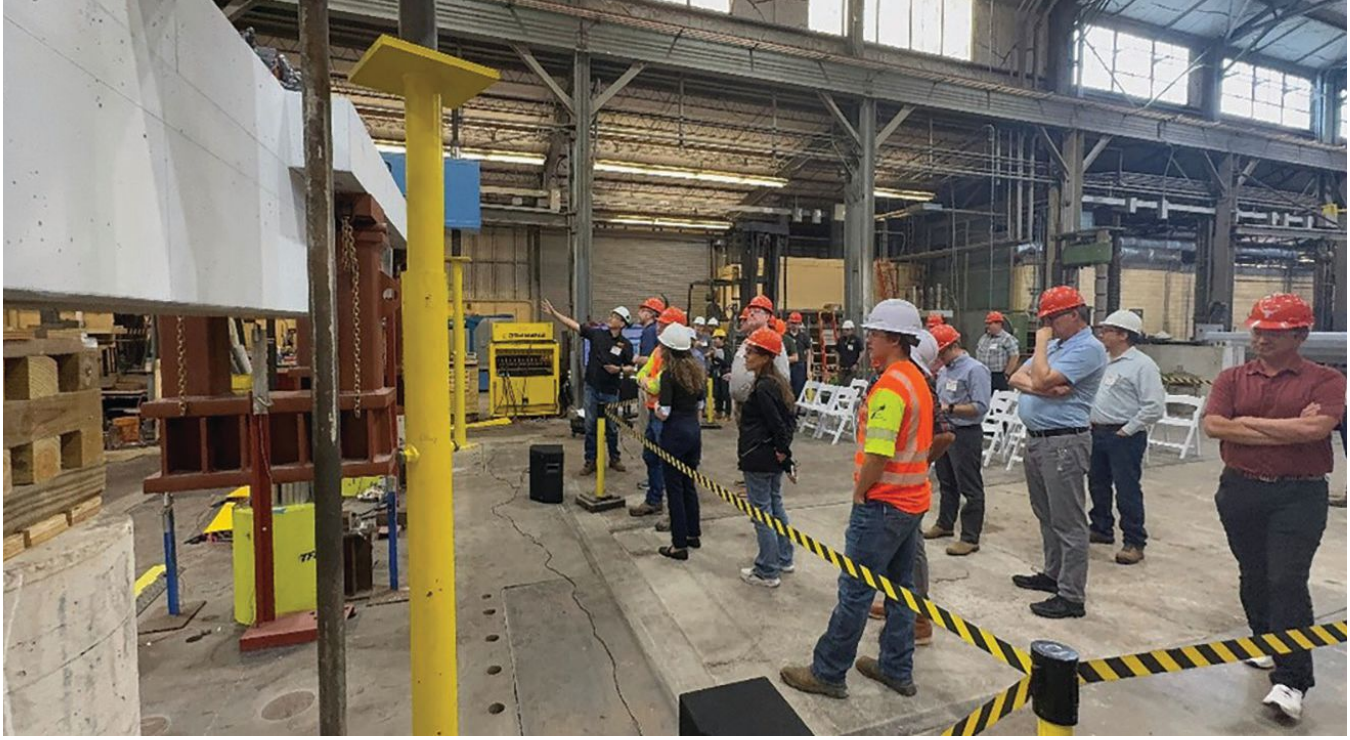
Continuing Growth of CBEI Courses and Collections

Since the kickoff of the Transportation Pooled Fund TPF-5(508) in June 2023, CBEI has continued to develop industry resources, including several courses. The Concrete Materials for Bridges and Bridge Deck Construction Inspection courses are being delivered, and the Post-tensioning Academy courses are scheduled for rollout later this year.

We are seeing a strong and growing demand to participate in the Bridge Deck Construction Inspection Course, which has received outstanding feedback about its format, delivery, and content since it was launched in 2025. CBEI recently delivered this course for the seventh time, and it is being offered nearly every month (see the Spring 2026 issue of *ASPIRE*).

A multidisciplinary approach is central to the Concrete Materials for Bridges Course, which continues to attract participation from concrete bridge industry professionals of all different experience levels who are seeking a deeper understanding of how material behavior influences bridge construction and long-term bridge performance. The course focuses on cement hydration and concrete mixture design, as well as the practical consequences of decisions related to curing, permeability, shrinkage, supplementary cementitious materials, environmental exposure, and construction practices. As transportation agencies increasingly emphasize service life and life-cycle performance in addition to the initial construction cost, understanding concrete materials has become more important than ever. Bridges are expected to perform reliably for 75 to 100 years or longer, often in aggressive environments with increasing traffic demands. Achieving these goals requires a strong understanding of durability and materials behavior throughout design and construction. What makes this CBEI course especially valuable is its practical and applied focus. Participants do not simply review specifications and theory; they evaluate real materials, discuss field performance, examine project case studies, and work collaboratively through durability-focused mixture design exercises. The course creates direct connections between materials decisions made during design and construction and the long-term maintenance and performance outcomes that transportation agencies experience decades later. For bridge professionals considering future offerings

Participants convene for structural behavior prediction testing on day 1 of the CBEI Conference and Workshop. All Photos: Concrete Bridge Engineering Institute.



Structural behavior predictions for an inverted tee specimen are tested at the CBEI Conference and Workshop.

of the course, the value extends across disciplines. Whether you are a bridge owner, engineer, inspector, contractor, manager, or materials professional, or serve in another role, you have likely run into questions related to the state of practice as it relates to concrete. Given the constantly changing market, questions such as those related to blended cements (especially ASTM C595 Type IL cement[1]) and the use of alternative binders are among the variables that concrete bridge professionals must understand and address.

Past attendees representing a diverse cross section of the industry have expressed appreciation for the value, learning, and context that they have gained from the Concrete Materials for Bridges Course. Some participants have noted that they now have a stronger understanding of constructability and durability issues and how construction practices influence long-term performance; others have emphasized that they gained greater insight into deterioration mechanisms and preventive strategies. The course is offered approximately quarterly, most recently in April.

