ASPIRE®

THE CONCRETE BRIDGE MAGAZINE

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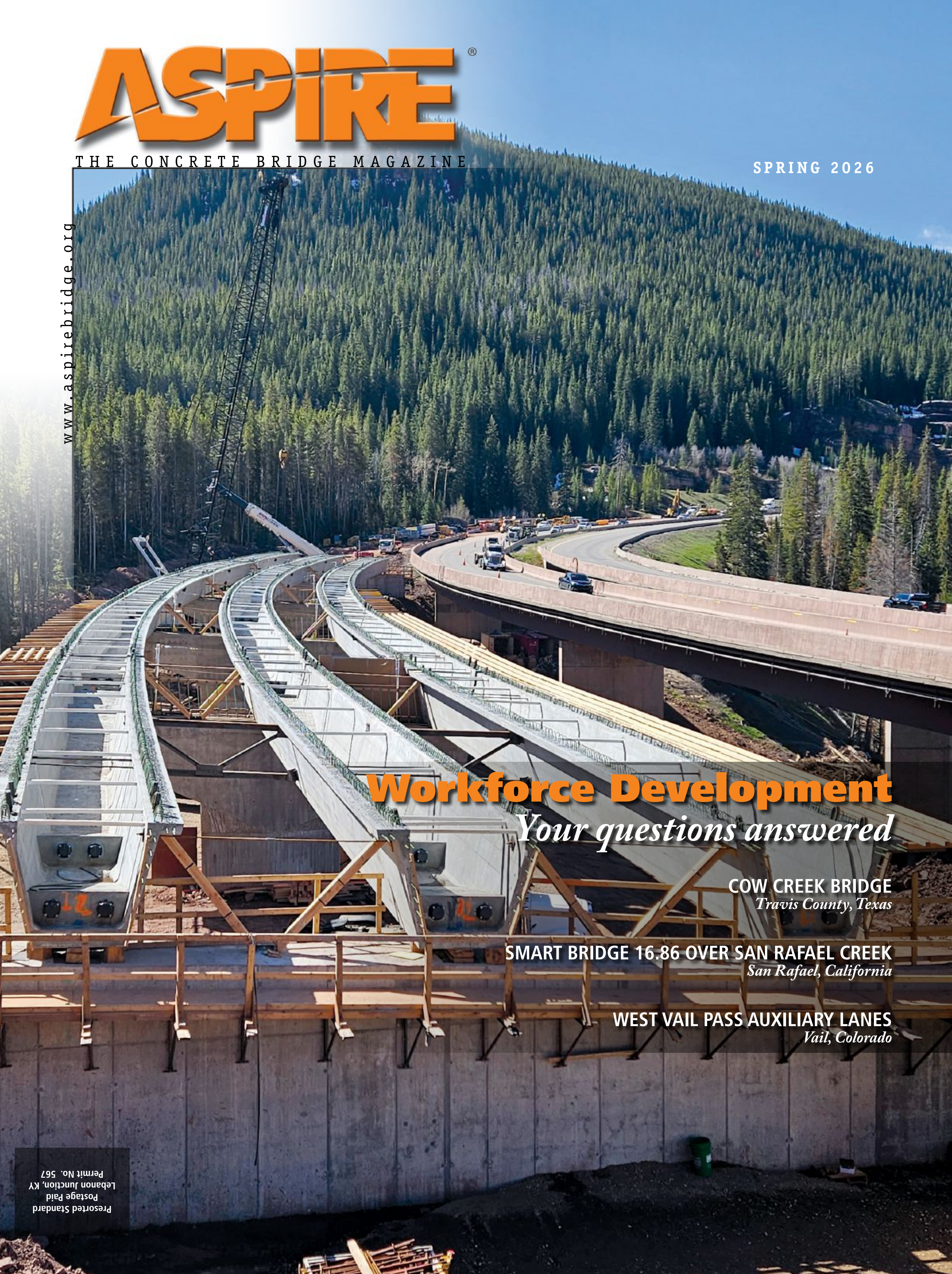

Workforce Development
Your questions answered

COW CREEK BRIDGE
Travis County, Texas

SMART BRIDGE 16.86 OVER SAN RAFAEL CREEK
San Rafael, California

WEST VAIL PASS AUXILIARY LANES
Vail, Colorado

Presorted Standard
Postage Paid
Lebanon Junction, KY
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Photo: Texas Department of Transportation.

Photo: Stacy Witbeck/Herzog.

Photo: Stacy Witbeck/Herzog.

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Photo: Texas Department of Transportation.

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READER FEEDBACK

The podcasters from *The Texas Slab Guys* podcast recently offered unique feedback about "A Crack Is Not a Crack: Torsional Cracking" by Dr. Oguzhan Bayrak, which appeared in the Fall 2025 issue of *ASPIRE*®. One of the podcasters said,

> "We spend a lot of time in the field thinking about crack patterns, load paths, and real-world structural behavior, so your article really resonated with us. We enjoyed it enough that we put together a short podcast discussion reflecting on the piece—focused on translating the ideas into practical, field-level understanding while staying true to the core message. Thank you for the work you continue to publish. It's refreshing—and genuinely helpful—to see nuanced discussions like this grounded in first principles and real structural behavior."

The podcast is available at https://youtu.be/ZbAp8Pd1q8U

EDITORIAL

Photo: PCI.

Mentorship—We Must Make It Common Practice Immediately

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

Lately, I've had a bit of time to reflect upon the folks who took the time to make a positive impact in both my professional and personal worlds. I strongly believe that there is a call to action here, partly due to how the COVID pandemic and social media seem to have altered our social norms. We need to revive our commitment to mentorship.

I've had many mentors: the college professors who taught engineering and how to look at things; the senior engineers in the Florida Department of Transportation's bridge department; E. Carl Driver, the man behind E. C. Driver and Associates (the first private bridge company I worked for); and all the members and staff (current and past) I have known since joining PCI. As I think back on what I've gained from my interactions with these exemplary people, I am concerned that such mentorship opportunities are not readily available for early- and mid-career professionals today.

None of us succeed alone. We all need someone in a mentorship position to take an interest in us, see something in us that perhaps others missed, and put their energy into our professional growth and well-being. The career journey of someone in our industry is one of interactions, conversations, and collaborations with highly knowledgeable, dedicated, and inspiring professionals. Most are incredibly passionate about their perspectives and approaches, me included (that's somewhat of an understatement for those that know me).

I briefly touched on the role of artificial intelligence (AI) in bridge design and construction in my last editorial. While I believe AI offers several benefits, I also deeply believe that mentorship is a critical ingredient to our success. Professionals and code officials need to carefully look at AI results for fatal flaws (strength and service) and detailing changes that put our past deemed-to-satisfy specifications out of kilter. That critical thinking may come through as a mentoring moment in your shop.

What does it take to become an effective mentor? Is there a course or online program? In actuality, mentorship is simple. You just need to engage, listen, and allow the person being mentored to navigate their way and ask questions.

There are plenty of opportunities for mentors to provide feedback and direction while showing and demonstrating "what right looks like." Your mentorship approach can be formal or informal; both are highly effective.

Mentorship is about putting people in a position to be successful and allowing them to work through difficult situations. It's about discussing mistakes and approaches. It's about engineering and the business side of the company (how the bills get paid, how to make payroll, why we need insurance). It's about finding a healthy work-life balance (something we never talked about back in the day).

Above all, mentorship is about connecting. Recently, I heard someone say, "relationships matter." That thought rang true recently while I was flying out of Chicago, where I happened to run into a governmental engineer I have known for years. As we talked, he thanked me for taking the time to help guide him along his bridge engineering journey. He said that he had initially thought I was a bit of a know-it-all, and close minded, but his perspective has changed as he nears retirement. He has even passed along a few of the pearls of wisdom I'd shared those many years ago. I told him that I had no ownership of those pearls—they'd been passed to me 40 years ago by a fabulous mentor who took the time to see that I got started on the right path.

Mentorship, rooted in our upbringings and our successes and failures, is also about the future. By responding to my call to action, investing your time, and becoming a mentor (and encouraging others to do the same), you can show your support of the industry. And who knows? Maybe you'll pass on a few pearls of your own.

PS: For more information on wellness in the workplace, check out the ad on page 55.

Editor-in-Chief
William N. Nickas • wnickas@pci.org

Managing Technical Editor
Dr. Richard Miller

Technical Editors
Monica Schultes, Emily Lorenz, Dr. Krista M. Brown

Program Manager
Trina Brown • tbrown@pci.org

Associate Editor
Angela Tremblay • atremblay@pci.org

Copy Editor
Elizabeth Nishiura

Layout Design
Walter Furie

Editorial Advisory Board
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Cover
Designing the twin bridges of the Interstate 70 West Vail Pass Auxiliary Lanes required innovative ideas and collaborative solutions from the design team, owner, and stakeholders. Photo: Kevin Petrillo.

Ad Sales
Scott Cunningham • scunningham7@aol.com
(678) 576-1487 (mobile)
(770) 913-0115 (office)

Reprints
lisa scacco • lscacco@pci.org

Publisher
Precast/Prestressed Concrete Institute
Bob Risser, President

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COMING SOON! THREE-PART WEBINAR SERIES
Anchoring to Concrete Updates for 2026

The 10th edition of the *AASHTO LRFD Bridge Design Specifications* includes significant revisions to the provisions for concrete anchorage in structures. It is critical for engineers to adopt these updates to enhance structural safety and comply with the latest design standards.

SHEAR LUG

Courtesy of Dr. P.J. Carrato

SCREW ANCHOR

Courtesy of Simpson Strong-Tie

POST-INSTALLED REINFORCING BARS

Courtesy of Hilti

NCBC will host a three-part webinar series, "Anchoring to Concrete Updates for 2026," which will discuss procedures to design, detail, and install anchors as they apply to highway bridges. The webinars will be presented by Bahram Shahrooz, PhD, professor emeritus of structural engineering at the University of Cincinnati.

The first webinar will be held on May 20 at 1 p.m. ET.

For more information, visit the NCBC website: nationalconcretebridge.org/webinars.

CONTRIBUTING AUTHORS

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.

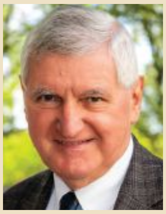
Doug Beer is a research engineer at the University of Texas at Austin.

Richard Brice is the bridge design technology unit manager with the Washington State Department of Transportation Bridge and Structures Office in Olympia, Wash.

Gregory Clauson is the Northeast Region manager and National Transportation manager for the Concrete Reinforcing Steel Institute.

Chad Hammond is a senior project manager with RS&H in Denver, Colo.

Greg Hunsicker is a research engineer at the University of Texas at Austin and deputy director of the Concrete Bridge Engineering Institute.



Dr. Andrzej S. Nowak is the Brasfield and Gorrie Professor in the Department of Civil and Environmental Engineering at Auburn University.

Dr. Karina Popok is an engineering technician at Gresham Smith. Her doctoral work focused on reliability-based calibration for the design and load rating of segmental bridges.

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.

April 13–16, 2026
CRSI Spring Business and Technical Meeting
Silverado Resort
Napa, Calif.

April 14, 2026
TRANS-IPIC UTC Workshop
Big Ten Office and Conference Center
Rosemont, Ill.

April 23–24, 2026
NCBC Prestressed Concrete Bridge Seminar
Austin Marriott South
Austin, Tex.

May 4–7, 2026
PTI Convention
Westin Long Beach
Long Beach, Calif.

June 15–17, 2026
International Bridge Conference
Gaylord National Resort
National Harbor, Md.

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

October 11–14, 2026
ACI Concrete Convention
Hilton Atlanta
Atlanta, Ga.

October 15–18, 2026
NRMCA ConcreteWorks 2026
Gaylord Opryland
Nashville, Tenn.

November 8–11, 2026
ASBI Annual Convention and Committee Meetings
Grand Hyatt Riverwalk
San Antonio, Tex.

November 15–18, 2026
CRSI Fall Business and Technical Meeting
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.

Workforce Development: Your Questions Answered

As engineering firms and agencies anticipate talent shortages, we discuss strategies on recruitment, retention, and long-term workforce stability with Donn Digamon, PE, state bridge engineer with the Georgia Department of Transportation.

by Monica Schultes

The infrastructure industry is confronting an industrywide labor squeeze that shows no sign of easing. Donn Digamon, PE, state bridge engineer with the Georgia Department of Transportation (GDOT), discusses how to drive meaningful change in workforce development. Answers have been edited for brevity and clarity.

Q: What is your general assessment of the development of the (industry) workforce?

A: When I was a group leader at GDOT, my supervisory position provided me with insight into our staff disparity. Many entry-level engineers were entering the department while 30-year veterans were planning to retire. I could see the "perfect storm" brewing, where there would be a gap with no established conduit or structure to transfer knowledge or train new employees. It is easy to overlook trends like this, and despite the industry's progress, there is a visible knowledge gap. Employers across the industry are searching for "unicorns" with 10 to 15 years of experience to fill the experience divide. We use the term "unicorn" because it is rare to find people with the right combination of experience and education. There is a limited number of engineers who can take on a project with little supervision and deliver on time, on scope, and with safety at the forefront. A common complaint is that we value our bright and talented young engineers, but they lack experience and exposure to all facets of the planning, design, construction, and maintenance of a bridge. DOTs [Departments of Transportation] are developing the pipeline by establishing mentors for technical areas involving bridges, and encouraging critical thinking from new hires. Just like asset management and preservation, we need to take better care of our human resources. We need to capture that body of knowledge from those nearing retirement age and transfer it to the next generation of bridge professionals.

Q: Is the labor shortage more critical now? Is there a greater need in the concrete bridge industry for engineers or for skilled laborers, DOT inspectors, technicians, and others?

A: The labor shortage is still critical across all vocations, even more so than a decade ago. We are in dire need to fill all roles in the industry, including designers, construction inspectors, contractors, engineers, fabricators, decision-makers with technical experience, the list goes on and on. There is a need to have an established pipeline to generate a consistent workforce devoted to the entire bridge design, construction, and maintenance process. A bridge project is a team effort, and when any part of that process is lacking, that will add an actual cost to the job. We especially need folks who want to be out in the field. Construction has its challenges, but the reward is being a part of the ultimate build.

Georgia Department of Transportation engineers-in-training visit a jobsite for hands-on experience as part of ongoing efforts to attract, train, and retain strategic thinkers committed to the transportation industry. All Photos: Georgia Department of Transportation.

The Federal Highway Administration's Strategic Workforce Development Program offers tools such as the *Identify, Train, Place* playbook[1] to help state transportation agencies recruit and train a diverse workforce.

Q: The Federal Highway Administration (FHWA) summarizes the key steps in resolving the labor shortage as "identify [potential employees], train, place, and retain."[1] Which aspect requires further attention?

A: I think all areas need work. The reality is that employees have so many priorities, which detracts from a focus on workforce development and knowledge transfer. Engineers, especially those early in their careers, need practical training in essential professional skills tailored to infrastructure projects. The FHWA initiative provides an overarching effort to establish methods to identify, train, place, and avoid redundancy. The "identify, train, place" initiative signifies three legs of a stool—all are equally important.

Q: What approaches are state agencies using to attract more women, minorities, high school graduates, veterans, and second-chance candidates?

A: The industry needs to engage early and often through presentations at high schools, technical schools, universities, and internships. We can show people from all demographics how their contributions impact the community. DOTs share success stories and inspire others to create a road map to achieve comparable results. We need champions to help spread the word. Starting early is key to introducing engineering and construction vocabulary to children. Many do not know about the range of opportunities in the concrete bridge industry. A successful recruitment effort focuses on team players and critical thinkers. We are looking for people who can function as individuals and fit as part of the team.

"We need champions to help spread the word. Starting early is key to introducing engineering and construction vocabulary to children. Many do not know about the range of opportunities in the concrete bridge industry."

Q: How do we leverage technology in attracting, training, and developing our workforce?

A: We need to challenge young staff members to find ways to improve current processes with technology but still complete the work with safety in mind. While the latest technology can attract young engineers, in my opinion a state bridge office is often a stronghold of tradition and slow to change! So, we need to shift our corporate culture to welcome those who have different backgrounds or mindsets and be open to a better way of accomplishing tasks. Just as we need to constantly update computers and other hardware, we also need to update our expectations about our people.

Q: What resources are available to agencies and consultants for skill development?

A: Industry organizations like the American Concrete Institute, American Segmental Bridge Institute, Precast/Prestressed Concrete Institute, and others have updated their learning resources. Training opportunities include seminars, webinars, and networking. I think there needs to be a greater emphasis on bridge-focused academic curriculum and training because they are not included in a typical university learning pathway.

Q: How do we inspire more universities to teach bridge engineering and construction topics?

A: Some champions in this arena of bridge-focused learning are the civil engineering programs at Purdue University led by Dr. Robert Connor and California State University, Sacramento, led by Dr. Eric Matsumoto. These two programs expose students to opportunities in the concrete bridge industry. (For more information, read the Professor's Perspective in the Spring 2023 issue of *ASPIRE*®.)

Q: Workforce development has been a hot topic for years. What have we learned and which strategies are working?

A: We need to focus on being consistent and persistent. If we are charting the course on a different path, we cannot be distracted. Focusing on attracting potential candidates and training current employees need to be prioritized. If there is no pipeline of critical thinkers joining your staff, your agency or firm will have a tough time completing the work. So, we need to be consistent and committed in every phase from recruitment to mentorship. An important lesson is to establish buy-in from leadership to ensure that the strategies are supported. Another lesson is to avoid sacrificing quality for quantity. I strongly believe that we should not lower the bar of professionalism and standards to increase the number of candidates and employees. We need to find individuals with sound judgment. But we also need to look further afield. For example, our asset management group found that industrial engineers bring a unique perspective and expertise despite not being trained specifically about bridges.

Q: What are some of the key factors that result in a new hire staying (or leaving)?