Other CBEI initiatives such as the Bridge Component Collection have also continued to evolve. The collection now provides tangible concrete bridge elements for visitors and course participants to see and to use in demonstrations.

2026 CBEI Conference and Workshop

In late April, CBEI hosted participants from across the United States at the inaugural CBEI Conference and Workshop held at the CBEI facility in Austin, Tex. This event brought together many stakeholders who are working together to continually improve the performance and durability of concrete bridges. As the meeting illustrated, conversation at CBEI is currently focused on developing education modules to support the needs of the concrete bridge community.

A departure from many traditional conferences built around lecture sessions alone, the CBEI Conference emphasized direct engagement with interactive and hands-on demonstrations, tours, full-scale bridge components, and collaborative technical discussions. The first day was a workshop dedicated to identifying present-day industry needs and considering what can be done to address these needs going forward.

A unique portion of the first-day event was the Structural Behavior Prediction Competition. Participants evaluated a large-scale structural component and predicted how it would behave under specified loading conditions before

Attendees at the CBEI Conference and Workshop discuss a poster presentation about ongoing concrete bridge–related research.

testing. This experience sparked thoughtful discussion and reinforced the value of hands-on education and physical experimentation in a profession that increasingly relies on analytical modeling and digital tools. The exercise highlighted various approaches for modeling an element's behavior and explored estimates of capacity, deflection, and cracking. An important topic of discussion was levels of approximation and how they fit into the various approaches. In this context, the term "levels of approximation" is adopted from the *fib* (International Federation for Structural Concrete) *Model Code 2020*.[2] This code acknowledges that all calculations are approximations of the true response of a structural member. First-cut analyses (Level I approximation) may be appropriate in some cases to obtain quick estimates of capacities. Between that level of approximation and the most sophisticated/rigorous approaches (Level IV approximation), there are several levels of approximation that we can adopt in our calculations. This type of framework becomes especially important in evaluating the existing inventory of bridges and/or load-rating efforts. The ongoing "A Crack Is Not a Crack" series of *ASPIRE* articles was also highlighted as part of the discussion.

On the second day of the conference, a larger group—including Transportation Pooled Fund members from departments of transportation around the United States, engineers, inspectors, researchers, contractors, students, consultants, exhibitors, and representatives of industry organizations—gathered for an interactive experience focused on practical bridge engineering challenges and emerging technologies. Participants explored stations showcasing industry-led training and certification programs, bridge inspection technologies, nondestructive evaluation methods, durable post-tensioning systems, concrete materials research, rehabilitation strategies, structural testing, precast concrete technologies such as precast concrete forms, and durability-focused construction practices. By combining theoretical understanding with real-world observation, CBEI strives to create learning and collaboration opportunities that are difficult to replicate in a classroom, online platform, or conference room setting.

CTR Symposium

On April 8, a few weeks before the CBEI Conference, representatives of CBEI participated in the Center for Transportation Research (CTR) Symposium at the University of Texas at Austin. The long-standing annual symposium is a chance to explore the latest initiatives in advancing transportation infrastructure through research, innovation, and implementation. The broader goal of the CTR Symposium is to connect transportation research with practical applications across multiple disciplines that affect mobility and infrastructure systems. Topics discussed during the symposium include bridge engineering, transportation planning, pavement systems, freight mobility, intelligent transportation systems, traffic safety, autonomous technologies, construction innovation, durability, resilience, and infrastructure management.

Within that larger transportation conversation at this year's symposium, CBEI represented one of several major research initiatives demonstrating how implementation-focused collaboration can accelerate innovation and improve infrastructure performance. During the symposium, CBEI presented updates on its activities and hosted a tour of CBEI's facilities and components.

The symposium reinforced an increasingly important reality for the transportation industry: infrastructure challenges need interdisciplinary strategies. Bridge performance today involves more than structural design alone. Materials science, construction quality, inspection technologies, durability modeling, maintenance strategies, data collection, sustainability, and workforce development all intersect.

On the second day of the CBEI Conference and Workshop, participants gather for the Concrete Materials for Bridges demonstrations and tour led by Dr. Kevin Folliard and Dr. Thano Drimalas.

Upcoming Opportunities

Please visit www.cbei.engr.utexas.edu for more information about CBEI. Registration for upcoming courses is available through the CBEI website at https://cbei.engr.utexas.edu/training-certification.

Concrete Materials for Bridges Course:
- August 18–19, 2026

Bridge Deck Construction Inspection Course:
- July 7–9, 2026
- August 25–27, 2026

References

1. ASTM International. 2025. *Standard Specification for Blended Hydraulic Cements*. ASTM C595/C595M-25. West Conshohocken, PA: ASTM International.
2. *fib* (International Federation for Structural Concrete). 2020. *fib Model Code 2020*. Lausanne, Switzerland: *fib*.

Dr. Oguzhan Bayrak is a chaired professor at the University of Texas at Austin, where he serves as the director of the Concrete Bridge Engineering Institute. Doug Beer is a research engineer at the Concrete Bridge Engineering Institute within the University of Texas at Austin. Dr. Thano Drimalas is a research associate at the University of Texas at Austin. Dr. Kevin Folliard is the Walter S. Bellows Centennial professor in the Department of Civil Engineering at the University of Texas at Austin. Gregory Hunsicker is a research engineer at the University of Texas at Austin and deputy director of the Concrete Bridge Engineering Institute. Deanna Mueller is a senior research program coordinator at the Concrete Bridge Engineering Institute within the University of Texas at Austin.

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

IN THIS ISSUE

https://www.aspirebridge.com/magazine/2018Spring/CBT-PracticalSolutionForSkewedGeometry.pdf
This is a link to a Spring 2018 *ASPIRE*® article, "Practical Solution for Skewed Geometry on Decked-Girder Bridges," by Nicholls and Kovich. That article, which is mentioned in the Focus article on page 5, describes a simplified method to calculate bearing elevations of individual girders on a skewed bridge that takes into consideration camber and cross and longitudinal slopes.

https://www.txdot.gov/business/resources/design-tools-training/txdot-fhwa-engineering-software.html
The Concrete Bridge Technology article about cracking on page 22 discusses the challenges associated with defining mass concrete and managing the heat of hydration generated by mass concrete during curing. This link is to the publicly available software package (ConcreteWorks) mentioned in the article. The software allows users to make multiple adjustments to manage heat generation and dissipation to minimize the risk of thermal cracking.

https://aspirebridge.com/magazine/2015Winter/Project-PlatformSpansManhatten.pdf
The concrete segmental bridge that supports a skyscraper for the Manhattan West project in New York City is noted in the Concrete Bridge Technology article on page 19. This link is to the Manhattan West project article published in the Winter 2015 issue of *ASPIRE*.

Pennsylvania Launches Accelerated Construction Inspector Pipeline to Address Infrastructure Workforce Shortage

On February 25, 2026, Pennsylvania's Construct PA initiative graduated its first cohort under the Commonwealth Workforce Transformation Program (CWTP), a program designed to address labor shortages in the construction and infrastructure sectors. Established by executive order in July 2023 and funded through the federal Infrastructure Investment and Jobs Act, CWTP is a first-of-its-kind state-level job training grant program that targets companies, contractors, and unions working on infrastructure projects.

The four-week accelerated training provides the Pennsylvania Department of Transportation's Transportation Construction Inspector certification; field experience with earthwork equipment and concrete testing; and certifications in worksite safety, flagging, and safe driving. Participants also complete coursework in construction mathematics, plans reading, and professional skills. On completion, graduates are connected directly with employers seeking to expand their teams.

Eligible participants are 18 to 24 years old, hold a GED or high school diploma, possess a valid driver's license, and have had prior contact with the justice system. The program provides safety equipment, and support services covering childcare and transportation costs are available.

Having launched in Harrisburg, Construct PA is expanding to additional Pennsylvania communities in 2026. Pennsylvania has increased funding for career, technical, and apprenticeship programs by roughly 50% since 2023, with the 2026–2027 budget proposing further workforce investment. Other states facing similar infrastructure labor shortages may look to the CWTP model as a replicable framework.

Source: Pennsylvania Department of Transportation. 2026. PennDOT Statewide News, February 26, 2026. https://www.pa.gov/agencies/penndot/news-and-media/newsroom/statewide.

https://nationalconcretebridge.org/webinars
The LRFD article on page 43 refers to an August 20, 2025, webinar hosted by the National Concrete Bridge Council (NCBC), which focused on implementation of Agenda Item 39 from the June 2025 meeting of the American Association of State Highway and Transportation Officials' Committee on Bridges. A recording of the webinar, "The New PCI Recommended Practice to Assess and Control Prestressing Strand/Concrete Bonding Properties," can be accessed from this link.

https://abc-utc.fiu.edu/mc-events/deldots-accelerated-and-innovative-entirely-precast-bridges-us-13-over-blackbird-creek
The Project article on page 14 and the Concrete Bridge Technology article on page 26 highlight the successful application of accelerated bridge construction techniques and ultra-high-performance concrete technology in a Delaware Department of Transportation (DelDOT) project to replace two bridges over Blackbird Creek. This is a link to a recorded Florida International University webinar about the project, titled "DelDOT Accelerated and Innovative Entirely Precast Bridges – US 13 Over Blackbird Creek."

Exploring Methods for Fast, Sustainable Partial-Depth Concrete Bridge Deck Repair

by Dr. Andrew D. Sorensen, Texas A&M University, and Dr. Israi I.H. Abu Shanab, AECOM

In 2025, the United States had over 4 billion ft^2 of in-service bridge deck, and 5% of that bridge deck was rated as being in poor condition.[1] That means approximately 200 million ft^2 of bridge deck needed either repair or replacement. One commonly used method for the repair of concrete bridge decks is partial-depth concrete repair, a technique in which small, partial-depth patches of deteriorated concrete bridge deck are removed and replaced with new cementitious material. By removing the deteriorated patches, the spread of the concrete degradation is limited, thus extending the service life of the bridge deck. While significant advances have been made in the development of rapid-setting cementitious materials for partial-depth bridge deck repair,[2] the concrete removal and patch preparation process is still laborious and time consuming.

Sustainability Study

In recent research,[3,4] the authors investigated methods to reduce the preparation time for concrete patching through automation. The goal was to improve the efficiency of partial-depth deck repair while simultaneously reducing air-pollutant emissions. A review of the existing literature showed that four common concrete removal methods are used in partial-depth repairs: saw and patch, chip and patch, mill and patch, and water blast and patch. Using a Utah bridge deck repair project as a case study,[2] investigators estimated five air-pollutant quantities (CO_2, CO, NO_x, SO_2, and PM_{10}) for each of the four removal methods by employing the MOVES2014b model[5] and GREET model.[6]

MOVES (MOtor Vehicle Emission Simulator) is an emissions modeling system that calculates air pollutants while adjusting for conditions such as speed, temperature, and vehicle age. GREET (Greenhouse gases, Regulated Emissions, and Energy use in Technologies) is a full life-cycle assessment model that evaluates the environmental impacts of technologies, fuels, and vehicle combinations across their entire life. These two complementary models were used to calculate emission quantities: MOVES2014b for vehicle activity and GREET for life-cycle analysis of fuel and vehicle production (see **Table 1**).