A: A big challenge is getting talent to stay. Key factors include providing a safe yet challenging environment for learning and advancement and making mentors available. Industry professionals look for job stability, purpose-driven work, and professional growth. It comes down to recognizing individuals and their different strengths rather than taking a one-size-fits-all approach. We encourage mentorship moments wherever and whenever they happen. Today's workforce can span several generations, each with its own communication style. It is important to discover ways to bridge gaps and foster understanding across those divides. Newer employees need to step up, but older generations need to take them by the hand to pull them up. Be a mentor and pay it forward because at one point in our careers, someone had an impact on us and that made all the difference.

Conclusion

As investment in infrastructure grows so will the demand for the bridge industry workforce. The labor shortage finds agencies and firms competing for talent as they recruit early-career engineers and seek to maintain midlevel staff capacity. The next generation of critical thinkers and problem solvers is waiting for the industry to reach out and engage them as they identify potential career paths. The industry continues to share its success stories about recruiting diverse demographics, improving corporate culture, and incorporating innovative training programs. (For more information, read the Perspective article on workforce development and recruitment methods in the Summer 2024 issue of *ASPIRE*.)

Training programs for project managers incorporate practical applications such as drones used for quality control, and documenting construction progress. Using new technology encourages new staff to be innovative.

Reference

1. Federal Highway Administration (FHWA). 2021. *Identify, Train, Place: A Playbook to Build Tomorrow's Highway Construction Workforce*. Washington, DC: FHWA. https://www.fhwa.dot.gov/innovativeprograms/centers/workforce_dev/hcwp/playbook_and_products/default.aspx.

Open BIM Standards: A Key to Digital Delivery for Bridge Design

Open BIM standards for bridges are a foundational shift toward transparent and efficient digital project delivery.

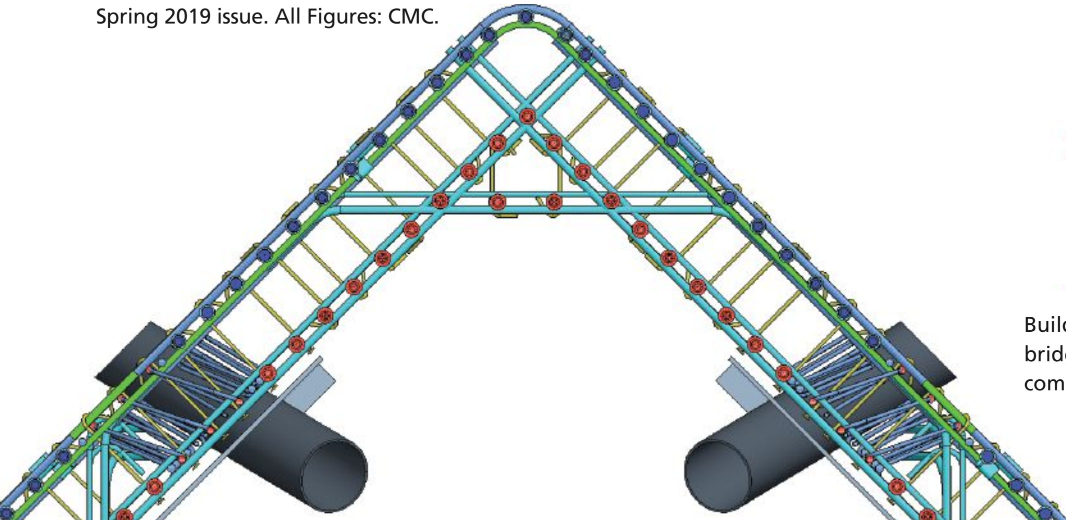
by Gregory Clauson, Concrete Reinforcing Steel Institute

Building information modeling (BIM) for bridges is reaching a pivotal moment. A pilot project in Pennsylvania is demonstrating that BIM models are no longer just references; they can also be contractual documents. This transformation is driven by open standards and promises to reshape the way that contractors, engineers, transportation agency officials, fabricators, and owners approach bridge delivery.

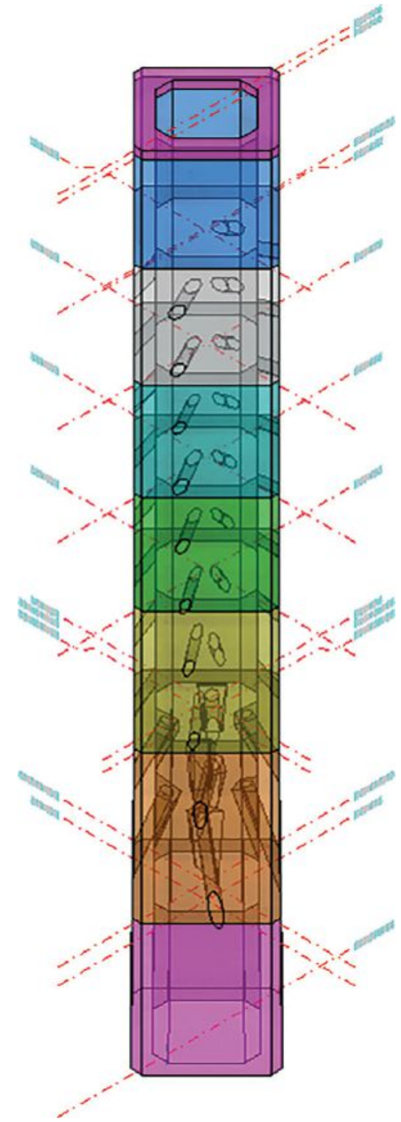
Open BIM Standards in Action: The Pennsylvania Department of Transportation Pilot

The Pennsylvania Department of Transportation's (PennDOT's) U.S. Route 6 (U.S. 6) French Creek Parkway Bridge #3 project made history by including an Industry Foundation Classes (IFC) model of reinforcing bar as a legal contract deliverable. This bridge replacement project is notable because a three-dimensional (3-D) model, and not just two-dimensional (2-D) drawings, is a contractual requirement. IFC is an open, standardized BIM format that ensures compatibility across all software platforms.[1] This shift to BIM allows project participants to use the tools they prefer while maintaining consistent, transparent, and accessible data.

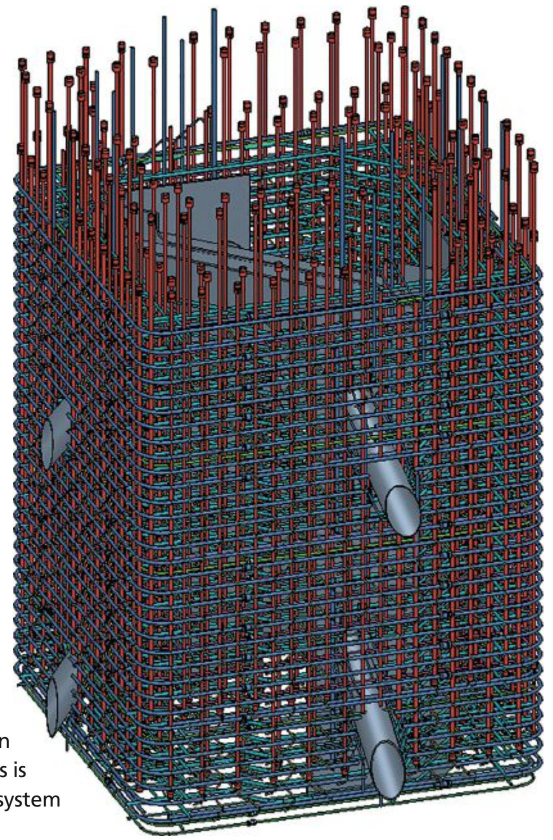
Of interest to the Concrete Reinforcing Steel Institute is that the project's BIM model incorporates not only the 3-D geometry of the reinforcement but also metadata such as reinforcing bar sizes, grades, spatial relationships, material properties, and pay-item information. This comprehensive dataset empowers contractors to automate material takeoffs, generate reinforcing bar lists, and perform accurate planning.



Corner detail of the Long Beach International Gateway–Gerald Desmond Bridge project is generated by building information modeling for bridges. The Gerald Desmond Bridge is discussed in the Spring 2021 issue of *ASPIRE*® and is the subject of a Project article in the Spring 2019 issue. All Figures: CMC.



Building information modeling for bridges can be used for assembly of components during construction.



Building information modeling for bridges is used to show cable system interaction.

Fabricators benefit from seamless translation of BIM model data into shop production, and field teams can integrate BIM models with digital layout tools.

While 2-D plan sheets are part of the project deliverables, the reinforcing bar detailing, coordination, and related communication in this pilot project are all entirely model based. This arrangement is a leap toward a digital-first future that reduces interpretation errors, shortens shop drawing cycles, and enhances project coordination.

Why Open BIM Standards Matter

The strategic use of open BIM standards in bridge projects ensures interoperability across all phases of a project. When models are shared using IFC, the same dataset can move into detailing platforms, cost estimation software, scheduling tools, robotic layout systems, and facility management platforms with far less information degradation than traditional file-conversion workflows. In practice, "translation loss" happens when BIM model geometry and metadata (objects, properties, classifications, and IDs) must be remapped during repeated exports and imports between platforms or proprietary formats, which can cause data to be dropped, simplified, or reinterpreted. IFC reduces that risk by providing a consistent, standardized structure for exchanging model content.

The use of open BIM standards enables automation at every stage of the project, from estimating and procurement to fabrication and field installation. It also reduces the risk of a phenomenon known as vendor lock-in, where it is difficult and expensive to switch from one software vendor to another. Furthermore, the use of open standards allows small and midsize firms to participate in digital delivery without the burden of purchasing proprietary software.

From a life-cycle perspective, IFC models serve as enduring digital assets. Bridges often exceed a 75-year lifespan, and their data must remain accessible and usable across decades of inspections, maintenance, and retrofits. An open-standard model assists in ensuring that asset information remains transferable and reliable across different systems and owners.

Policy Momentum: AASHTO and Federal Support

This shift in delivery methods aligns with national and federal policy movements. In 2019, the American Association of State Highway and Transportation Officials (AASHTO) adopted Administrative Resolution AR-1-19,[2] designating IFC as the national standard for exchanging electronic engineering data. AASHTO also established the Joint Subcommittee on Data Standardization (J-STAN) to coordinate BIM-related activities across all state transportation agencies.

Meanwhile, the Federal Highway Administration launched the Accelerated Implementation and Deployment of Advanced Digital Construction Management Systems (ADCMS) initiative. With $100 million in funding through the Infrastructure Investment and Jobs Act, ADCMS supports states adopting digital tools such as BIM. PennDOT's pilot project is part of this program.

Preparing for the Shift

The following are five actionable steps that stakeholders can take now to prepare for the digital future of bridge delivery:

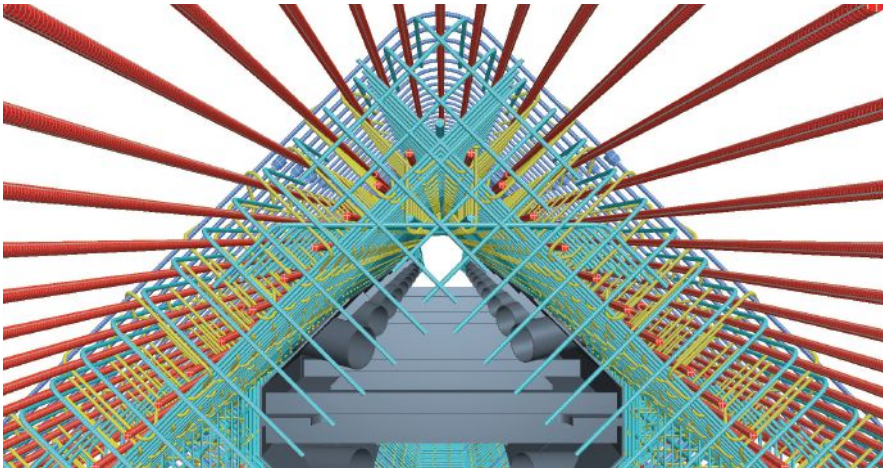
Adopt Industry Foundation Classes (IFC)–compatible tools: Evaluate software tools for compliance with IFC standards. Upgrade or integrate solutions that support open data exchange.

Participate in standards development: Engage with the American Concrete Institute, Concrete Reinforcing Steel Institute, Precast/Prestressed Concrete Institute, Post-Tensioning Institute, and National Concrete Bridge Council to influence emerging model content standards and implementation guides.

Collaborate early: Clarify expectations around model use, level of detail, and responsibilities at project kickoff. Effective collaboration facilitates smooth workflows and reduces downstream rework.

Monitor incentives and pilots: Keep track of Advanced Digital Construction Management Systems funding and agency-led pilot opportunities. Demonstrate your firm's readiness to participate in digital delivery projects.

Update contracts and training: Work with legal advisors to develop model-based contract templates. Train staff in digital modeling workflows, collaboration practices, and quality control.



This image produced by building information modeling for bridges shows the interaction of components in the Long Beach International Gateway–Gerald Desmond Bridge.

As more agencies pursue ADCMS grants, experience with digital delivery will become a competitive advantage. Contractors and suppliers that demonstrate proficiency in model-based delivery may be eligible for incentives, bonuses, or even preference in bidding.

Industry organizations such as the Association of General Contractors of America, American Council of Engineering Companies, and American Road and Transportation Builders Association have publicly endorsed digital delivery. This unified front suggests that model-as-legal-document practices will become increasingly common in public infrastructure contracts.

Stakeholder Benefits Across the Board

The application of open BIM standards in bridge delivery provides advantages for all stakeholders:

- Designers and engineers benefit because detailed models improve clarity of design, reduce the number of requests for information, and speed up the review process. Conflict (clash) detection and quantity verification can be automated directly from the model early in design, reducing rework before 2-D sheets are generated or finalized.
- Contractors benefit because model-based estimating, scheduling, and site layout tools simplify planning and execution. Enhanced accuracy in takeoffs helps reduce risk and increases competitiveness.
- Fabricators benefit because reinforcing bar detailers and precast concrete producers can extract fabrication data directly from the design model, ensuring that their products align with project requirements. Automated fabrication equipment can be driven from digital inputs, improving consistency and quality.
- Owners and transportation agencies benefit because accurate, comprehensive as-built models support asset management and long-term maintenance. Open standards also help protect data integrity throughout the life of the structure.
- Legal and procurement teams benefit because contracts tied to BIM models require clear definitions of scope, liability, and model ownership. While these requirements present new challenges, they ultimately enhance transparency and dispute resolution.

Embracing the Future of Bridge Construction

U.S. 6 French Creek Parkway Bridge #3 is more than a pilot project. It's a signal that the transformation to digital delivery in bridge construction is accelerating. Open BIM standards, supported by policy and funding, are redefining what it means to collaborate on infrastructure.

By investing in open standards, participating in development efforts, and preparing internal systems and teams, stakeholders across the bridge industry—from reinforcing steel fabricators to precast concrete producers—can ensure that they are ready to lead in this new era.

BIM based in open standards isn't just a technology; it's a strategic choice. Now is the time to build bridges that are not only strong and durable but also digitally connected for decades to come.

References

1. International Organization for Standardization (ISO). 2024. *Industry Foundation Classes (IFC) for Data Sharing in the Construction and Facility Management Industries—Part 1: Data Schema*. ISO 16739-1:2024. Geneva, Switzerland: ISO.
2. American Association of State Highway and Transportation Officials (AASHTO). 2019. *Adoption of Industry Foundation Classes (IFC) Schema as the Standard Data Schema for the Exchange of Electronic Engineering Data*. Administrative Resolution AR-1-19. Washington, DC: AASHTO. https://transportation.org/data/wp-content/uploads/sites/45/2023/12/AR-1-19-IFC-Schema-Resolution-Board-Adopted-FINAL.pdf.

EDITOR'S NOTE

Building information modeling (BIM) is transforming how bridge engineers design, deliver, and manage infrastructure, and this issue of ASPIRE *offers two perspectives that together illuminate both the conceptual foundation and the legal contract deliverable.*

The Perspective article by Richard Brice on page 11 provides essential context for readers who are still defining what BIM means for their practices. His article situates bridge BIM within the full asset life cycle—from design and construction through decades of operations and maintenance—and introduces the standards frameworks and national roadmap shaping how transportation agencies are approaching this transformation.

This Perspective article provides a real-world example to putting BIM into practice: a transportation project that incorporates BIM model as a legal contract deliverable. It examines what open BIM standards mean in practice for designers, contractors, fabricators, and owners, and offers concrete steps for stakeholders preparing to compete in an increasingly digital delivery environment.

Beyond 3-D Models: BIM for Bridge Project Delivery and Asset Management

by Richard Brice, Washington State Department of Transportation Bridge and Structures Office

Civil engineering software platforms have evolved from two-dimensional (2-D) line-drawing tools to data-rich, three-dimensional (3-D) model-authoring environments. For many bridge projects, the initial step into such an environment involves producing traditional 2-D plan sheets extracted from a coordinated 3-D model, where changes automatically propagate through plans, sections, and elevations. While this initial step improves document consistency and project coordination, it represents only an early phase of a much broader digital transformation.

As transportation agencies progress in their digital efforts, terms such as "digital delivery," "model as legal document," and "building information modeling" (BIM) become more prevalent. BIM is often equated with 3-D models, but this view minimizes its scope. International Organization for Standardization's (ISO's) standard 19650 defines BIM as the "use of a shared digital representation of a built asset to facilitate design, construction, and operation processes to form a reliable basis for decisions."[1] In this context, BIM includes both the digital representation and the processes governing how information is created, shared, and managed across an asset's life cycle. Three-dimensional models are an important component, but not the entirety of BIM.

Why Adopt BIM?

During project development and delivery, BIM improves communication of design intent in ways that traditional 2-D plan sheets cannot. Designers conceive projects, and contractors build them, in three dimensions, but we have traditionally communicated design intent in two dimensions. Three-dimensional models assist teams by enabling early identification of spatial conflicts, constructability issues, and sequencing challenges, thereby reducing risk and minimizing costly downstream changes.

BIM also supports efficient digital workflows. The use of models can shorten review cycles, improve accountability in issue tracking and resolution, and reduce reliance on paper-based processes. When combined with digital cost and schedule information, BIM enables teams to work efficiently and make informed decisions throughout design and construction.

Beyond project delivery, BIM provides significant value to transportation agencies as long-term asset owners. State departments of transportation (DOTs) are responsible not only for delivering projects but also for operating and maintaining transportation systems for many decades. These responsibilities require accurate, timely, and accessible asset information to support planning, maintenance, and investment decisions.

BIM provides significant value to transportation agencies as long-term asset owners.

BIM enables information generated during design and construction to flow into operations, maintenance, and asset management. When paired with life-cycle information management processes, such as those defined in ISO 19650, BIM reduces reliance on redundant data sources and supports consistent, authoritative asset information across an agency. Today, asset data are often collected and maintained within organizational silos, limiting visibility by other departments and requiring staff to gather the same information multiple times. This duplication wastes time and resources and exposes staff to unnecessary field risks. BIM processes help break down these silos by creating a "single source of truth"; this strategy reduces data management costs while supporting accurate asset analysis and well-informed decision-making.

BIM is an organization-wide endeavor spanning the full life cycle of transportation assets. While 3-D models are highly visible, they represent only one part of a larger information ecosystem.

The BIM Road Map

The process of adopting BIM across individual projects and throughout a transportation agency is not a simple transition. It requires a deliberate, phased approach supported by policy, guidance, training, and technology.

In June 2021, the Federal Highway Administration published *Advancing BIM for Infrastructure: National Strategic Roadmap*,[2] which outlines a 10-year vision for transitioning from traditional 2-D plan delivery to a fully digital approach encompassing design, construction, operations, and maintenance.

Two Transportation Pooled Fund (TPF) studies are further supporting this transformation. Approximately 25 state DOTs are participating in TPF-5(480), BIM for Infrastructure,[3] and TPF-5(523), BIM for Bridges and Structures, Phase II.[4] These efforts focus on developing practical guidance and specifications for BIM-based project delivery. One key outcome is the American Association of State Highway and Transportation Officials' *AASHTO Information Delivery Manual for the Design-to-Construction Data Exchange for Highway Bridges*,[5] which defines the minimum information requirements needed for bridge project letting. Each participating DOT is at a different point in their BIM transition, but all are making progress with support from the TPF efforts.

Adoption Challenges

Two primary challenges to BIM adoption are people and technology. On the people side, education, training, and communication are essential. BIM represents a fundamental shift in how information is produced and managed. Overcoming resistance to change requires sustained investment in workforce development and clear communication of benefits.

On the technology side, sharing information across the asset life cycle is challenging when stakeholders rely on different software platforms. Designers, contractors, owners, and operators use specialized tools tailored to their workflows, many of which rely on proprietary data formats. These formats often require custom solutions to exchange information, increasing cost and risk for stakeholders.

Open data standards are critical to addressing this challenge. AASHTO has adopted ISO 16739, *Industry Foundation Classes (IFC) for Data Sharing in the Construction and Facility Management Industries*,[6] which was developed by buildingSMART International in cooperation with the ISO as the preferred data exchange standard. IFC supports robust definitions of geometry, object properties, and relationships in an open, nonproprietary format. As an international, text-based standard, IFC also supports the long-term preservation of asset information for assets whose service lives can exceed 75 to 100 years.

Open standards enable interoperability between software platforms, support collaborative workflows, preserve flexibility in technology choices, and safeguard long-term access to asset information.

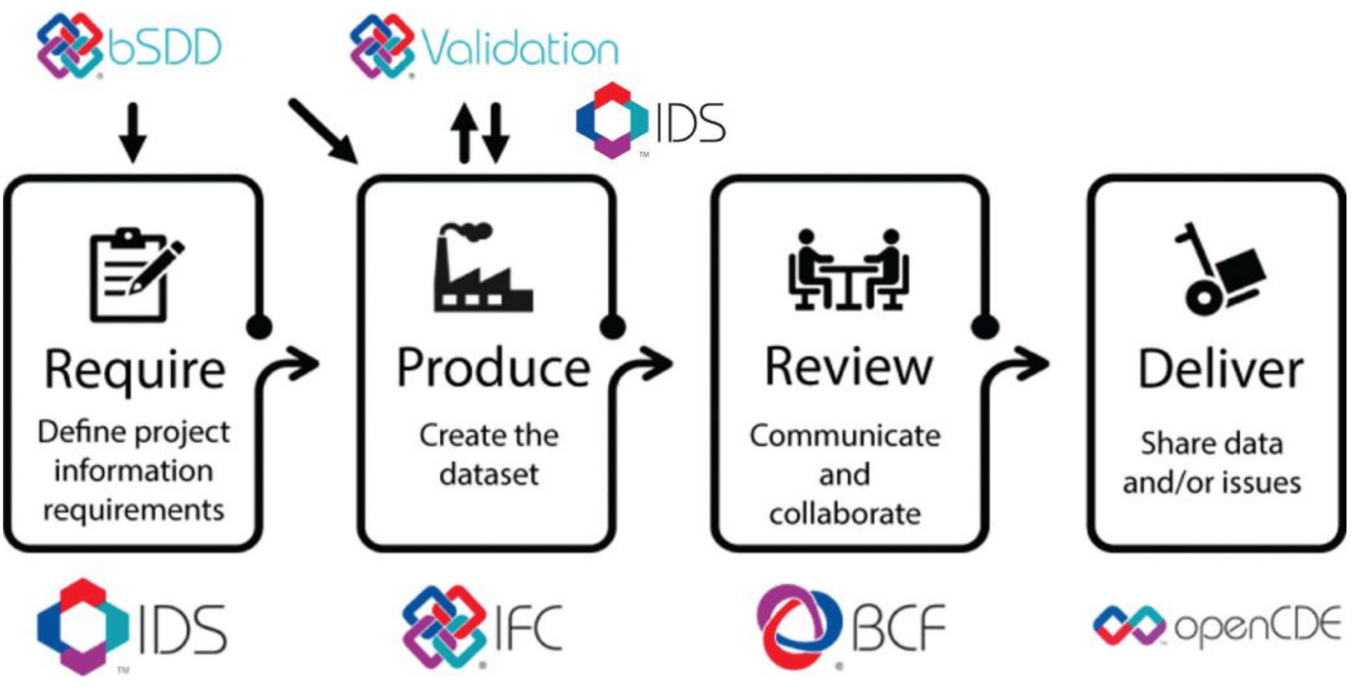
The openBIM Workflow

buildingSMART International develops and maintains open standards that support digital transformation across the built environment. To illustrate how these standards work together, buildingSMART has defined the openBIM workflow (**Fig. 1**).

The workflow begins with defining information requirements, followed by model creation, collaboration, validation, and delivery of information to downstream stakeholders. The buildingSMART Data Dictionary[7] enables standardized definitions of modeled entities to be customized and shared, whereas the Information Delivery Specification[8] defines content and properties required for BIM models. Models created using the IFC schema can be validated against these requirements to ensure completeness and consistency using the buildingSMART validation service.

The BIM Collaboration Format[9] supports model-based communication by capturing views, comments, and issue-tracking information without embedding markups directly in the model. Models and associated documents are delivered through a common data environment (CDE), and the openCDE standard enables information exchange across multiple CDE platforms using a nonproprietary protocol.

Figure 1. The openBIM workflow and associated buildingSmart International standards and services. Note: bSDD = buildingSMART Data Dictionary; BCF = BIM collaboration format; CDE = common data environment; IDS = information delivery specification; and IFC = Industry Foundation Classes. Figure: buildingSmart International.



Conclusion

BIM is often thought of as being only 3-D models because they are the most visible element of the process. However, models are only one component of a broader framework that supports information management across design, construction, operations, maintenance, and asset management. BIM supports users in reducing project risk, streamlining data collection and management, and bolstering informed decision-making throughout the life cycle of a transportation asset. To achieve these benefits, stakeholders must adopt new technologies and maintain a sustained commitment to open standards, defined processes, and organizational change.

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PROJECT

Texas Replaces Washed-Out Cow Creek Bridge in 29 Days

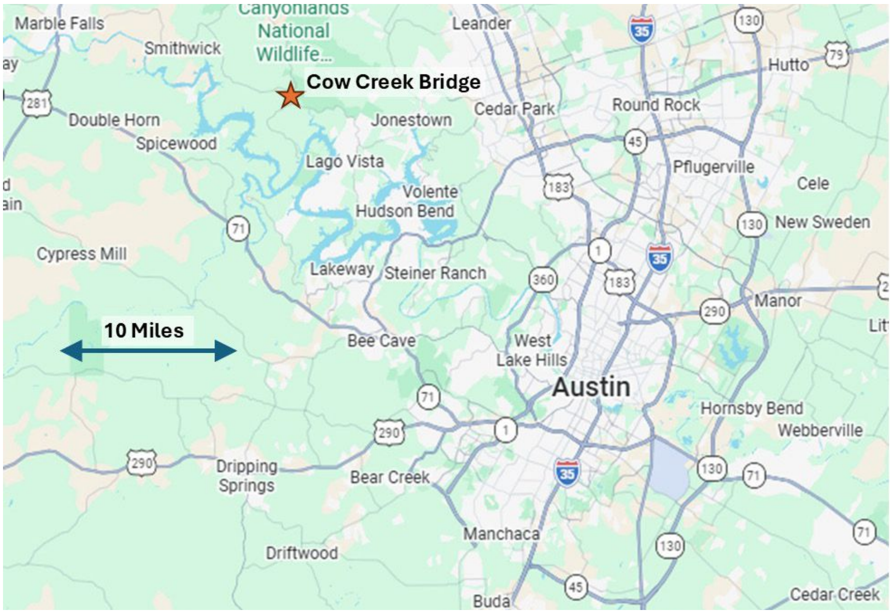
by Biniam Aregawi and Michael Hyzak, Texas Department of Transportation

On July 4, 2025, as the United States celebrated Independence Day, severe flooding swept through central Texas, causing widespread damage to infrastructure across the region. One of the hardest-hit areas was northwest Travis County, where the rapidly rising waters of Cow Creek posed an immediate and dangerous threat. In the early hours of July 5, the Ranch to Market Road (RM) 1431 bridge over Cow Creek—a 240-ft-long, six-span reinforced concrete girder bridge built in 1960—washed out under the force of the floodwaters. Despite being rated in satisfactory condition during its inspection in January 2025, the bridge was overwhelmed by the intensity of this extreme hydrologic event. Two people died in the collapse, and a vital link between the communities of Marble Falls and Lago Vista was severed, resulting in a 40-mile detour for the 3500 vehicles that used this route daily.

Historic Hydrologic Event

The Texas Hill Country is known as "flash flood alley" because it has one of the greatest risks for flash flooding in the United States. The region is prone to flash floods because of its steep terrain, shallow rocky soils, and unusual rainfall intensity. Bridge structures can become inundated during lower-probability events. Minor arterials and collectors such as RM 1431 typically are designed for a 4% annual exceedance probability or 25-year annual recurrence interval. Stormwater velocities can be up to 20 ft per second, and these stormwaters can carry substantial debris.

Cow Creek Bridge is 40 miles from Austin, Tex., and located on one of the few roads in the area, which had a 40-mile detour after the bridge washed out. Figure: Google maps.



In the July 2025 storm, approximately 17 in. of rainfall occurred during a 24-hour period and 8.6 in. of rain fell in a 2-hour period in the vicinity of Cow Creek, which amounted to an estimated 200- to 300-year event. During the preceding winter, ice storms combined with ongoing drought resulted in significant amounts of woody debris. The original 1959 design of Cow Creek Bridge was developed using the rational method to calculate the peak hydraulic flow for a drainage area of 33,000 acres. Manning's equation for hydraulic analysis resulted in a 25-year design event with a peak flow of 28,050 ft^3/sec and 1.75 ft of available freeboard. These design methods do not meet the current requirements in Texas Department of Transportation (TxDOT) and Federal Highway Administration guidance for hydrologic and hydraulic calculations. Investigators for the National Oceanic and Atmospheric

profile

COW CREEK BRIDGE / NORTHWEST TRAVIS COUNTY, TEXAS

BRIDGE DESIGN ENGINEER: Austin District Bridge and Hydraulics Section, Texas Department of Transportation

PRIME CONTRACTOR: Hunter Industries Ltd., San Marcos, Tex.

CONCRETE SUPPLIER: Texas Materials Ready Mix, Austin, Tex.

PRECASTER: Bexar Concrete Works Ltd., San Antonio, Tex.

OTHER MATERIAL SUPPLIERS: Reinforcing steel: CMC Steel, Seguin, Tex.; bearing pads: Bearing LLC, Tyler, Tex.

The original bridge was overtopped by Cow Creek, and segments were displaced some distance away. All Photos: Texas Department of Transportation.

Administration (NOAA) Physical Sciences Laboratory conducted a more modern and refined analysis, based on a 2025 Travis County study, using LIDAR for drainage area delineation, NOAA Atlas 14[1] precipitation depths, US Army Corps of Engineers Hydrologic Engineering Center Hydrologic Modeling System data for losses and routing, and Hydrologic Engineering Center River Analysis System data for hydraulics. The analysis provided a more accurate 25,000-acre area and determined that the same 25-year event generated a flow of nearly 37,700 ft^3/sec, resulting in calculated overtopping of the existing bridge.[2]

Rapid Response Plan

As soon as the floodwaters receded on July 6, engineers from the TxDOT Austin District conducted a field assessment. They confirmed the total failure of the structure and discovered entire bridge spans scattered up to 100 yards from their original location.

A maintenance contract was used to allow a contractor to quickly clear the debris and prepare the site for reconstruction. Closure of the road provided the contractor with unimpeded access without facing traffic challenges through the work zone.

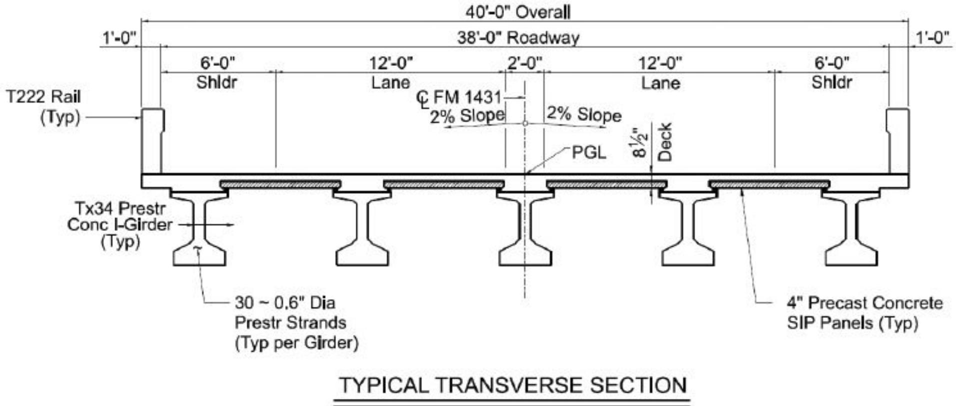
TxDOT administration set a goal of completing the reconstruction effort within an aggressive two-month time frame, as the community wanted to have the roadway open before the start of the new school year. The Austin District initiated design efforts on Monday, July 7, just two days after the bridge was washed away.

Daily TxDOT coordination meetings were held to guide the design process. The effort relied on an approach that is well established in Texas for emergency situations that need rapid implementation from event to finished product. Specifically, in such situations, TxDOT uses standardized designs that are common and familiar across the state, with readily available components from Texas fabricators and contractors. (For more information, see the State article on Texas in the Fall 2020 issue of *ASPIRE*®.) Through consistent collaboration and long workdays, the design team was able to deliver 90%-stage plans by Friday, July 11.

Design Improvements

Compared with the bridge that washed away, the new bridge includes several structural improvements to enhance resistance to extreme flood events. The diameters of the round columns were increased from the typical 36 in. to 49 in., and the cross sections of the rectangular bents were also increased in size. Whereas the original bridge had six span lengths of 40 ft, the new structure's three spans are 80 ft to improve the hydraulic flow. Enhanced concrete shear keys were incorporated at the substructure-to-superstructure interface at all bents to enhance lateral and overturning force resistance.

Geometrically, the existing roadway followed the topography of the hilly



Cross sections of new Cow Creek Bridge. The new bridge was widened a nominal amount. Figure: Texas Department of Transportation.

TEXAS DEPARTMENT OF TRANSPORTATION, OWNER

BRIDGE DESCRIPTION: 240-ft-long, 40-ft-wide precast, prestressed concrete bulb-tee girder (TX girder) bridge

STRUCTURAL COMPONENTS: Fifteen TX34 bulb-tee girders; approximately 6240 ft^2 of 4-in.-thick precast concrete deck panels with a 4½-in.-thick cast-in-place concrete deck; two cast-in-place abutments; two cast-in-place bent caps; six 48-in.-diameter cast-in-place columns; eight 36-in.-diameter drilled shafts; six 48-in.-diameter drilled shafts

BRIDGE CONSTRUCTION COST: $4,884,423 for total project; $2,358,575 for bridge items ($245.68/$ft^2$). (Note: Bridge items costs do not include incentive bonus.)

terrain with a sag curve at the bridge location. In addition, a local road intersects RM 1431 near the east end of the bridge. Ideally, the proposed design would improve the hydraulics, provide a 45-mph design speed, and accommodate the intersecting local road. The geometric design was a balancing act of minimizing the scope (and time) of the reconstruction while improving the roadway and the hydraulic and structural designs. The original vertical profile of a single sag curve was changed to a pair of sag curves with a constant tangent grade section over the length of the bridge. The result was a 35-mph design speed (matching the existing design speed) with a 4.5-ft raise in vertical alignment to provide measurable freeboard for a 10-year flood and no overtopping for a 25-year flood. A design waiver was required for maintaining design speed, vertical profile, and reduced shoulder width. These design decisions avoided right-of-way acquisition, major utility adjustment, a potential need for retaining walls, and excessive impact to the intersecting local road, whose profile grade was affected slightly.

Accelerated Construction

While Texas has a robust precast, prestressed concrete industry, fabrication of components still takes months. Through an established network, the TxDOT Austin District reached out to fabricators and neighboring TxDOT districts to identify available beams and precast concrete subdeck panels. Fortunately, the Cow Creek project could use Texas bulb-tee girders (TX34 girders) and precast concrete subdeck panels that had already been fabricated for a project in the adjacent Yoakum District. Integrating these elements into the design significantly accelerated the construction timeline.