Comparing the equipment use time, investigators also estimated the traffic delays for each method (**Table 2**). The results show that the methods that produce the most pollutants (mill and patch, and water blast and patch) require less time, whereas those that produce smaller amounts of pollutants (saw and patch, and chip and patch) require more time. This relationship demonstrates the difficulty in balancing sustainability with efficiency in construction practices. However, increased traffic delays may also increase the amount of emitted pollutants. This topic is discussed in a previous article by the authors.[3]

Reduced Preparation Time Study

After completing the sustainability portion of the study, the authors investigated whether the methods with lower levels of pollutant emissions could be automated to reduce the amount of time needed to prepare the patches. With this outcome in mind, the saw and patch method was chosen for additional study due to its popularity

Table 1. Estimated air-pollutant emissions associated with the method of removing 31 m^3 of concrete (based on a Utah bridge deck repair project case study)[2]

Method	CO_2, kg	CO, kg	NO_x, kg	SO_2, kg	PM_{10}, kg	Total emissions, kg	Total emissions, kg/m^3
Saw and patch	720.371	3.792	2.019	0.0824	0.300	726.565	23.7
Chip and patch	649.231	2.602	1.911	0.0819	0.293	654.119	21.2
Mill and patch	5756.444	3.934	9.998	0.101	0.531	5771.008	187.5
Water blast and patch	3317.890	4.671	13.874	0.142	0.921	3337.497	108.4

Table 2. Estimated traffic delays for case study based on average lane capacity of 560 vehicles per hour[2]

Method	Required time, hours	No. of vehicles delayed
Saw and patch	92.24	51,654
Chip and patch	88.49	49,554
Mill and patch	57.61	32,262
Water blast and patch	33.66	18,850

among contractors and its potential for automation.

Four unreinforced concrete slabs measuring 5 ft wide, 5 ft long, and 10 in. deep were cast, each with different concrete mixture proportions and different compressive strengths. The target compressive strengths for all specimens were between 5000 and 7000 psi. Each concrete slab was then marked into four equal areas measuring 2 ft by 2 ft (**Fig. 1**), and the concrete was removed to a target removal depth of 3 in. following four different discretized sawing patterns. The concrete saw used was a gas-powered model with a 14-in.-diameter blade, and the jackhammer model produced a triaxial vibration of 5.9 m/s^2 for chiseling into concrete. For each method, the removal and equipment usage times for each of the slab sections were measured. The differences between each method are the number of saw-cut lines (from 4 to 10) and the quantity of subpieces required to be removed. Figure 1 shows the discretization, with each drawn line representing a saw-cut line.

Investigators found that as the number of saw-cut lines increased, the removal time, which includes both saw cutting and jackhammering, decreased.[4] The explanation for this finding stemmed from discussion with the operator who removed the concrete patches. The operator, a semiskilled laborer with previous experience using both the concrete saw and jackhammer, noted that using the saw required much less physical exertion than using the jackhammer. Furthermore, because the concrete pieces for the more-discretized patches were smaller in size, the operator could simply "pop off" the concrete with the jackhammer (as shown in **Fig. 2**). As such, the removal of the concrete was easier and the physical toll on the laborers was less; therefore, the workers could be more efficient and work for longer intervals. The results also show the potential for future automation as saw cutting seems to be more easily automated than jackhammering. Subsequent studies in this area are currently underway.

Finally, the results of the secondary study were applied to the original case study of the Utah bridge deck to demonstrate the removal time for each method. The results show a dramatic decrease in removal time and a corresponding decrease in overall construction time as the number of saw-cut lines increases in a given surface area of bridge deck (**Table 3**).

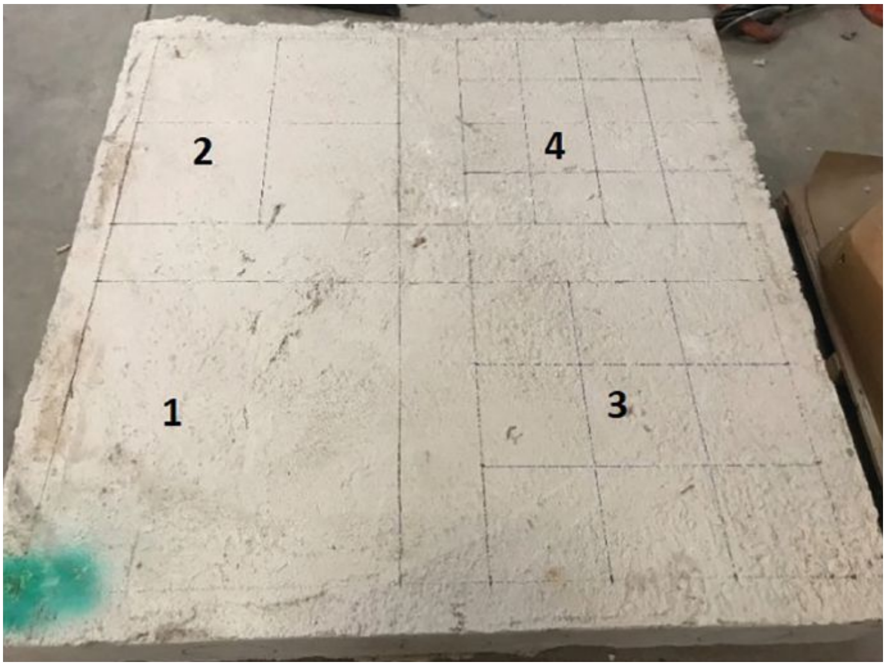


Figure 1. Saw-cutting patterns on the slab specimens help establish the relationship between time and air-pollutant emissions during partial-depth concrete deck repair methods. All Figures: Courtesy of Israi Abu Shanab.

Figure 2. Concrete removal method 4 uses the most saw cutting of the four tested methods to reduce the jackhammering time. The concrete removal time, equipment usage, and labor required for partial-depth concrete deck repairs were compared for each method.

Conclusion

This study showed that, of the most widely used repair preparation methods, saw and patch produces the least amount of pollutants. Additionally, by increasing the number of saw cuts, the removal time can be decreased dramatically.

Table 3. Required time for concrete removal using the four methods for the case study bridge deck

Method	Total predicted removal time, hours
1 (4 saw-cut lines)	656.71
2 (6 saw-cut lines)	610.07
3 (8 saw-cut lines)	598.02
4 (10 saw-cut lines)	426.05

Note: Time was predicted based on extrapolation of time for each method in the laboratory setting.