Through established emergency maintenance contract methods, the Austin District's preliminary plans were immediately distributed to a few preselected contractors for their review and feedback. Before bidding occurred, TxDOT held a meeting with the prequalified contractors on July 12 at the Cow Creek site to ensure that they fully understood the project scope. Final signed and sealed design plans were released on July 14, bids were received on July 16, and the project was awarded the very next day.

Installation of the 79.5-ft-long precast, prestressed concrete TX34 bulb-tee girders is complete on two of the three spans. The creek typically only carries a small amount of water, but with high-intensity rainfall events, the hydraulics can change immensely.

The project time estimate was set at a total of 60 working days and 50 days for substantial completion, based on a 7-day workweek. However, the contract had an incentive/disincentive of $50,000 per day for early substantial completion of a maximum of 20 working days. The contract award was for approximately $4 million, but the contractor would receive a $1 million bonus if work were substantially completed in 30 working days or sooner.

The contractor mobilized quickly, and construction operations began on Monday, July 21, with two crews working 12-hour shifts during various critical periods throughout the project. Two drilling rigs were mobilized to install four 36-in.-diameter drilled shafts at each of the two abutments and three 48-in.-diameter drilled shafts at each of the two interior bents. Foundations were completed by July 26, and then matching cast-in-place round columns with rectangular bent caps and spill-through abutments were constructed. The 15 precast, prestressed concrete bulb-tee girders, with lengths just under 80 ft, were erected on August 5.

A key to the efficiency of the bridge construction was using concrete mixture proportions that provided higher earlier strengths for the cast-in-place concrete. For this project, the 28-day specified strengths were 3600 psi for the substructure and 4000 psi for the bridge deck; there were no concerns about creep and shrinkage with such a quick placement because the girders were from another project and had sufficiently aged. TxDOT specifications place restrictions on concrete strength for formwork removal and next-sequence construction, so accelerating strength gain is vital to rapid replacement with cast-in-place concrete elements. TxDOT allowed the use of preapproved straight cement mixture designs that achieved full design strength for drilled shafts, columns, bent caps, and bridge deck within 48 to 72 hours. Concrete durability for the bridge components was maintained by ensuring adherence to TxDOT's curing requirements for method and duration. During construction, a nearby ready-mix concrete plant was solely dedicated to provide concrete on demand for this project.

After the girders were erected, the next critical phase was the bridge deck. Crews installed and adjusted partial-depth precast, prestressed concrete panels and overhang brackets, tied the deck's steel reinforcement, and placed the expansion joint assemblies. In the early morning hours of August 12, the bridge deck was ready for concrete placement. The contractor used two pump trucks to facilitate a speedy placement of the bridge deck concrete. A few days later, crews installed the concrete bridge rails using welded-wire reinforcement and concrete slip-forming technique. Each of these steps marked a key milestone in the accelerated construction timeline.

Throughout the construction process, design and construction personnel were available to respond quickly to any field change requests, avoiding the typical delays of a traditional change-order process. A ribbon-cutting ceremony was

The cast-in-place concrete deck is placed at at night to accommodate the project schedule. However, it is common to cast concrete decks at night during the summer months in Texas.

held on August 19, 2025, just 45 days after the collapse and 29 days after start of construction.

Conclusion

The new Cow Creek Bridge is a precast, prestressed concrete bulb-tee girder structure designed to meet current safety standards and enhance hydraulic capacity. Elevated and widened, the bridge consists of three 80-ft spans with a contiguous deck unit of typical Texas link slabs, totaling 240 ft in length. It features an overall width of 40 ft, including a 38-ft-wide roadway that accommodates two 12-ft travel lanes, a 2-ft striped median, and 6-ft shoulders on both sides. This configuration improves traffic flow and safety, while the larger and more robust substructure elements increase resilience against future flood events. From initial damage to full restoration, the entire reconstruction process took only 45 days, which is likely a record for TxDOT.

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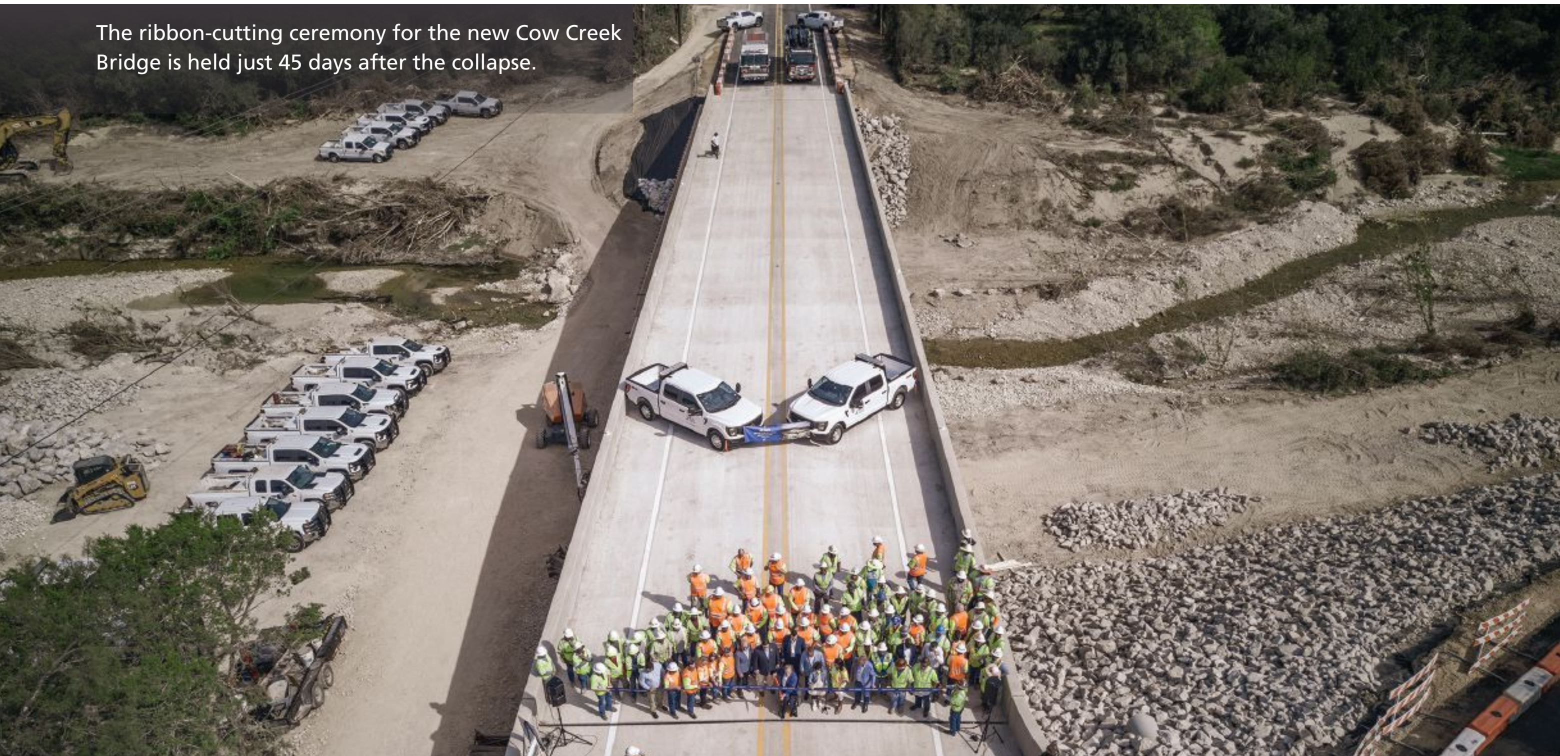
Biniam (Bin) K. Aregawi is Bridge and Hydraulics Section manager in the Austin District of the Texas Department of Transportation. Michael Hyzak is a bridge engineer with the Bridge Division of the Texas Department of Transportation in Austin.



The ribbon-cutting ceremony for the new Cow Creek Bridge is held just 45 days after the collapse.

Innovative Precast Concrete Trough Bridge for SMART's San Rafael Creek Crossing

by Brian Olp, STV

The Sonoma–Marin Area Rail Transit (SMART) system has been expanding rail service along the Highway 101 corridor since 2017, including a major design-build extension from San Rafael to Larkspur, Calif. The Larkspur terminal enables convenient connections to the Golden Gate Ferry and transit into San Francisco, making the final 2.1 miles of track a key link in the regional network.

Among the most technically challenging components of the extension was the San Rafael Creek crossing, which is located just south of the San Rafael Station and adjacent to Second Street. Tight geometric constraints, corrosive soils, a high-skew channel, and a narrow construction window demanded an unconventional approach. The SMART request for proposals originally envisioned a shallow concrete trough (U-shaped) bridge, a concept driven primarily by the need to avoid the use of steel in the corrosive tidal environment and to maintain a low profile between existing roadway elevations.

A traditional cast-in-place (CIP) concrete solution, however, was impractical. Environmental restrictions prohibited in-channel falsework, and the short summer construction window limited the feasibility of time-intensive CIP operations. Given these constraints, a precast concrete trough superstructure emerged as the most efficient and constructable solution. The owner's preference for ballasted track for ease of long-term maintenance was another reason to select an economical, shallow, and robust precast concrete section.

Design Development

Following contract award, the design-build team explored several precast concrete alternatives for the trough configuration. Early concepts used a traditional box girder with a ledge to support double-tee floor beams. As the design was refined, the design-build team transitioned to prestressed concrete rectangular voided-slab floor beams and eliminated the girder ledge, instead opting for a CIP longitudinal closure joint between the beams and edge girders.

The final design consisted of the following:

- Two precast, prestressed concrete voided-box edge girders
- Precast, prestressed concrete rectangular voided-slab floor beams
- A CIP longitudinal edge joint to tie floor beams and girders together
- Two levels of transverse post-tensioning (PT) to engage all elements into a unified trough
- Longitudinal PT in the floor beams to ensure shear continuity

This configuration created a fully integrated superstructure with no tension in the concrete permitted, in accordance with requirements of the American Railway Engineering and Maintenance-

Sonoma–Marin Area Rail Transit (SMART) system map. All Figures: STV.

profile

BRIDGE 16.86 OVER SAN RAFAEL CREEK / SAN RAFAEL, CALIFORNIA

BRIDGE DESIGN ENGINEER: STV, San Francisco, Calif.

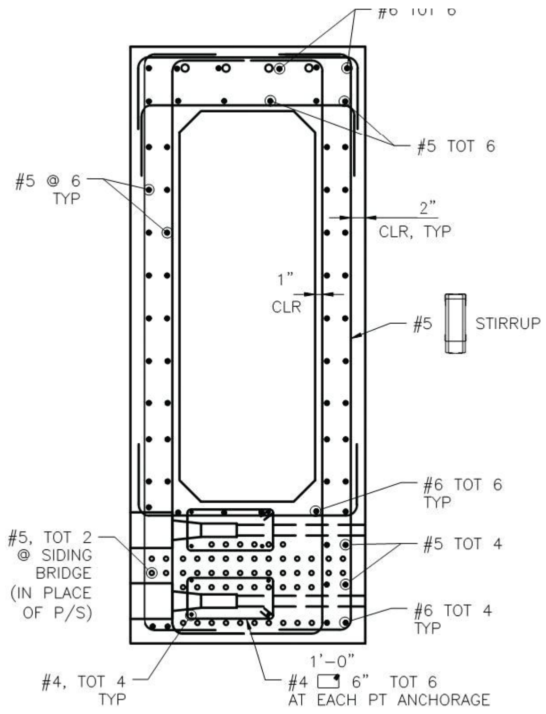
GEOTECHNICAL ENGINEER: HDR/WRECO, Roseville, Calif.

PRIME CONTRACTOR: Stacy Witbeck, Alameda, Calif.

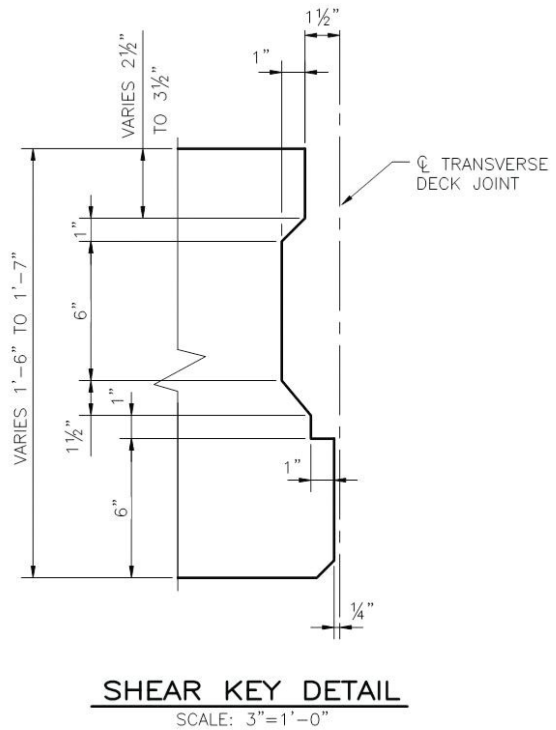
CONCRETE SUPPLIER: Shamrock Materials, San Rafael, Calif.

PRECASTERS: Kie-Con Inc., Antioch, Calif.—a PCI-certified producer, and Con-Fab California, Lathrop, Calif.—a PCI-certified producer

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



Cross section of a prestressed concrete edge girder showing reinforcement.



Floor beams have recessed shear keys along their longitudinal edges.

of-Way Association (AREMA). The design team used specialized structural design software to model staged construction, time-dependent effects, and temporary support conditions.

Geometry and Loads

The project design followed the *SMART Design Criteria Manual*[1] and *AREMA Manual for Railway Engineering*[2] for a 100-year design life. Loads included Cooper E50 maintenance trains and SMART's four-axle diesel multiple unit (DMU) with 46.7-kip axle loads. Owing to the significant skew of the channel, two separate, straight bridges were designed for the mainline and siding tracks. The bridge for the mainline tracks has a final span of 76 ft ½ in., while the final span of the siding-tracks bridge is 64 ft ¼ in. For each bridge, two edge girders and floor beams are used to create the trough shape. Clear spacing between edge girders is 18 ft 8 in. for the mainline tracks and 18 ft 0 in. for the siding tracks.

The bridge depth from the top of rail to soffit was held to 3 ft 6 in. despite the use of ballasted track. Compared with the elevation specified in the request for proposals, this depth provides a 6 in. raise in the soffit of the bridge, providing crucial vertical clearance over the waterway crossing.

Foundation and Superstructure Corrosion Protection

Site borings revealed highly corrosive soils, with chloride concentrations more than 10 times the limit set by the California Department of Transportation (Caltrans). To address this issue, the design team specified the following:

- Precast, prestressed concrete piles with a water–cementitious materials ratio of 0.37
- Calcium nitrate inhibitor at 3.5 gal/yd³ of concrete
- Epoxy-coated reinforcement in the abutment caps and all superstructure elements
- Increased concrete cover (3.5 in. for abutments; 2 in. for girders)

Precast concrete piles are approximately 35 ft long, with a specified distance to below-grade surface of 34 ft, to penetrate weak bay mud and reach competent-bearing strata. Sheet piles were used to slightly adjust abutment locations and minimize structure length.

Design Details

Each prestressed concrete edge girder is a 2-ft 9-in.-wide and 7-ft 0-in.-tall voided box designed to house two levels of transverse PT tendons, with

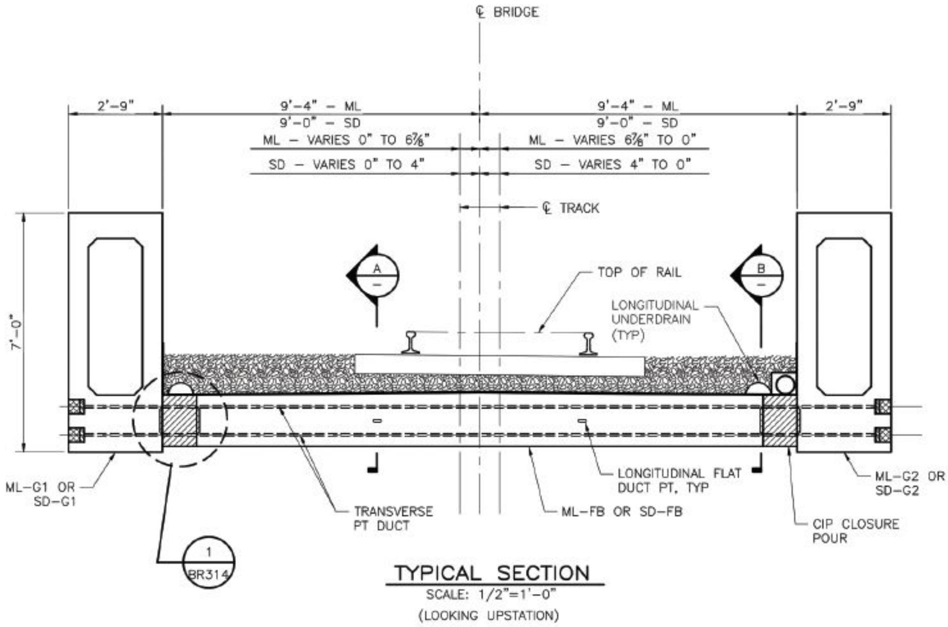
SONOMA–MARIN AREA RAIL TRANSIT (SMART), OWNER

OTHER MATERIAL SUPPLIERS: Sheet piles: National Pipe & Piling; abutment reinforcing bars: Harris Rebar, Livermore, Calif.; bearing pads: Scougal Rubber, McCarran, Nev.

BRIDGE DESCRIPTION: Two single-span, single-track precast, prestressed and post-tensioned concrete trough railroad bridges.

STRUCTURAL COMPONENTS: Four precast, prestressed concrete edge box girders; thirty-five precast, prestressed concrete floor beams; cast-in-place edge joint and abutments; thirty-eight 14-in.-square precast, prestressed concrete piles.

BRIDGE CONSTRUCTION COST: $1.5 million



Typical cross section of the trough (U-shaped) bridges.

View from beneath an edge girder. No falsework was permitted in the creek, so the team developed a temporary support system to support the floor beams during construction. The supports were later repurposed to facilitate installation of the bridge's transverse post-tensioning. All Photos: Stacy Witbeck/Herzog.

a solid 3 ft 10 in. length at each end. The concrete compressive strength at transfer was 5 ksi, with a final design compressive strength of 8 ksi. Longitudinal reinforcement in the edge girders consists of straight longitudinal prestressing strands, a portion of which was debonded at the girder ends, and transverse PT to connect the girders with the floor beams. Mainline girders and siding girders used 58 strands and 46 strands, respectively, of 0.6-in.-diameter low-relaxation prestressing steel that was initially tensioned to $0.75f_{pu}$.

Recessed shear keys and overlapping hooked dowels connect each girder to the floor beams within the CIP longitudinal joint. The surface of the edge girders at the joint was form finished, not intentionally roughened. A small-aggregate, 5-ksi concrete was used to ensure flowability; the concrete also includes a corrosion-resistance admixture. The girders are supported at the abutments by 3-in.-thick, steel reinforced elastomeric bearing pads. Dowels were connected by a form-saver cast into the girder, with a U-bar cast into the floor-beam ends that extend into the CIP concrete joint.

The 18-in.-deep, approximately 4-ft-wide precast concrete floor beams were prestressed with eight 0.6-in.-diameter strands each. These beams vary in thickness from 19 in. at the center to 18 in. at the end, and they are topped with a slight cross slope for drainage. Concrete for the floor beams was designed for an initial compressive strength of 4 ksi, and a design compressive strength of 5 ksi. Each beam has recessed shear keys along their longitudinal edges that were grouted with 5-ksi non-shrink grout after placement, and ducts for longitudinal PT connecting the entire floor system. The floor beams were connected by PT before the longitudinal closure joint was placed.

The CIP closure joint between the floor beams and edge girders required dense reinforcement and staggered mild-steel reinforcement couplers to maintain constructability. Detailed analysis ensured that even in a hypothetical tendon-loss scenario, which was used to check for redundancy in case any one strand was compromised, shear and flexural capacity would remain adequate. Transverse PT

AESTHETICS **COMMENTARY**

by Frederick Gottemoeller

Not every bridge is a landmark bridge like the Corpus Christi Harbor Channel Crossing featured in the Winter 2026 issue of *ASPIRE*®. Many bridges, particularly those used for rail or transit like the Sonoma–Marin Area Rail Transit's (SMART's) San Rafael Creek crossing, are background bridges. They do not have a prominent scenic location; they are not meant to attract attention. However, that is not an excuse for designers to be careless about the appearance of such structures. These background bridges can still be part of important urban or rural scenes. They are aesthetic elements that can affect the overall visual quality, enjoyment, or even economic success of their surrounding communities. The SMART bridge located close to the San Rafael Station and Second Street performs that role in San Rafael.

With background bridges, visual simplicity is especially important. The SMART bridge's rectangular box girders are as simple as it gets. No shadow lines, texture, or splashes of color have been added. None are needed. The soffit (which would be important if the bridge were crossing a street) displays only the lines dividing the precast concrete slabs, and a relatively light, reflective color is used to brighten the space below the bridge. Again, this is as simple as it gets.

Although the designers selected the bridge components for structural and constructability reasons, not for visual quality, the San Rafael Creek crossing is yet another example of a "twofer," when a decision made for reasons of efficiency and economy also improves appearance. Neither a park nor a commercial establishment nearby would have any reason to complain about the detrimental effect of the appearance of the SMART bridge.

Crew members use a single, 400-ton crane to install one of the prestressed concrete edge girders for the bridge of the siding track on the south side of the creek.

A tandem crane pick is used to place a 154-kip mainline track edge girder on the abutments.

Timber struts provide bracing to mitigate the inward rotation of the edge girders during installation of the floor beams. After each floor beam is set, the shims at the strut ends are adjusted.

with four 0.6-in.-diameter strands per duct was the final operation, fully engaging the girders and floor beams into a rigid monolithic trough.

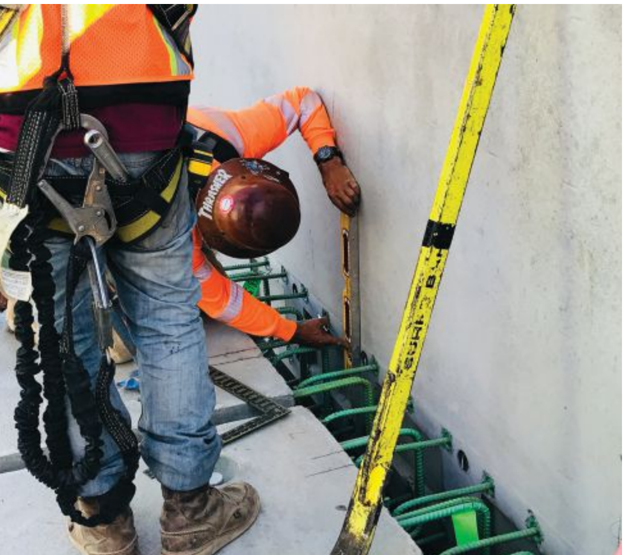
Two local precasters fabricated the precast concrete; one supplied the piles and the other provided the edge girders and floor beams. Because no falsework was permitted in the creek, the team developed a specialized temporary support system in which steel, hollow-structural-section (HSS) brackets embedded into the underside of the precast concrete box girders supported the floor beams during placement. This system allowed crews to set all the beams from above without any temporary supports in the channel.

Construction Sequence and Challenges

Construction required careful staging to accommodate the brief environmental window (June 15–October 15), roadway constraints at Francisco Boulevard West, and overhead utilities that had to be moved underground.

Pile and Abutment Work

Sheet piles were first installed using a vibratory hammer. High tides overtopped the sheets on the south side, requiring frequent dewatering—an issue later mitigated by driving taller steel sheet piles with sealed interlocks on the north side. Precast concrete piles were driven with a diesel hammer and cut to elevation before CIP abutment caps were placed. 35-ft-long precast concrete piles were selected for the highly corrosive site. Pile caps are 5 ft wide, 4 ft 6 in. thick, and 26 ft 8 in. long, with a 1-ft



A worker verifies the post-tensioning duct alignment between floor beams and edge girder.

As the precast concrete floor beam is placed, the shear key and blockouts for the bridge's longitudinal post-tensioning are visible.

Epoxy-coated reinforcement (green) and post-tensioning ducts (red) are visible in the floor beam–to–girder joint before placement of the closure pour.

backwall on top of each cap. Shear keys at the end of the pile caps were designed to remain elastic under AREMA level 1 loading but fail under higher loading levels to protect the foundations.

Installation

The 154-kip, 76-ft-long edge girders required a tandem pick using two cranes with lifting capacities of 400 and 350 tons positioned on opposite sides of the creek. The girders were initially set on dunnage to allow the handoff between the cranes, and then set on bearings.

As previously noted, the team developed a specialized, temporary support system using HSS brackets embedded in precast concrete box girders. This allowed the floor beams to set without temporary supports in the creek.

Because the floor beams initially imposed eccentric loading, each edge girder was anchored to the abutment with pipe bracing to maintain plumbness during erection.

Alignment of PT ducts between each floor beam and the girders required precise layout. To mitigate the inward girder rotation under the asymmetric load, the team installed full-length timber struts across the top of the girders before placing any beams. After each beam was set, crews adjusted shims at the struts to keep the girders plumb.

Grouting and Post-Tensioning

Blockouts in the floor beams allowed the bridge's longitudinal PT ducts to be connected before grouting. After the longitudinal PT was tensioned, the CIP edge joint was placed. This step was particularly demanding because of the tight reinforcing bar spacing and coupler congestion.

Because the pockets for the transverse PT were located above the creek, using traditional lifts for the jacking operation was not feasible. The team ingeniously repurposed the HSS temporary brackets, installing lumber and planking to create a suspended walkway for PT operations that was compliant with Occupational Safety and Health Administration requirements.

Conclusion

The San Rafael Creek Bridge demonstrates how innovative precast, prestressed concrete solutions can overcome severe site constraints and environmental limitations. By integrating precast concrete edge girders, prestressed voided-slab floor beams, and a fully PT trough structure, the team delivered a highly durable, corrosion-resistant structure with accelerated bridge construction methods and minimal impact to the sensitive channel.

This project showcases the adaptability of precast, prestressed concrete to complex railroad applications and highlights the value of collaborative design-build delivery in achieving both performance and constructability goals.

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Brian Olp is principal and senior project engineer for STV in Denver, Colo.

Crew members tension the transverse post-tensioning tendons from a temporary platform supported by the repurposed support system used during the installation of the floor beams.

View of the two completed trough bridges. The mainline track is on the right, and the siding track is on the left.

Innovative Design for Colorado's West Vail Pass Bridges

by Chad Hammond, RS&H

The Interstate 70 (I-70) West Vail Pass auxiliary lanes project included updating the roadway curvature to current standards, widening the lanes and shoulders, and replacing the eastbound and westbound bridges over Polk Creek. See the Project article on page 20 of the Winter 2026 issue of *ASPIRE*®. for general information about the bridges and construction. This article covers some of the unique design aspects of the project in detail.

Structure Type Study

The design of the twin bridges required innovative ideas and collaborative solutions from the design team with input from the construction manager/general contractor, the Colorado Department of Transportation (CDOT) as the owner, and other stakeholders. The bridges are approximately 575 ft long with an out-to-out width of 55 ft and a constant 1680-ft radius horizontal curve along their entire lengths. Based on the aesthetic criteria for the project, the bridge girders had to be curved and girder shapes were limited to tub or box girders. The Structure Type Study evaluation determined that a simple-made-continuous, precast concrete tub girder option offered cost and schedule advantages. Therefore, this option was carried through to final design.

Final Design

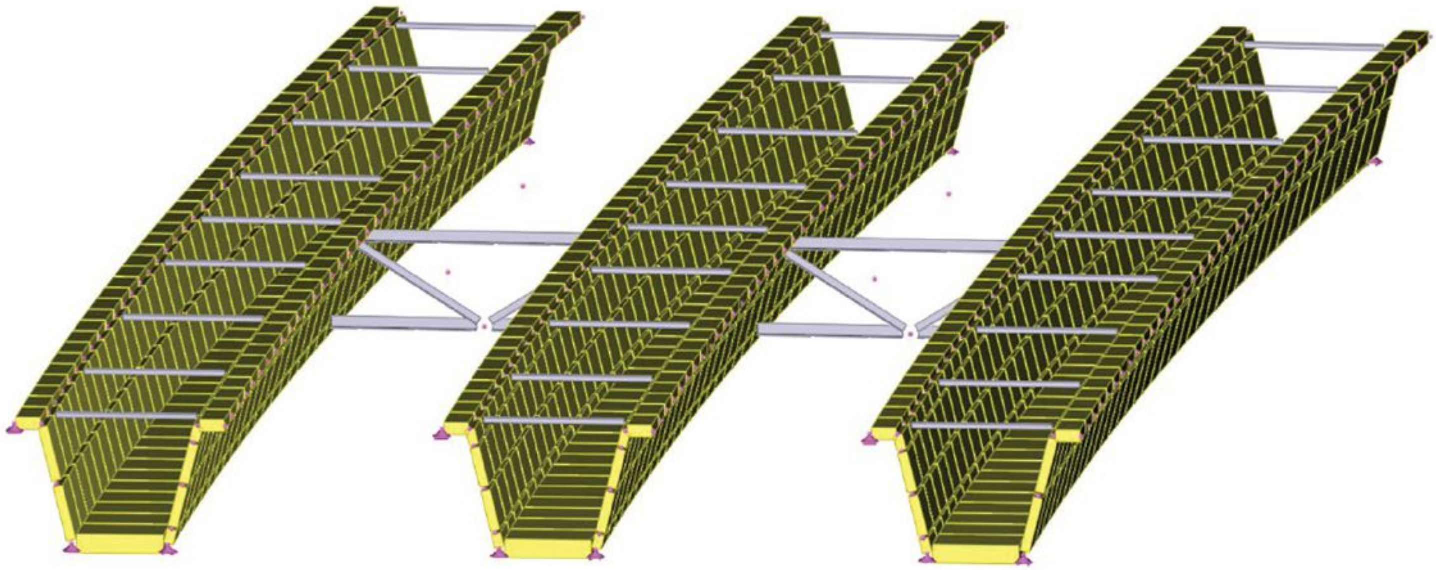
For the simple-made-continuous design, the precast concrete tub girders are assumed to behave as simple spans for noncomposite loads applied before the deck cures, but continuous for loads applied to the composite section (that is, superimposed dead and live loads).

The final design of the precast concrete tub girder alternative used the typical references for CDOT bridge designs and a few specialty resources:

- The American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*, 9th edition[1]
- The *CDOT Bridge Design Manual*[2]
- PCI's *Guide Document for the Design of Curved, Spliced Precast Concrete U-beam Bridges*[3]
- The California Department of Transportation's *Bridge Design Memo 5.26: Anchorage Zone for Cast-in-Place Prestressed Box Girder Bridges*[4]
- National Cooperative Highway Research Program Report 356, *Anchorage Zone Reinforcement for Post-Tensioned Concrete Girders*[5]

Three finite element models were used in the analysis of the bridge. The first model was a time-dependent line-girder model used to track stresses and forces in the superstructure at various stages of construction. Results from this

A three-dimensional finite-element plate model of the longest span was used to evaluate torsion and distortion in the girder before and during deck placement. Temporary and permanent bracing was designed based on the results of these analyses. All Figures: RS&H.



The difference in width and curvature between the original Interstate 70 (I-70) over Polk Creek bridges (left) and the nearly completed westbound bridge (right) is clear in this photo. By widening traffic lanes and shoulders and modifying existing curves to meet current federal design standards, the project improves safety and operations on I-70 for the traveling public. Photo: Dennis Humphrey, Cirque Civil.

model were used for the superstructure longitudinal design, prestressed concrete stress checks throughout construction, shear and moment design of the composite girder, camber and dead-load deflections, time-dependent effects, creep, shrinkage, prestress losses, and restraining moments due to continuity.

The second model was a three-dimensional plate and member model of the full bridge—superstructure and substructure—which was used to look at girder torsion for the closed section after deck placement and for substructure and bearing reactions. The pier diaphragms are detailed to perform as pinned connections. The girders sit on leveling pads at the piers and vertical reinforcing bar dowels run along the centerline of the piers to create a pinned connection.

At the abutments, the girders are supported by CDOT Type III bearings, which are a disc-style bearing that allows significant movement. The lower sole plate of the bearing assembly slips over a polytetrafluoroethylene surface on the upper bearing plate. The upper bearing plate sits on a urethane disc and is restrained by a steel pin. The center girder uses a guided bearing to prevent transverse movement of the superstructure at the abutments, and all movements were assumed to act along the curve due to the large radius. The boundary conditions for piers and abutments were used in this finite-element model, which allowed the design team to obtain accurate substructure reactions from this model.

The third model was a three-dimensional plate model of the longest span looking at torsion and distortion in the girder before and during deck placement. This model was used to develop the temporary diaphragm design, which was used to limit girder distortion during deck placement. The reactions derived from this model were provided to the contractor for the temporary support of the girder ends.

The project team used guidance from the *CDOT Bridge Design Manual* for the continuity diaphragm design. The manual indicates that when girders are aged at least 90 days, continuity diaphragms should be designed for a positive moment of 1.2 times the girder cracking moment and the negative moment calculated for the specific span configuration. In this case, the negative moments were pulled from the finite-element model and positive restraining moments were compared to the cracking moment to verify that the cracking moment controlled positive moment design at the diaphragms.

The final design uses 6-ft-deep precast concrete tub girders that are 4.5 ft wide at the bottom flange and 9.5 ft wide at the top. The design concrete compressive strengths for the precast concrete girders were 6.5 ksi at transfer and 8.5 ksi at 28 days. All girders have four post-tensioning tendons in the bottom flange. However, there was only room to anchor two tendons at any given location, so one anchorage was placed at the ends of the girders and a second anchorage was placed 12 ft from the ends. The number of 0.6-in.-diameter strands in each tendon was either 12 or 19, depending on the span length. Tendons were grouted after tensioning.

Because the girders were designed as simple spans made continuous, an interesting and unique part of the staging took place at the precast concrete producer's facility. Girders were removed from the casting bed and placed on temporary supports 30 ft from the girder ends—for all the girder lengths—until they were post-tensioned. These support locations were intended to limit the positive moment at midspan before

One of the precast concrete tub girders for span 3 of the westbound structure—with a girder weight of approximately 220,800 lb—is lifted to its final position on the piers. The simple-made-continuous for live-load concept for the Interstate 70 over Polk Creek bridges saved considerable time by eliminating the need for falsework towers. Photo: Kevin Petrillo, RockSol.

Temporary diaphragms were installed between girders to prevent girder distortion during deck placement and ensure stability. The bridge engineer designed temporary diaphragms at midspan (steel K-frames in this photo). The contractor designed temporary diaphragms at girder ends (wood timber and steel rods in this photo). Photo: Kevin Petrillo, RockSol.

post-tensioning and induce upward camber growth. The supports were shifted to the girder ends immediately after post-tensioning.

Design Challenges

One of the design hurdles for the simple-made-continuous option was girder weight. The *CDOT Bridge Design Manual* limits girder weight to 240 kip to ensure that girders can be shipped. The design team made several adjustments to the girders to reduce their weights. The largest girder in the final design was on the eastbound bridge (span 2) and weighed 249,300 lb. To reduce weight, the girder web thickness was minimized and set at 5 in. This revision was possible because the girders by themselves are not continuous over the piers, so post-tensioning is always in the bottom flange for the full length of the girder. Continuity over the piers is established using mild reinforcement cast into the deck along with the cast-in-place continuity diaphragms at each of the piers. Another weight-saving measure was the elimination of the concrete lid slab typically placed on top of curved concrete tub girders after fabrication. To replace the bracing typically provided by the lid slab, steel interior diaphragms were added to support the top flanges and webs for handling and shipping of the girders. A temporary steel diaphragm was installed at midspan between girders to prevent girder distortion during deck placement.