Acknowledgments

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AASHTO LRFD Bridge Design Specifications: Minimum Reinforcement Requirements, Strand Bond, and Prestressed Concrete Piles

by Dr. Oguzhan Bayrak, University of Texas at Austin

During the June 2025 meeting of the American Association of State Highway and Transportation Officials' (AASHTO's) Committee on Bridges and Structures (COBS) in Dallas, Tex., COBS approved several agenda items regarding changes to be incorporated in the forthcoming 11th edition of the *AASHTO LRFD Bridge Design Specifications*.[1] This article provides an in-depth overview of three of those approved items: agenda items 36, 39, and 40. For discussion of other approved AASHTO agenda items, refer to articles in the Fall 2025 and Spring 2026 issues of *ASPIRE*®.

Agenda Item 36

This agenda item implements the proposed recommendations from the National Cooperative Highway Research Program's *LRFD Minimum Flexural Reinforcement Requirements* (NCHRP Report 906),[2] which focuses on the minimum flexural reinforcement requirements of the AASHTO LRFD specifications. The estimate of the in situ tensile strength of concrete plays a significant role in determining the required minimum flexural strength of prestressed concrete components. For additional information about this topic, interested readers are referred to *fib* (International Federation for Structural Concrete) Bulletin 1, *Structural Concrete*,[3] and an article by Tuchscherer and Bayrak.[4] According to *fib*,[3] as beam depth increases, the flexural tensile strength of concrete (as determined by a beam test or a modulus of rupture test) approaches the tensile strength measured in a direct tension test. An in-depth investigation of the variability of tensile strength of concrete and its dependence on the member depth is discussed in *fib* Bulletin 1. This size effect that influences the tensile strength of concrete can be quite significant, and deeper members may crack under lower tensile stresses. Incorporating this behavior into the AASHTO LRFD specifications through the findings of NCHRP Report 906 provides significant benefits when calculating minimum flexural reinforcement. These requirements are intended to ensure ductility by specifying sufficient reinforcement to carry the tensile resultant force in the concrete and transfer it to the reinforcement crossing the flexural crack if and when flexural cracking occurs. With this discussion serving as background, Agenda Item 36 makes several modifications to the AASHTO LRFD specifications.

The notation in Article 5.3 will be revised as follows:

h = *overall thickness or depth of a member (in.); lateral dimension of the cross-section in the direction considered (in.); overall dimension of precast member in the direction in which splitting resistance is being evaluated (in.); least thickness of component section (in.) (5.6.3.3) (5.8.4.5.3) (5.9.4.4.1) (5.10.6)*

α_s = *strength factor for minimum reinforcement (5.6.3.3)*

Article 5.4.2.6 will be revised to read:

Unless determined by physical tests, the modulus of rupture, f_r*, for lightweight concrete with specified compressive strengths of up to 10.0 ksi and normal weight concrete with specified strengths up to 15.0 ksi may be taken as* $0.24\lambda\sqrt{f'_c}$ *where* λ *is the concrete density modification factor as specified in Article 5.4.2.8.*

Where physical tests are used to determine modulus of rupture, the tests shall be performed in accordance with AASHTO T 97[5] *and shall be performed on concrete using the same proportions and materials as specified for the structure. The test units shall be cured in the same manner as the production components.*

Commentary C5.4.2.6 will be revised as follows:

Most modulus of rupture test data on normal weight concrete are between $0.24\lambda\sqrt{f'_c}$ *and* $0.37\lambda\sqrt{f'_c}$ *(ksi) (Walker and Bloem, 1960*[6]*) (Khan, Cook, and Mitchell, 1996*[7]*). A value of* $0.37\lambda\sqrt{f'_c}$ *has been recommended for the prediction of the tensile strength of high-strength concrete (ACI 363, 2010*[8]*). However, the modulus of rupture is sensitive to curing methods, and nearly all of the test units in the dataset mentioned previously were moist cured until testing. Carrasquillo et al. (1981*[9]*) noted a 26 percent reduction in the 28-day modulus of rupture if high-strength units were allowed to dry after 7 days of moist curing over units that were moist cured until testing.*

The flexural cracking stress of concrete members has been shown to decrease with increasing member depth. Past research has suggested that the flexural cracking stress may be considered to be proportional to member height (Shioya et al., 1989[10]*) (Carpinteri and Corrado, 2011*[11]*). For example, the research shows a 36.0 in. deep girder achieves a flexural cracking stress that is 31 to 57 percent lower than that of a 6.0 in. deep modulus of rupture test specimen.*

Since modulus of rupture units are either 4.0 or 6.0 in. deep and typically moist cured up to the time of testing, the modulus of rupture should be significantly greater than the flexural cracking stress of a typical bridge member composed of the same concrete.

The properties of higher-strength concretes are particularly sensitive to the constitutive materials. If test results are to be used in design, it is imperative that tests be made using concrete with not only the same mix proportions, *but also the same materials and curing procedures as the concrete used in the structure.*

Article 5.6.3.3 will be revised as follows:

Unless otherwise specified, at any section of a noncompression-controlled flexural component, the amount of prestressed and nonprestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r*, greater than or equal to the lesser of the following:*

- α_s *times the factored moment required by the applicable strength load combination specified in Table 3.4.1-1;*

$$M_{cr} = \gamma_3\left[\left(\gamma_1 f_r + \gamma_2 f_{cpe}\right)S_c - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right)\right] \quad (5.6.3.3\text{-}1)$$

where:

α_s = *strength factor for minimum reinforcement*

For prestressing steel:

$$1.0 \le \alpha_s = 1.0 + \frac{0.33\left(\varepsilon_t - \varepsilon_{cl}\right)}{\varepsilon_{tl} - \varepsilon_{cl}} \le 1.33 \quad (5.6.3.3\text{-}2)$$

For nonprestressed reinforcement:

$$1.0 \le \alpha_s = 1.0 + \frac{0.2\left(\varepsilon_t - \varepsilon_{cl}\right)}{\varepsilon_{tl} - \varepsilon_{cl}} \le 1.2 \quad (5.6.3.3\text{-}3)$$