Tendon anchorage for the precast concrete tub girders. There was only room to anchor two tendons at any given location, so one anchorage was placed at the ends of the girders and a second anchorage was placed 12 ft from the ends. Photo: RS&H.

A major consideration during design was girder deflection and distortion due to the curvature in the girders. Addressing this issue was challenging because the girders were not closed sections with a lid slab. As previously discussed, deflections were analyzed using a finite element model of the girders with plate elements. The girder ends were assumed to be fixed. Deflection and twist were checked for two cases—girder self-weight and the girder supporting the wet deck concrete. Temporary diaphragms were added to the model to ensure that the girders would be sufficiently restrained during placement of the deck and to prevent racking of girders that could lead to excessive deck thickness. The plate model used to check deflections also provided end reactions for the girders. These reactions were included in the plans to aid the contractor in the design of girder end bracing.

Construction of the westbound I-70 bridge over Polk Creek was completed in 2023, and feedback about that bridge was incorporated into the final design for the eastbound bridge, which was completed in 2025. The innovative application of simple-made-continuous precast concrete girder concept proved to be cost-effective and saved time in construction. The successful design and construction of this project required innovative solutions and collaboration among the team members. The bridge replacements are an important milestone in the West Vail Pass auxiliary lane project, which is improving safety and operational capacity in the region.

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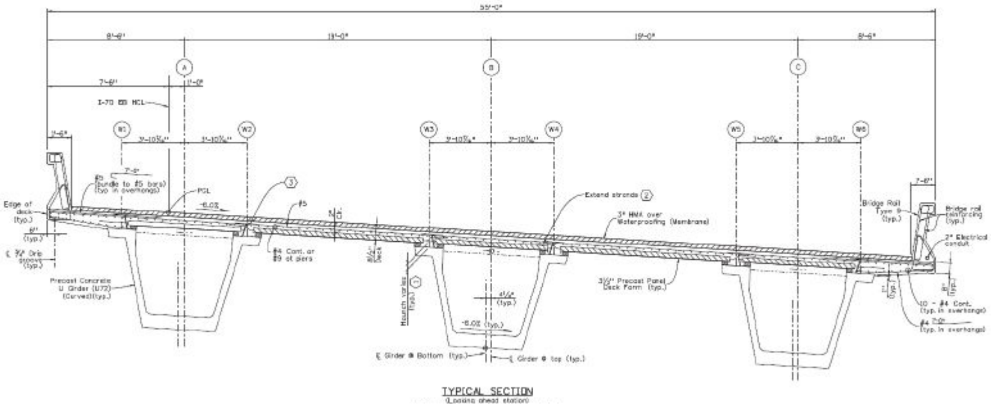
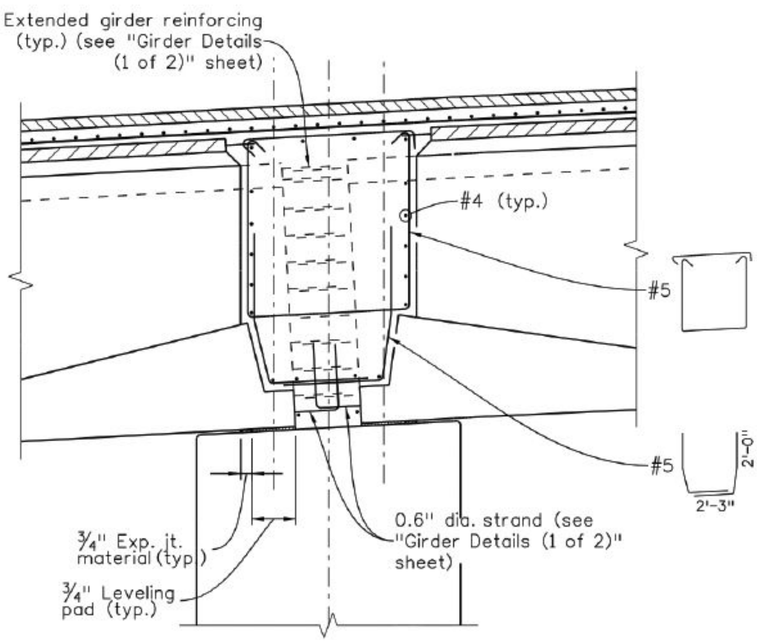


Bridge typical section showing the precast concrete tub girders with partial-depth precast concrete deck panels, cast-in-place concrete topping, and asphalt wearing surface.



Section showing the continuity diaphragms at the piers. The hooks extending from the girder bottom flanges provide a positive moment connection. The reinforcement in the cast-in-place deck provides the negative moment continuity reinforcement.

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EDITOR'S NOTE

A supplement to the Summer 2015 issue of ASPIRE® tracks the development of curved, precast, post-tensioned concrete U-girders. It can be found on the ASPIRE website under the Resources tab at www.aspirebridge.com/magazine/2015Summer/ASPIRESupplementSummer2015.pdf.

Drawing files related to the U-girder can also be found under the Resources tab of the ASPIRE website at www.aspirebridge.com/additionalresources/index.shtml

EDITOR'S NOTE

The West Vail Pass Auxiliary Lanes project has a number of interesting characteristics that make it worth another article in ASPIRE (see the Project article on page 20 of the Winter 2026 issue).

It is said that necessity is the mother of invention, but it is also the mother of innovative engineering. Engineers may not always consider what happens to precast concrete girders while they are stored at the precaster's facility, but that was an important consideration for this project. This project presents some innovative solutions tailored to the challenging site conditions.

The girders for the Polk Creek bridges did not have initial prestressing, so there was no initial camber. Therefore, there was a possibility these girders could develop sag while sitting in the yard before the post-tensioning was applied. To control stress, deflection, and ultimately the camber after post-tensioning, the girder support points (bunking) were moved in 30 ft from the girder ends until the post-tensioning was applied. The supports were then moved to the end of girder.

To accommodate limitations on the girder shipping weight, engineers employed several inventive practices. They chose to use a continuous–for–live load structure for reasons stated in the Project article. That choice in turn allowed for thinner webs, which reduced girder weight.

Engineers also elected to ship and construct without the concrete lid slab attached, making the girders open sections, rather than closed. Curved girders want to twist; the difference in torsional rigidity between an open section and a closed one is usually orders of magnitude. The solution started with analysis of the open one to understand the stresses and distortions that would occur when the deck was placed; those stresses and distortions were controlled by the innovative use of temporary steel frames.

From Concept to Reality: Idaho Champions a Seismic-Resilient Precast Concrete Bridge Pier System

by Dr. Mustafa Mashal and Jared Cantrell, Idaho State University, and Zak Johnson, Idaho Transportation Department

Precast concrete offers many advantages over cast-in-place (CIP) concrete construction. It plays an important role in accelerated bridge construction, which aims to reduce traffic disruptions, improve quality, enhance on-site safety, and limit environmental impacts, among other benefits. Despite these advantages, the use of precast concrete in bridge substructures (such as piers) has been largely limited to non-seismic regions due to concerns about the strength and ductility of connections during earthquakes.

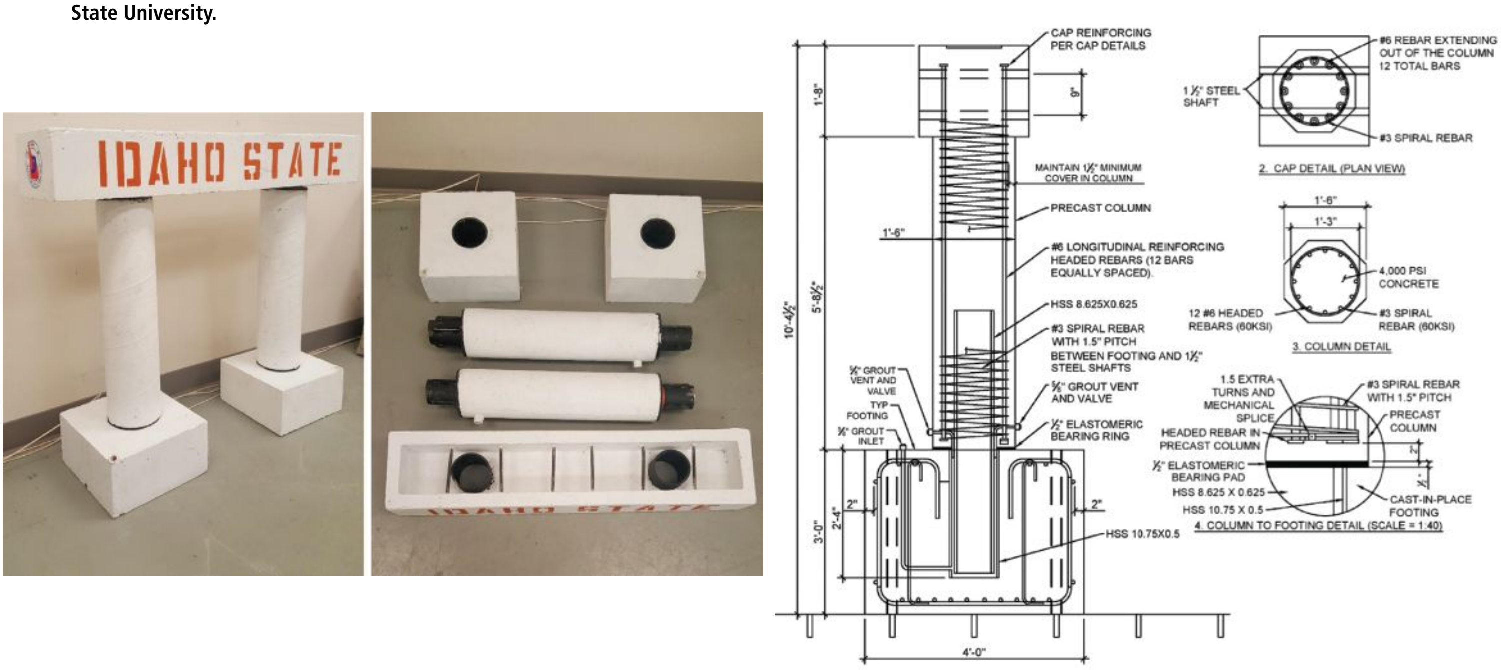
Recently, the Idaho Transportation Department (ITD) partnered with Idaho State University (ISU) to develop and validate the seismic performance of an innovative connection for precast concrete piers through large-scale experimental testing in ISU's structural laboratory. The project was conducted in two phases. The first focused on concept validation through large-scale testing of cantilever and bent specimens under simulated cyclic lateral loads and displacements, with performance compared against benchmark CIP samples.[1,2] The second phase focused on post-earthquake retrofitting strategies for the proposed precast concrete pier using ultra-high-performance concrete (UHPC).[3]

Concept

The precast concrete pier system uses precast concrete columns with concrete-filled steel (round, 8.625-in.-diameter HSS) tubes (CFSTs) protruding from each end (**Fig. 1**). The specified compressive strength for all test-specimen concrete was 4000 psi at 28 days. Each CFST fits inside a larger receiving HSS tube, with one receiving tube located in the precast concrete cap beam and another in the footing for the column-to-cap and column-to-footing connections, respectively. The gap between the CFST and the receiving tube (typically 0.5 to 1.0 in.) is grouted on site after assembly. The precast concrete cap may be solid or partially hollow, depending on size, weight, and transportation constraints.

Figure 1 shows the shop drawings for the proposed connection in a cantilevered

Figure 1. Components of the precast concrete pier system and shop drawings for the cantilever precast concrete pier. All Figures and Photos: Idaho State University.



precast concrete pier used for laboratory testing. An elastomeric pad is placed at the column-to-footing interface to help mitigate column cracking during smaller earthquakes by allowing limited rocking. For moderate to large earthquakes, the CFST located in the plastic-hinge zone provides shear strength, flexural resistance, and confinement. The proposed connection offers advantages, such as simplified construction and generous installation tolerances, which makes it a competitive alternative to other precast concrete connection systems such as grouted dowel connections.

Experimental Validation

Experimental validation of the ducts performed in ISU's structural laboratory involved four large-scale specimens: a CIP cantilevered column, a precast concrete cantilevered column, a two-pier CIP bent, and a two-pier precast concrete bent. The precast concrete connection concept for column-to-footing and column-to-cap are essentially the same (that is, CFSTs with HSS receiving tubes). For the cantilevered specimens, the focus was on the column-to-footing connection where the plastic hinge will be located. This plastic hinge location is applicable to both the CIP and precast concrete specimens. For the bent connection, both the column-to-footing and column-to-cap connections where plastic hinges will be formed were evaluated. **Figure 2** shows the experimental setup. Each specimen behaved as expected under quasi-cyclic lateral loading adopted from the loading protocol in the American Concrete Institute's *Guide for Testing Reinforced Concrete Structural Elements Under Slowly Applied Simulated Seismic Loads* (ACI 374).[4] The precast concrete specimens achieved greater energy dissipation and extended drift ratios for both the cantilevered (Fig. 2) and bent specimens, exhibiting better performance when subjected to seismic loading. Each test included a simulated vertical axial load throughout the experimental load protocol.

The CIP bent developed extensive cracking early in the test and ultimately failed after 15 cycles, following the fracture of the longitudinal reinforcement in the plastic hinges. In contrast, the precast concrete bent incorporating the HSS tubes in the plastic hinges showed noticeably less cracking but more pronounced spalling. Its failure occurred at 24 cycles, governed by deformation and eventual tearing of the HSS tube. Initial stiffness differed between the two specimens. The precast concrete bent reached 38.7 kip/in., compared with 56.7 kip/in. for the CIP bent. The difference was attributed to the flexibility in the precast concrete bent introduced by the elastomeric bearing. Although its stiffness was lower, the precast concrete bent achieved a higher ultimate lateral capacity of 71.4 kip, compared with 66.0 kip for the CIP bent.

The yielding performance of the specimens also varied. The CIP bent exhibited an initial drift of 0.5%, whereas the precast concrete bent yielded at 1.13%, which again reflects the added flexibility of the bearing pad. Global yield measurements showed the CIP bent reaching yield at 0.596 in. and 392 kip-ft, while the precast concrete bent yielded at 1.246 in. and 433 kip-ft. Energy dissipation further distinguished the precast concrete specimens as the higher-performing system. The lower stiffness of the precast concrete bent resulted in reduced early-cycle energy dissipation. However, as cycling progressed, the stabilizing effect of the HSS tube allowed the precast concrete system to develop a more consistent

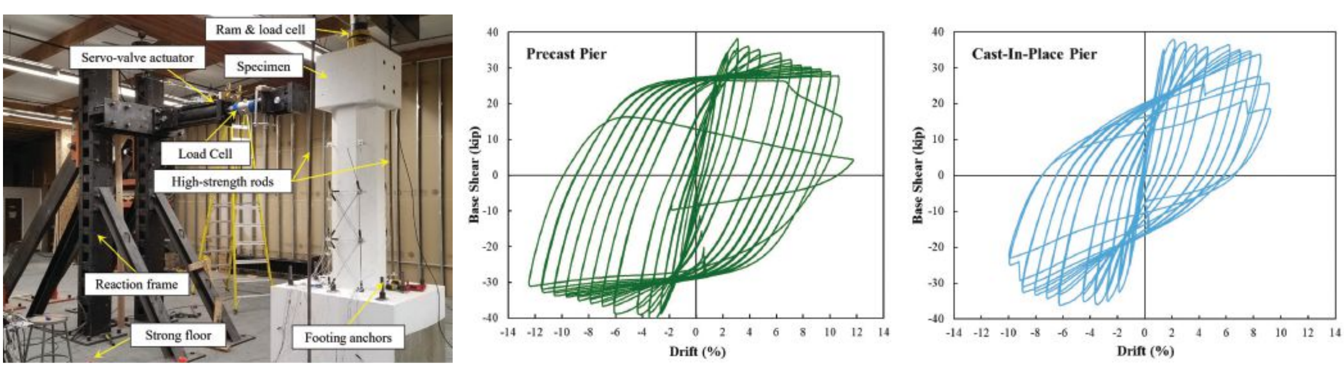


Figure 2. Experimental test setup and lateral force–drift hysteresis response for cantilevered piers.

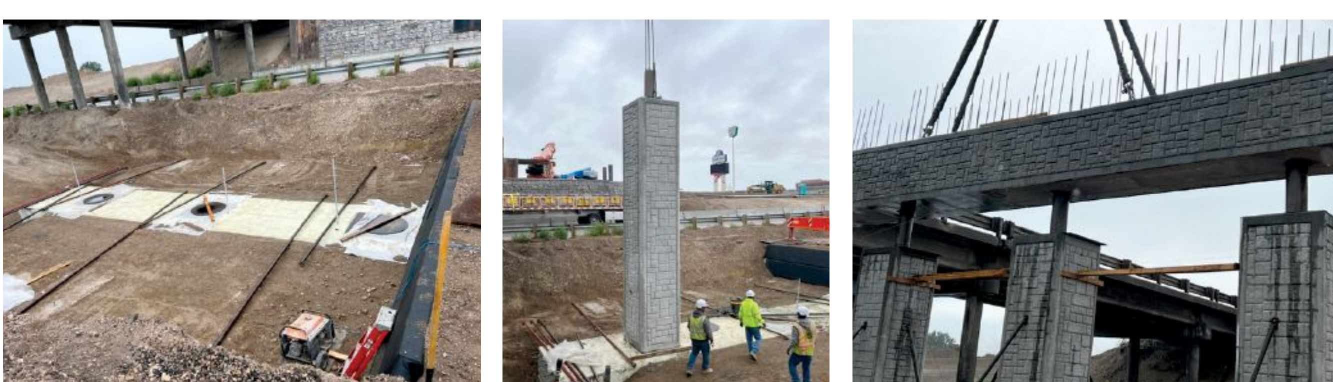


Figure 3. Construction of the Fort Hall Interchange Bridge over Interstate 15 in southeast Idaho. This project introduced seismically resilient precast concrete pier-element connections developed by Idaho State University and the Idaho Transportation Department.

Figure 4. The completed Fort Hall Interchange project features a 222-ft-long, 88-ft-wide bridge spanning Interstate 15 in Fort Hall, Idaho, with newly developed elements crucial for its location in a high-seismic region.

hysteretic response. At 15 cycles (the CIP bent failure point), the precast concrete bent had dissipated 466 kJ compared with 342 kJ for the CIP bent. The higher deformation capacity of the precast concrete bent enabled that specimen to complete 24 cycles, reaching a total cumulative energy dissipation of 2125 kJ.

Overall, the precast concrete system matched or exceeded the CIP performance, validating the proposed connection as an effective emulation of CIP behavior under seismic loading.

Implementation

The precast concrete pier connection was implemented in the design and construction of the Fort Hall Interchange over Interstate 15 in southeast Idaho, which is the most seismically active region in the state. The bridge is 222 ft long and consists of two spans. The innovative connection was used at the middle pier for the column-to-footing interface, while the column-to-cap connection was designed as "pipe pins."

The task of designing these innovative elements presented an engaging challenge for the bridge designers at ITD. The department consistently seeks methodologies to streamline and expedite construction processes, and the development of these CFST connections in collaboration with ISU constituted a significant achievement. Although there was a learning curve associated with integrating the new connection into ITD's standard design practices, the assistance provided by ISU's team facilitated smooth implementation without any complications.

The use of the CFST connection demonstrated a significant improvement over the conventional precast concrete connections. Typically, precast concrete elements are assembled by inserting multiple reinforcing bars (dowels) protruding from one element into corresponding ducts of the other element, a process that is time intensive and requires a high degree of precision during both casting and erection phases. In contrast, CFST construction simplified the assembly process from the precaster's yard to the jobsite, as it involved only a single component that needed to be placed and fitted (**Fig. 3**). The CFST system allows the contractor to erect the elements more efficiently and accelerates the overall timeline of the bridge. ITD received positive feedback about the connections from the contractor, who appreciated the ease of assembly. Furthermore, the contractor conveyed their desire for ITD to use CFSTs in future projects involving accelerated bridge construction.

Following the successful implementation at the Fort Hall Interchange (**Fig. 4**) and the favorable response from the contractor on that project, ITD is planning to employ CFST construction on upcoming bridges along the Interstate 15 corridor from Pocatello to Idaho Falls, Idaho. This corridor experiences high traffic volumes and provides access to several national and state parks. Therefore, any construction activities on this corridor must be expedited to minimize the disruption to the traveling public. The Fort Hall Interchange project demonstrated that CFST elements can effectively reduce the impact of construction for all parties involved.

Summary

The proposed precast concrete pier system offers advantages in seismic regions because of its simplicity and ease of installation. The precast concrete system uses HSS tubes filled with concrete to dissipate seismic energy, enhancing its performance and accelerating construction schedules.

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Dr. Mustafa Mashal is the associate vice president for Research Innovation Partnerships and a professor of civil engineering at Idaho State University. Jared Cantrell is the associate director for Strategic Partnerships and Innovation in the Office for Research at Idaho State University in Pocatello, Idaho. Zak Johnson is a professional engineer at Idaho Transportation Department District 5 in Pocatello, Idaho.

A Crack Is Not a Crack: Alkali-Silica Reaction–Induced Cracking

by Dr. Oguzhan Bayrak, University of Texas at Austin

This article, which is the seventh article in this series, focuses on cracking induced by alkali-silica reaction (ASR) and the structural implications of ASR-related cracks. Let us start by acknowledging that stewardship of our concrete bridges is receiving significant national attention—and rightfully so. We have come a long way since the days of the initial development and implementation of the Dwight D. Eisenhower National System of Interstate and Defense Highways. Starting with that effort in the 1950s, we have built a network of controlled-access highways in the United States. As the concrete bridge industry continues its work to expand, improve, and build upon our established system of highways, a key priority is to maintain the existing inventory of our bridges with consideration for public safety and the responsible use of resources. Within this context, the structural evaluation of bridges affected by ASR is a significant challenge. To address this challenge for all concrete structures (not just bridges), *fib* (International Federation for Structural Concrete) published a groundbreaking structural evaluation guideline within the *Model Code for Concrete Structures 2020* (MC 2020).[1] The objective of MC 2020 is to provide uniform guidance for the design of new structures and the evaluation of existing structures. This article will reference MC 2020 recommendations to draw the attention of the concrete bridge community to MC 2020 and the benefits it offers for the structural evaluation of ASR-affected concrete bridges.

Context: What Is Alkali-Silica Reaction?

ASR is a chemical deterioration mechanism that affects concrete structures; it is a particular concern in those structures that were built before our current knowledge on this issue was developed. In general terms, ASR occurs when reactive aggregates and an alkali pore solution in concrete form a hygroscopic gel as a reaction product. By its very nature, this reaction product expands as it absorbs water. At some point when the expansive pressures internal to concrete reach the tensile strength of concrete, the concrete element will develop ASR cracks. **Figure 1** shows a generic example of ASR cracking. For new construction, the use of fly ash (as approximately 25% of cementitious materials) within the concrete mixture and other mitigation techniques offer effective ways to manage reactive aggregates. By using an appropriate dosage of fly ash, we can mitigate expansion and cracking due to ASR. For older structures, we must have effective ways to identify the presence of ASR and evaluate its structural implications.

Expansion Mechanics

ASR is a bulk expansion mechanism. In other words, ASR-affected concrete tends to expand in all directions during this chemical process. Research performed

Figure 1. Cracking related to alkali-silica reaction (ASR) in a cube that is reinforced in one direction (into the page). The specimen was cast in 2015, and the photo was taken in 2023 as part of long-term monitoring of ASR-affected reinforced concrete cubes. Photo: Concrete Bridge Engineering Institute.

at the Ferguson Structural Engineering Laboratory at the University of Texas at Austin investigated the mechanics of ASR-affected concrete restrained in a variety of ways.[2] Highlights of the findings regarding the structural behavior of ASR-affected concrete are summarized herein:

- Unrestrained (plain) concrete expands in all directions almost uniformly, although slightly greater levels of expansion may be observed in the direction that the concrete was cast. Setting this subtlety aside, randomly oriented ASR cracks result in expansion in all directions. This expansion mechanism leads to severe degradation of the mechanical properties of concrete. Free expansion levels of 0.4% to 0.5% may reduce the compressive strength, tensile strength, and modulus of elasticity of concrete up to approximately 40% to 60%.[1] The cracking patterns in plain concrete samples are reasonably uniform and are frequently referred to as "map cracking."
- Concrete restrained by the presence of reinforcement in one, two, or three directions expands differently than plain concrete. The presence of reinforcement in one direction serves to restrain expansion in that direction. Figure 1 shows the side of an approximately 19 in. cube that is not restrained within the plane of the photograph but is restrained by the cast-in deformed reinforcement (with machine-cut threads visible at the plates) placed in the perpendicular direction (that is, into the page). The restraint in one direction results in reduced expansion in the same direction (not visible in Fig. 1). Similarly, the presence of reinforcement in two directions creates a preferential expansion in the direction that lacks reinforcement. For example, in bridge decks, the preferential expansion direction would be the direction of gravity because bridge decks do not have through-thickness reinforcement. If there is reinforcement in all three directions, concrete benefits from the confining effects in all directions and cracking observed in such cases tends to be far less severe. Note that external restraints (for example, rigid supports) provide a similar effect.

With consideration given to the differences observed in free and restrained expansions, MC 2020 provides a comprehensive guideline that can be useful in conducting structural evaluations for concrete bridge components and therefore overall evaluation of the bridges themselves.

Diagnosis

As previously discussed, and as shown in Fig. 1, ASR-affected structures commonly display map cracking. The observed cracking pattern is influenced by internal (reinforcement) and external, boundary restraint conditions. In addition to map cracking, in some cases, ASR-gel exudation, dark reaction rims, and other features of ASR may be apparent on the surface. Observing such crack patterns and ASR features is the starting point of structural evaluation. Ultimately, petrographic analysis is necessary to identify the root cause of such cracking, as there are other conditions such as restrained shrinkage and delayed ettringite formation (DEF) that may result in similar cracking patterns. (See the Summer 2018 and Spring 2019 issues of *ASPIRE*® for in-depth information on ASR and DEF.)

To this end, MC 2020 states:

> *Presence of ASR gel in hardened concrete, as observed in petrographic analysis, is necessary. However, this condition is not sufficient to attribute distress to ASR. Cracking observed in the microstructure of concrete should be tied to ASR gel pockets and expansions observed by those gel pockets. It is possible to identify some ASR gel in many concrete structures but presence and extent of ASR may or may not be sufficient to cause distress (i.e., cracking) observed in the structure.*

MC 2020 also states:

> *ASR-affected structures shall be diagnosed by using petrographic analysis. A representative number of cores shall be extracted from affected portions of the structure under investigation and a petrographer shall examine the cores to positively identify the root cause of the degradation mechanism as being ASR.*

After ASR is positively identified, it becomes necessary to conduct an extent-of-condition analysis. This analysis will result in grouping various locations and components in the bridges based on their levels of ASR damage, ranging from no ASR degradation to mild, moderate, and severe levels of ASR degradation. In this way, a formal methodology can be established for subsequent actions.

Monitoring Bridges Affected by Alkali-Silica Reaction

A plan should be developed to monitor the expansion behavior of the key structural components, in particular those components displaying high levels of ASR degradation and those with low levels of excess design margin. With regard to monitoring, MC 2020 specifies the following:

> *ASR-affected structures shall be monitored to identify and quantify progression of damage. Monitoring expansions and/or engineering strains in three principal directions is necessary to inform validity to the structural evaluations performed. … Use of crack indexing, mechanical measurements, mechanical property degradation to estimate expansions in one or more directions under consideration shall be permitted.*

Structural Evaluation of Bridges Affected by Alkali-Silica Reaction

MC 2020 adopts a framework of analysis known as Levels of Approximation (LoA). This approach is useful to calibrate the amount of engineering effort deployed in structural evaluation for a particular case. LoA I represents the most conservative analysis, and LoA IV is the most rigorous, time-consuming analysis.

The MC 2020 recommendations for the structural evaluation of ASR-affected structures are as follows:

> *LoA I analysis can be performed by making conservative assumptions. In this level of approximation, the most damaged location in the structure can be assumed to represent all areas of the structure and the impact of ASR on structural performance can be performed on the basis of this assumption. The use of linear and nonlinear analysis techniques to estimate the additional demands imposed on structural components can be deemed*

appropriate by the engineer provided that the objectives of structural analysis are consistent with the simplifying, albeit conservative, assumptions made. The method of analysis chosen by the structural engineer has to be benchmarked against experimental data available in the literature. For the purpose of benchmarking large-scale tests should be preferred over small-scale tests, since large-scale specimens will be less impacted by alkali leaching and surface effects seen in all experimental programs that focused on ASR.

LoA I analysis typically provides overly conservative estimates of demands and capacities. In some cases where the structural design is not governed by load application, LoA I analysis may prove to be satisfactory to ensure structural adequacy. If such is not the case a more refined analysis (LoA II) can be performed. In this level of approximation, a thorough mapping of ASR cracks and identification/classification of areas with varying levels of ASR damage will be necessary. Portions of the structure subjected to water ingress or exposure to high levels of relative humidity are likely to display greater levels of ASR damage, i.e. ASR-induced expansions. The severely-, moderately-, and slightly-damaged portions of the structure should be modelled in such a manner that different expansions/displacements can be imposed on different parts. This level of refinement is likely to reduce demands imposed on the structure by ASR, as it provides a more realistic approximation between a numerical model and actual structure experiencing ASR-damage. Once again, equivalent linear or nonlinear techniques can be employed depending on the primary objectives of analysis.

LoA III analysis necessitates modelling the interaction between the expanding concrete and reinforcement explicitly, in addition to making accommodation for variation of ASR damage throughout the structure. This level of refinement is only justified in checking portions of a structure to check reinforcement yielding or in areas of low structural margin to gain improved insights on the reinforcing cage, ASR-damaged concrete interaction.

LoA IV analysis is the highest level of analysis possible to get the best estimate, albeit with the least level of conservatism implicit in structural analysis. This level of refinement requires modelling nonlinear material response of concrete, reinforcing bars, and effects of confinement explicitly and through experimentally-verified models. Computational effort required by this level of refinement may or may not be justified depending on the cost of the required structural retrofits. In other words, it may be simpler and more cost-effective to deploy structural retrofit than try to conduct LoA IV analysis, given the need for experimental verification or benchmarking.

To explore the LoA approach, let us take a case from a research study completed at the Phil M. Ferguson Structural Engineering Laboratory in 2009.[3] In that study, the shear strength of reinforced concrete bent caps with details that comply with the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[4] was evaluated for short (strut-and-tie method shear) and long (beam shear) shear spans. Interested readers are encouraged to read the complete report. To better understand the MC 2020 recommendations, let us consider the compressive strength of the concrete obtained by testing cores extracted from specimens nR1 (no ASR) and R2 (with ASR). The compressive strengths of concrete in nR1 and R2 were 7.2 ksi and 4.2 ksi, respectively. Importantly, ASR expansions caused shear reinforcement to yield (strain = 0.63%) before testing. Without the benefit of the large-scale test results, we may conclude that the shear strength of the bent cap must have been compromised by substantial reduction in the compressive strength of concrete and stirrups that yielded due to ASR expansions before load testing. As can be seen in the test results (**Fig. 2**), the structural capacity (that is, the shear strength) is "not compromised." That is to say, the load-carrying capacity of the specimen with ASR degradation is higher than the capacity of the specimen without any ASR damage. However, as a structural engineer, I would recommend ignoring any additional capacity observed in the specimen damaged by ASR or any other active degradation mechanism. This observed behavior can be explained by taking a deeper look into the core testing. Triaxially confined concrete within the structural core of a bent cap behaves quite differently within its structural context than it does after that structural context is altered through coring. After the core sample is extracted from the bent cap, it loses the confining stresses that are present within the structure. Understanding this behavior helps us make decisions in the structural evaluation process. Importantly, if we were to use the compressive strength of unconfined cores directly with the expressions for shear strength in the AASHTO LRFD specifications, we would calculate greatly reduced capacities—which would be in conflict with the observed behavior summarized in Fig. 2.

The test results presented in Fig. 2 show that there is no adverse impact of ASR on the shear strength of the bent caps discussed in this example. For the ASR-expansion levels tested within the beam R2, ASR did not reduce the shear strength. This observation can simply be used in an LoA I or II analysis. Alternatively, it is also possible to use nonlinear finite element analyses, bring the interaction between the ASR-expansions and reinforcing cage into the picture, reduce the compressive strength of unconfined concrete substantially (from 7.2 ksi to 4.2 ksi in this case), and numerically restore the structural context (that is, explicitly account for the confining stresses imposed on ASR-affected concrete numerically within the analysis). When done properly, the results should match the observed experimental behavior. This type of approach takes us toward LoA III and IV analyses. In this example, leveraging the observed structural performance in a representative structural-scale test will drive the structural evaluation to an expedient conclusion. In more complicated cases, a detailed analysis could be appropriate. The LoA framework described within MC 2020 accommodates different approaches and emphasizes the importance of benchmarking. The example discussed herein highlights the importance of confinement. As long as the integrity of the reinforcing cage that is confining the ASR-affected concrete can be maintained, concerns over structural performance can be directly answered, as specified in MC 2020:

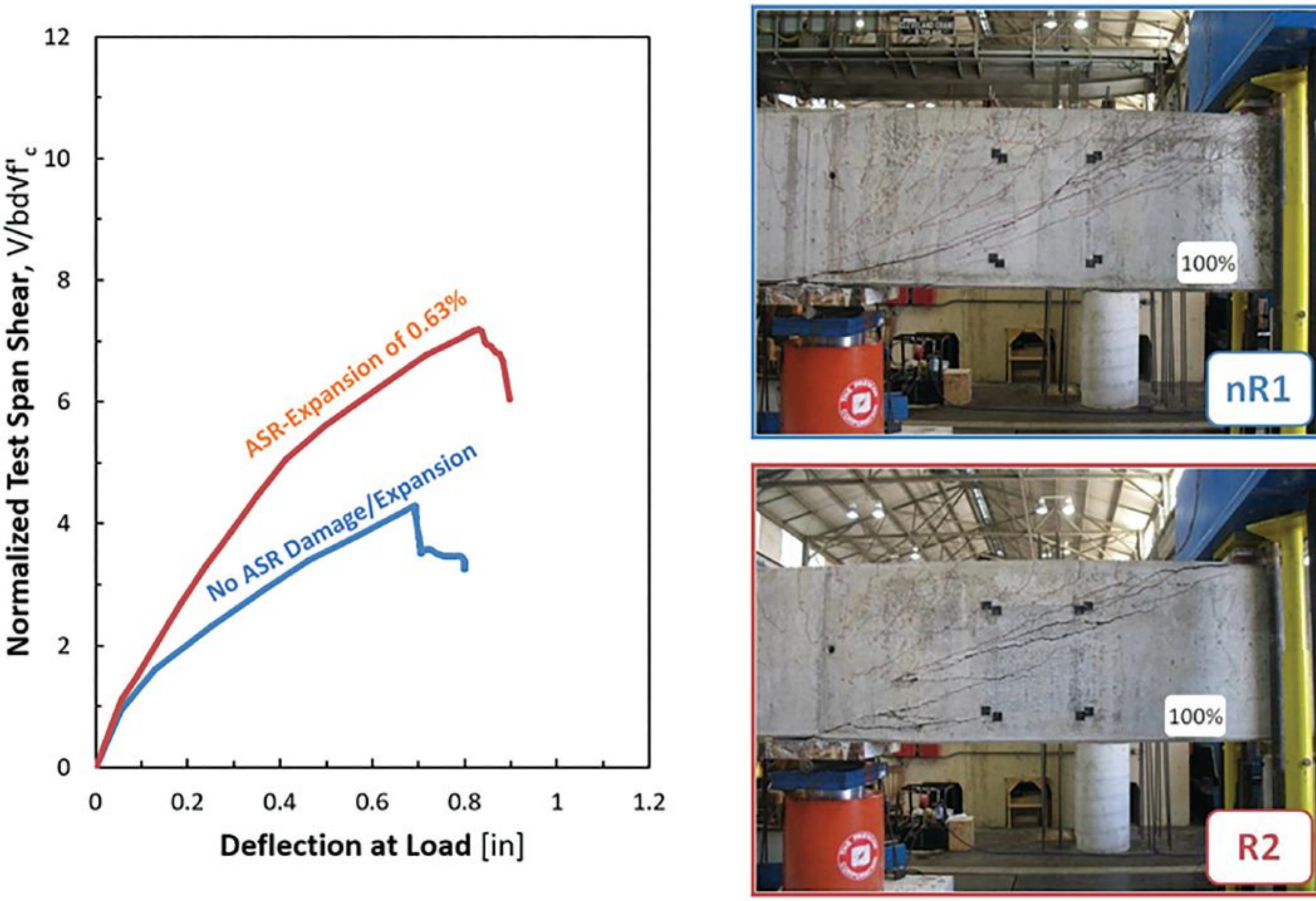

Figure 2. Load deflection behavior of a large-scale reinforced concrete bent cap specimen damaged by alkali-silica reaction (R2) compared with a control specimen (nR1).[3] Figure: University of Texas at Austin.