ε_t = *net tensile strain in the extreme tension steel at nominal resistance (in./in.)*

ε_{cl} = *compression-controlled strain limit in the extreme tension steel (in./in.)*

ε_{tl} = *tension-controlled strain limit in the extreme tension steel (in./in.)*

M_{cr} = *cracking moment (kip-in.)*

f_r = *modulus of rupture of concrete specified in Article 5.4.2.6*

f_{cpe} = *compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads (ksi)*

S_c = *section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (in.^{3})*

M_{dnc} = *unfactored dead load moment acting on the monolithic or noncomposite section (kip-in.)*

S_{nc} = *section modulus for the extreme fiber of the monolithic or noncomposite section where tensile stress is caused by externally applied loads (in.^{3})*

Appropriate values for M_{dnc} *and* S_{nc} *shall be used for any intermediate composite sections. Where the beams are designed for the monolithic or noncomposite section to resist all loads, substitute* S_{nc} *for* S_c *in the above equation for the calculation of* M_{cr}.

The following factors shall be used to account for variability in the flexural cracking stress of concrete members, variability of prestress, and the ratio of nominal yield stress of reinforcement to ultimate:

γ_1 = *flexural cracking variability factor*

= *1.2* $(h/12)^{-0.15}$ *for precast segmental structures*

= *1.6* $(h/12)^{-0.15}$ *for all other concrete structures,*

where h *is the overall thickness or depth of a member (in.)*

The first and third paragraphs in Commentary C5.6.3.3 will be revised to read:

Minimum reinforcement provisions are intended to decrease the probability of brittle failure by providing flexural capacity greater than the cracking moment. If this condition is not met, additional flexural strength is required by multiplying the required factored moment by α_s*. For tension-controlled sections,* α_s *is 1.33, which is equivalent to the inverse of the resistance factor (*ϕ*) for compression-controlled sections. For compression-controlled and transition sections,* α_s *is decreased to avoid double counting the additional strength requirement for decreased ductility that is already accounted for in* ϕ*. Based on the experimental data, a member having the minimum reinforcement is expected to possess a minimum displacement capacity of 1.0% of the span length.*

For precast segmental construction, cracking generally starts at the segment joints. Research at the University of California, San Diego, has shown that flexure cracks occur adjacent to the epoxy-bonded match-cast face, where the accumulation of fines decreases the tensile strength (Megally et al., 2003[12]). Based on this observation, a decreased γ_1 *factor of 1.2* $(h/12)^{-0.15}$ *is justified.*

A new fourth paragraph will be added to Commentary C5.6.3.3 that reads:

The flexural cracking stress of concrete members has been shown to decrease with increasing member depth. Sritharan et al. (2019[2]) observed that the flexural cracking strength is proportional to $h^{-0.15}$*. A similar equation for estimating the flexural cracking strength on the basis of the depth is found in* fib Model Code 2010 for Concrete Structures.[13]

Agenda Item 39

PCI has published the *Recommended Practice to Assess and Control Strand/Concrete Bonding Properties of ASTM A416 Prestressing Strand*,[14] which includes κ factors for the transfer length equation from NCHRP Report 603, *Transfer, Development, and Splice Length for Strand/Reinforcement in High-Strength Concrete*.[15] The recommended κ factors are 0.8, 1.0, and 1.6. The 0.8 κ factor is used as a lower bound for evaluating end region stresses. The 1.0 and 1.6 κ factors are based on high-bond strand and standard-bond strand, respectively. The PCI recommended practice establishes ASTM A1081[16] minimum values for standard-bond and high-bond strand. The standard-bond strand is considered as the strand typically used in pretensioned applications. The PCI recommended practice does not explicitly include the κ factors within the development length equation. This agenda item incorporates the latest update to the PCI recommended practice, which includes resolution testing. Agenda Item 39 makes the following changes.

Article 5.9.4.3.1 is revised, and a new seventh paragraph is added to read as follows:

In determining the resistance of pretensioned concrete components in their end zones, the gradual buildup of the strand force in the transfer and development lengths shall be considered.

The stress in the prestressing steel may be assumed to vary linearly from zero at the point where bonding commences to the effective stress, f_{pe}*, at the end of the transfer length.*

Between the end of the transfer length and the development length, the strand stress may be assumed to increase linearly, reaching the stress at nominal resistance, f_{ps}*, at the development length.*

For the purpose of this article, the transfer length may be taken as 60 strand diameters and the development length shall be taken as specified in Article 5.9.4.3.2.

The effects of debonding shall be considered as specified in Article 5.9.4.3.3.

The provisions of Article 5.9.4.3 may be used for design concrete compressive strengths specified in Article 5.1, including normal weight concrete with design concrete compressive strengths up to 10.0 ksi at transfer (f'_{ci}*) and up to 15.0 ksi for design (*f'_c*).*

The resolution testing methods articulated in PCI's Recommended Practice to Assess and Control Strand/Concrete Bonding Properties of ASTM A416 Prestressing Strand *(2025) shall be used to directly evaluate the bond quality of strands. The refined transfer and development length expressions in PCI Recommended Practice are intended for use in resolution testing and data analysis, and they shall not be used in lieu of a transfer length of 60 strand diameters and development length given in Equation 5.9.4.3.2-1.*

Commentary C5.9.4.3.1 is revised as follows:

Between the end of the transfer length and development length, the strand stress grows from the effective stress in the prestressing steel after losses to the stress in the strand at nominal resistance of the member.

The extension of the transfer and development length provisions to normal weight concrete with design concrete compressive strengths up to 10.0 and 15.0 ksi for f'_{ci} *and* f'_c*, respectively, is based on the work presented in NCHRP Report 603 (Ramirez and Russell, 2008).*

PCI's Recommended Practice to Assess and Control Strand/Concrete Bonding Properties of ASTM A416 Prestressing Strand *(2025) establishes ASTM A1081 minimum average values for standard-bond strand and high-bond strand. Standard bond strands are deemed to satisfy the performance expectations for bridge pretensioned members except for cases where limited internal redundancy provided by strands is critical. The anchorage of the tension force specified in Article 5.7.3.5 is critically important, particularly for members with a large percentage of debonded strands.*

ASTM A1081 testing involves the use of a surrogate material. The testing is intended to demonstrate to the precast concrete producer that the QC [quality control] practices of the strand manufacturer are consistent and repeatable. In addition, the PCI's Recommended Practice provides methods for assessing transfer and development length through testing in production concrete and using refined calculation methods. In the event that results representative from QC and QA [quality assurance] testing contradict each other, the resolution testing articulated in the PCI's Recommended Practice and the associated transfer and development length expressions may be used by the Owner for evaluation and making decisions.

The concrete mixture designs used at a precast concrete plant have been demonstrated to influence the bond behavior of the strands in precast concrete products as discussed in the PCI Recommended Practice. The resolution testing methods described in PCI's recommended practice directly and accurately benefit from the use of actual bond behavior between the strands and surrounding concrete in pretensioned members.

The implementation of this agenda item is covered in a Concrete Bridge Technology article in the Fall 2025 issue of *ASPIRE*. In addition, the August 20, 2025, webinar hosted by the National Concrete Bridge Council (NCBC) focused on this item. The recording of the webinar can be accessed through the NCBC website (https://nationalconcretebridge.org/webinars).

Agenda Item 40

The AASHTO LRFD specifications do not explicitly address the structural design of prestressed concrete piles. Instead, the specifications rely on provisions originally developed for the design of reinforced concrete compression members. This agenda item emphasizes that prestressed concrete piles represent a specialized type of compression member and therefore their designs can benefit from boundary conditions specific to such deep-foundation members. The unsupported portion of a foundation pile is an extension of the supported portion, which may be several times longer than the unsupported length. Thus, such a pile is deeply embedded into the ground and at some depth can be considered fixed against translation and rotation. Many static load tests have been performed on long, slender piles that were driven through very soft material and then into firm soil. These tests show that even very soft soils provide lateral restraint and tend to prevent buckling. Other differences between prestressed concrete piles and typical reinforced concrete columns include the effects of prestressing on cracking behavior under flexural loads, the degree of redundancy between piles in a footing and that in a column, and the determination of an effective column length for buckling considerations. The modifications introduced in Agenda Item 40 address these issues and directs the user to follow ANSI/PCI 142-24, *Specification for Precast, Prestressed Concrete Piles*,[17] as discussed herein.

Article 5.11.3.2.4 is revised as follows:

For piles extending above grade, the top of pile confinement length shall be taken as defined in Article 5.11.4.5.2., and the bottom-of-pile confinement length shall be the upper 20 ft of the pile below grade or the segment of *the pile from grade to the location of maximum moment below grade plus three pile dimensions, whichever is greater. For fully embedded piles, the confinement length shall be the upper 20 ft of the pile or the segment from the top of the pile to the location of maximum moment below grade plus three pile dimensions, whichever is greater. The pile dimension shall be taken as the outside diameter for round piles, and the perpendicular distance between flat surfaces for square and hexagonal piles.*

For piles using a circular prestressed reinforcement configuration, the ratio of spiral or hoop reinforcement, ρ_s*, within the confinement length shall not be less than:*

$$\rho_s = 0.04\left(\frac{f'_c}{f_{yh}}\right)\left(2.8 + \frac{2.3P_u}{f'_c}\right) \qquad (5.11.3.2.4\text{-}1)$$

For piles with a square prestressed reinforcement configuration, the total area of confinement reinforcing provided in each orthogonal direction, A_{sh}*, within the confinement length shall not be less than:*

$$A_{sh} = 0.03sh_c\left(\frac{f'_c}{f_{yh}}\right)\left(2.8 + \frac{2.3P_u}{f'_c}\right) \qquad (5.11.3.2.4\text{-}2)$$

Maximum spacing of transverse reinforcement within the confinement length shall be the minimum of 1/5 times the pile dimension, six times the strand diameter, or 4 in.

For the remaining length of pile outside the confinement length, ρ_s *and* A_{sh}*, shall not be less than half that required within the confinement length.*

where:

A_{sh} = *total cross-sectional area of transverse reinforcement provided separately in each direction, including cross-ties where applicable (in.²)*

h_c = *cross-sectional dimension of pile core measured out-to-out of square tie reinforcement (in.)*

f_{yh} = *specified yield strength of transverse reinforcement (ksi)*

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

P_u = *factored axial load on pile, as determined using all load combinations that include the effect of seismic load,* EQ, *per Table 3.4.1-1 (kips)*

s = *pitch or spacing of transverse reinforcement measured along the length of the pile (in.)*

ρ_s = *ratio of spiral or hoop reinforcement to total volume of pile core (5.6.4.6)*

A new Article, Article 5.11.4.5.6, is added as follows:

5.11.4.5.6 Precast Prestressed Piles

For piles extending above grade, the top of pile confinement length shall be taken as defined in Article 5.11.4.5.2, and the bottom-of-pile confinement length shall be the upper 35 ft of the pile below grade or the segment of the pile from grade to the location of maximum moment below grade plus three pile dimensions, whichever is greater. For fully embedded piles, the confinement length shall be the upper 35 ft of the pile or the segment from the top of the pile to the location of maximum moment below grade plus three pile dimensions, whichever is greater. The pile dimension shall be taken as the outside diameter for round piles, and the perpendicular distance between flat surfaces for square and hexagonal piles.

For piles using a circular prestressed reinforcement configuration, the ratio of spiral or hoop reinforcement, ρ_s*, within the confinement length shall not be less than:*

$$\rho_s = 0.06\left(\frac{f'_c}{f_{yh}}\right)\left(2.8 + \frac{2.3P_u}{f'_c}\right) \qquad (5.11.4.5.6\text{-}1)$$

For piles with a square prestressed reinforcement configuration, the total area of confinement reinforcing provided in each orthogonal direction A_{sh}*, within the confinement length shall not be less than:*

$$A_{sh} = 0.04sh_c\left(\frac{f'_c}{f_{yh}}\right)\left(2.8 + \frac{2.3P_u}{f'_c}\right) \qquad (5.11.4.5.6\text{-}2)$$

Maximum spacing of transverse reinforcement within the confinement length shall be the minimum of 1/5 times the pile dimension, six times the strand diameter, or 4 in.

For the remaining length of pile outside the confinement length, ρ_s *and* A_{sh}*, shall not be less than half that required within the confinement length.*

where:

A_{sh} = *total cross-sectional area of transverse reinforcement provided separately in each direction, including cross-ties where applicable (in.²)*

h_c = *cross-sectional dimension of pile core measured out-to-out of square tie reinforcement (in.)*

f_{yh} = *specified yield strength of transverse reinforcement (ksi)*

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

P_u = *factored axial load on pile, as determined using all load combinations that include the effect of seismic load,* EQ, *per Table 3.4.1-1 (kips)*

s = *pitch or spacing of transverse reinforcement measured along the length of the pile (in.)*

ρ_s = *ratio of spiral or hoop reinforcement to total volume of pile core per (5.6.4.6)*

The last paragraph of Article 5.12.9.3.2 is revised as follows:

The full length of longitudinal steel shall be enclosed with spiral reinforcement or equivalent hoops. The spiral reinforcement shall be as specified in Article 5.12.9.4.4.

A new Article 5.12.9.4.1 is added as follows:

5.12.9.4.1 General

Precast prestressed concrete piles shall be designed, detailed, and installed using provisions of PCI 142-24, Specification for Precast, Prestressed Concrete Piles, *which is incorporated by reference, unless those provisions are specifically amended herein.*

A new commentary section, C5.12.9.4, will read:

C5.12.9.4.1

PCI 142-24 specification for precast prestressed concrete piles uses pound units instead of the kip units used in the AASHTO LRFD Bridge Design Specifications.

Excerpts from PCI 142-24 have been provided in this Article for emphasis. The absences herein of any provisions from PCI 142-24 do not negate their validity as part of that specification.

PCI 142-24, Section 3.2.2.1 implies that piles with compressive loads exceeding 40 tons require load testing. PCI 142-24, Section 4.7.4.1 imposes a compressive load limit on piles in moderate and high seismic risk zones.

Article 5.12.9.4.1 is renumbered Article 5.12.9.4.2, Pile Dimensions. Article 5.12.9.4.2 is renumbered as Article 5.12.9.4.3, Concrete Quality. Article 5.12.9.4.3 is renumbered as Article 5.12.9.4.4, Reinforcement, and the associated commentary is renumbered as C5.12.9.4.4.

The new Article 5.12.9.4.4 is as follows:

5.12.9.4.4 Reinforcement

Unless otherwise specified by the Owner, the prestressing strands should be spaced and stressed to provide a uniform compressive stress on the cross-section of the pile per Table 5.12.9.4.4-1 at the service limit state:

Table 5.12.9.4.4-1—Minimum Effective Prestress in Piles

Pile Length, *L*	Minimum Effective Prestress
$L < 30$ ft	0.40 ksi
30 ft $\leq L < 50$ ft	0.55 ksi
$L \geq 50$ ft	0.70 ksi

The full length of the prestressing strands shall be enclosed with spiral reinforcing as follows:

For piles not greater than 24.0 in. in diameter:

- *spiral wire not less than W3.5;*
- *spiral reinforcement at each end of piles shall be five turns at 1.0-in. pitch followed by 16 turns at 3.0-in. pitch; and*
- *for the remainder of the pile, the spiral reinforcement shall not have more than 6.0-in. pitch.*

For piles greater than 24.0 in. in diameter:

- *spiral wire not less than W4.0;*
- *spiral reinforcement at each end of piles shall be four turns at 1.5-in. pitch followed by 16 turns at 2.0-in. pitch; and*
- *for the remainder of the pile, the spiral reinforcement shall not have more than 4.0-in. pitch.*

For piles in Seismic Zone 2, reinforcing in the confinement zone shall be as specified in Article 5.11.3.2.4. For piles in Seismic Zones 3 and 4, reinforcing in the confinement zone shall be as specified in Article 5.11.4.5.6.

The new Commentary C5.12.9.4.4 will read:

The purpose of the minimum compression is to prevent cracking during handling and installation. A lower compression may be used if approved by the Owner.

A new Article 5.12.9.4.5 is added as follows:

5.12.9.4.5 Performance-Based Design

Where performance-based design is used as the design methodology for prestressed piles, Section 4.8 of PCI 142-24, Specification for Precast, Prestressed Concrete Piles *shall be used as the basis of design.*

Concluding Remarks

The three agenda items discussed in this article and four others discussed in my Fall 2025 and Spring 2026 *ASPIRE* articles that followed the June 2025 COBS meeting will be incorporated in the forthcoming 11th edition of the AASHTO LRFD specifications. An additional agenda item that was also approved in the same meeting will be included in the forthcoming fourth edition of the *Manual for Bridge Evaluation*.[18] (For discussion of that item, see the LRFD article in the Winter 2026 issue of *ASPIRE*.) The next COBS meeting will take place from June 28 to July 2, 2026, in Charlotte, N.C. I look forward to reporting on the new developments and ballot items that are approved at that meeting.

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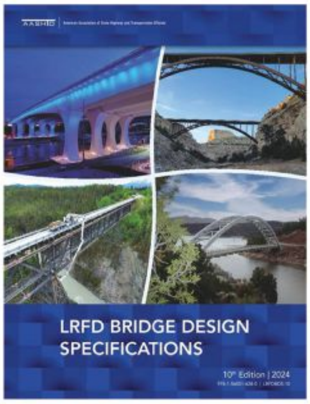

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