The potential of reinforcing bar fracture in ASR-affected concrete structures shall be examined. If reinforcing bar bend diameters of the ASR-affected structures are in compliance with those of Model Code 2020, the potential of reinforcing bar fracture shall be taken as negligibly small.

This requirement in MC 2020 is informed by findings from the investigation of reinforcing bar fractures observed in Japan. Japanese researchers Imai et al.[5] conducted a root-cause analysis and concluded that the tight mandrel diameters used in Japan, ASR expansions, and corrosion due to high chloride concentrations collectively led to the observed bar fractures. That is to say, the ASR expansion and the corrosion of carbon steel served as stressors on the "precracked" bar bends due to the tight mandrel diameters. MC 2020 addresses these field observations by suggesting that designers use the mandrel diameters recommended in MC 2020 (and American Concrete Institute, ASTM International, and AASHTO standards in the United States), as those diameters will render bar fracture at 90-degree bends to be a concern of a "negligibly small" magnitude.

Concluding Remarks

As discussed in this article, if ASR is identified to be the root cause of cracking, the MC 2020 recommendations can provide a formal approach to evaluate the appropriate condition analyses, structural monitoring, and structural evaluation processes. Understanding the root causes of cracking is paramount to developing solutions. Not all cracks are created equal. After all, a crack is not a crack.

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EDITOR'S NOTE

Previous articles in **ASPIRE** ***have covered various aspects of alkali-silica reaction. For more information on this subject, see the Winter 2021, Fall 2020, Spring and Summer 2019, and Summer 2018 issues of*** **ASPIRE.**

Raising the Bar for Better Bridges—Bar Reinforcement Support Standard for Reinforced Concrete

by Gregory Clauson, Concrete Reinforcing Steel Institute

In every concrete element, whether cast-in-place (CIP) or precast concrete, the reinforcement must be held at the correct elevation until the concrete is placed and hardened. Small devices—beam bolsters, high chairs, continuous chairs, slab bolsters, and other shapes—perform this duty. These devices are typically made from steel wire, plastic, or precast concrete, and they hold reinforcement in place during concrete placement and vibration. Reinforcing bars can sag or float when too few supports or weak supports are used, reducing concrete cover and thereby increasing the risk of corrosion or misalignment of the reinforcement. For many years, contractors relied on guidance about reinforcement supports from manuals, but there was no nationally recognized standard defining minimum performance requirements. Practices varied, and the quality of reinforcement support products and their installation—including load capacity, corrosion protection, and the ability to hold bars at the specified elevation to maintain concrete cover during placement and vibration—was inconsistent from project to project.

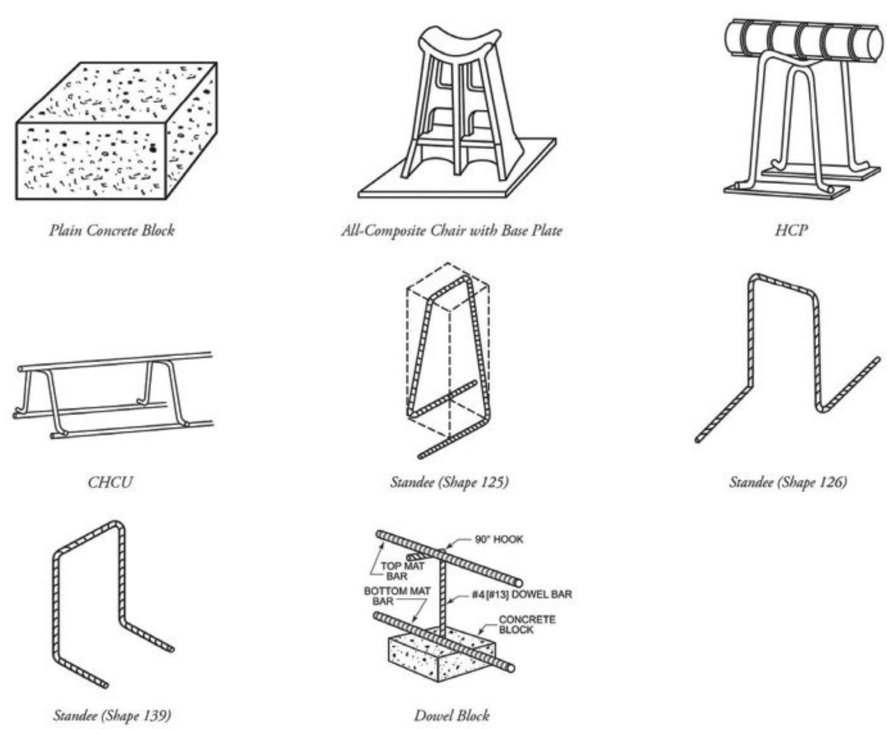
Developing a Consensus Standard

In 2013, the Concrete Reinforcing Steel Institute (CRSI) became an ANSI-accredited Standards Development Organization, which allowed CRSI to assemble balanced committees of designers, contractors, and manufacturers to develop consensus standards. Their first effort, *Supports for Reinforcement Used in Concrete* (ANSI/CRSI RB4.1),[1] was published in 2014. Unlike earlier guidance, the ANSI/CRSI RB4.1 standard uses mandatory language so that it can be cited in codes and project specifications. After being adopted and incorporated in the American Concrete Institute's *Specifications for Structural Concrete* (ACI 301-16),[2] ANSI/CRSI RB4.1 became the national standard for bar support performance requirements. Work is now underway to have it referenced in the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications* and *AASHTO Bridge Construction Specifications*. In April 2022, ANSI recorded final action on ANSI/CRSI RB4.1-2022,[3] a revision of the 2016 version of the standard, which reaffirmed the performance requirements and clarified phrasing.

Performance Requirements and Testing

ANSI/CRSI RB4.1 considers bar supports to be engineered components rather than generic accessories. The 2016 edition introduced four key tests: a load test to measure how much weight a support can carry, an impact test to check resilience under accidental collisions, a water absorption test for plastic supports, and a concrete consolidation test to ensure that supports do not create voids or impede concrete flow. Bar support manufacturers must declare a load category—Non-Rated, 200 lb, 400 lb, 600 lb, or 800 lb—and report pass/fail results for impact, water absorption, and consolidation tests.

A variety of reinforcing bar supports of various materials and configurations can be used when placing reinforced concrete. *Supports for Reinforcement Used in Concrete* ANSI/CRSI RB4.1 provides objective criteria for selecting and approving reinforcement supports based on load category and corrosion protection class. All Photos and Figures: Concrete Reinforcing Steel Institute.



Workers place epoxy-coated reinforcing steel for a bridge deck, with bars supported at the specified elevation to maintain concrete cover during concrete placement.

While states vary in how they implement ANSI/CRSI RB4.1 testing requirements, the standard establishes the test methods and reporting requirements. The following is an example of the RB4.1 load test rather than a full-state-approval process.

In the load test, the support is loaded vertically through the reinforcement on a steel base plate until the rated load is reached and held for one minute, and then the specimen is evaluated based on deflection and damage. The load category is tied directly to the minimum test load in pounds, and the support must sustain that load without breaking or permanent deformation while limiting deflection to 0.25 in. Continuous supports are evaluated using multiple loading points to represent distributed loading. The impact, water absorption, and concrete consolidation tests are performed and reported independently as pass or fail.

Classes of Corrosion Protection

ANSI/CRSI RB4.1 categorizes supports by corrosion protection class rather than by material. Class 1 supports are intended for moderate to severe exposures; these types of supports are often plastic or protected by plastic. Class 1A supports include additional barrier systems compatible with epoxy-coated reinforcement. Class 2 supports provide enhanced metallic or nonmetallic protection for moderate environments. Class 3 supports have no special coating and are used only where blemishes are acceptable or the support is fully embedded. The standard does not dictate that supports in a particular class must be metal, plastic, or concrete; plastic supports can be designed and tested to qualify for any class, provided the specified performance and class requirements are met.

Adoption of ANSI/CRSI RB4.1 by Agencies and Codes

Major decision-makers have embraced ANSI/CRSI RB4.1. The U.S. Department of Defense's (DOD's) *Unified Facilities Guide Specification (UFGS) 03 30 00: Cast-in-Place Concrete*,[4] references ANSI/CRSI RB4.1 and requires compliant reinforcement supports on all DOD projects. UFGS 03 30 00 also calls for appropriate corrosion-resistant supports when corrosion-resistant reinforcement is used, prohibits using supports as runways for concrete conveying equipment, and requires supports for coated or galvanized bars to be coated at least 50 mm beyond the point of contact. State transportation agencies have adopted similar measures. For example, the Iowa Department of Transportation's Materials Instructional Memorandum 451.01[5] references ANSI/CRSI RB4.1 for approval of plastic bar supports.

ANSI/CRSI RB4.1 is also cited in municipal bid and construction specifications, including the City of Republic, Mo., the Town of Juno Beach, Fla., and the Borough of Rockaway, N.J., which specifies ANSI/CRSI RB4.1 Class 1A for epoxy-coated reinforcement. By using ANSI/CRSI RB4.1 with ACI 301, owners can enforce consistent requirements across projects.

What ANSI/CRSI RB4.1 Does Not Cover

ANSI/CRSI RB4.1 only addresses support performance and corrosion protection. It does not prescribe how many supports to use or where to place them. Those decisions are typically made by the contractor or producer based on constructability, reinforcement stability during placement, and the project's cover and tolerances, and are governed by the project documents. The standard also warns that supports should never be used to bear heavy equipment or serve as runways. Designers must therefore combine requirements from ANSI/CRSI RB4.1 with their own detailing standards and the requirements of ACI 301 and the *Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318R-19)*[6] to determine support layout and quantity.

Benefits for Contractors, Precast Concrete Manufacturers, and Transportation Agencies

Specifying ANSI/CRSI RB4.1-compliant supports reduces the risk of reinforcement displacement during concrete placement and vibration by requiring that supports meet defined load and impact performance. This helps maintain intended bar position and concrete cover and, in turn, reduces the likelihood of durability problems due to inadequate cover or corrosion.

Using the standard classification system also simplifies communication with contractors, designers, and inspectors because support selection can be verified against objective criteria rather than subjective product descriptions.

For public owners, referencing ANSI/CRSI RB4.1 ensures consistency and durability. Performance tests give confidence that supports will keep reinforcement in place and withstand incidental impacts. Following mandates to pair coated supports with epoxy-coated bars ensures that corrosion protection will be preserved. Uniform adoption of the standard across agencies reduces the likelihood of disputes over acceptable products and simplifies inspection and quality assurance.

Implementation Considerations

Specifiers should reference the latest version of ANSI/CRSI RB4.1 and specify the required class for each concrete element. Contractors should use certified products that meet the declared load category and pass the specified performance tests. Supports must not be overloaded or used to carry heavy equipment. Inspectors should verify compliance and proper coating where required.

Conclusion

ANSI/CRSI RB4.1 plays a crucial role in the quality and longevity of reinforced concrete. By defining load-tested performance requirements, corrosion-protection classes, and objective acceptance criteria, ANSI/CRSI RB4.1 gives engineers, manufacturers, and owners a common language and measurable expectations. The adoption of this standard in ACI 301, UFGS, and the requirements of several state transportation agencies demonstrates its relevance to both private and public projects. For precast concrete manufacturers, CIP contractors, and transportation agencies seeking to improve durability and reduce variability, referencing ANSI/CRSI RB4.1 is a practical step to ensure consistent reinforcement support performance and reliable reinforcement placement in both precast concrete elements and CIP bridge structures.

ANSI/CRSI RB4.1, *Supports for Reinforcement Used in Concrete*, mandates that supports for epoxy-coated reinforcing bars must be nonconductive to avoid compromising the coating's corrosion protection.

Continual efforts to monitor and update ANSI/CRSI RB4.1 will allow the industry to respond to new materials and methods. As new materials or conditions emerge, the standard will likely evolve to address them. Meanwhile, consistent adoption of ANSI/CRSI RB4.1 across projects fosters a culture of quality and reliability. By embracing ANSI/CRSI RB4.1, contractors, precast concrete manufacturers, and public owners can contribute to a more uniform, durable built environment.

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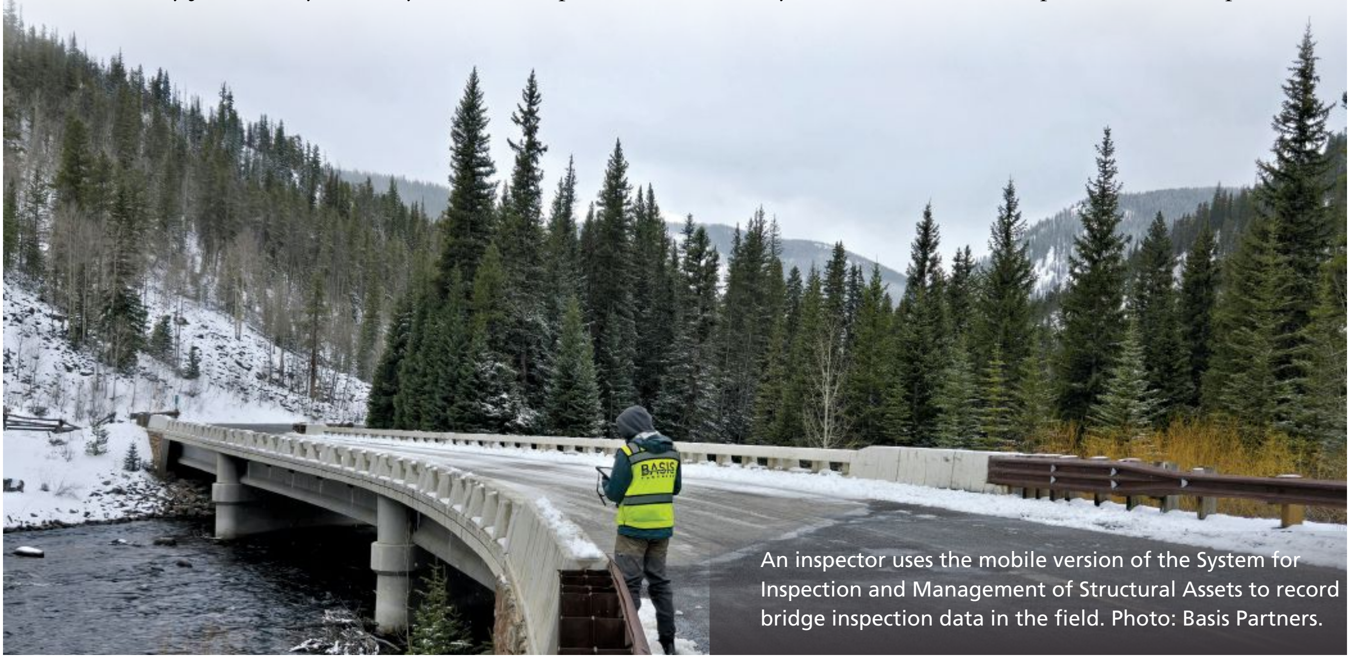
Gregory Clauson is the Northeast Region manager and National Transportation manager for the Concrete Reinforcing Steel Institute (CRSI). He leads CRSI's Transportation Task Group efforts focused on integrating ANSI/CRSI RB4.1, **Supports for Reinforcement Used in Concrete*****, into national bridge specifications, working closely with AASHTO, PCI, and allied industry committees to align fabrication and design practices with modern durability and constructability standards.***

Colorado's System for Inspection and Management of Structural Assets:

Modernizing the Future of Structural Asset Management

by Jessica Terry, Berkeley Williams, Stephen Ranum, and Nyssa Beach, Colorado Department of Transportation



An inspector uses the mobile version of the System for Inspection and Management of Structural Assets to record bridge inspection data in the field. Photo: Basis Partners.

In the past, the life story of a Colorado bridge was documented in many places and in many ways. An inspector or designer looking for the history of a single structure might have to hunt through dusty boxes of hard-copy as-builts, search for a specific CD-ROM burned in 2005, or track down a thumb drive sitting in a desk drawer of a regional office. With 27,400 structural assets to maintain in Colorado, this mix of disconnected digital and physical data created a significant barrier to efficient and effective asset management.

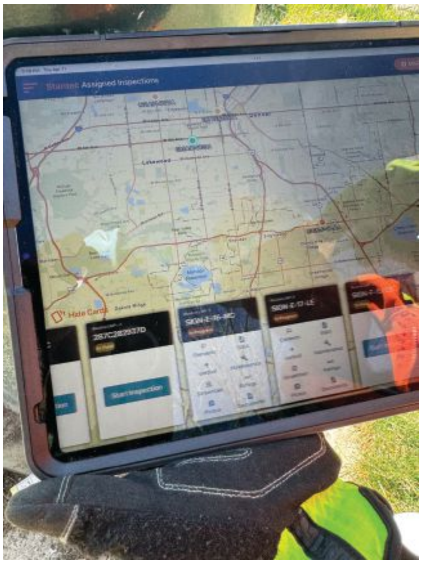
The Colorado Department of Transportation (CDOT) recognized the need for a modern solution that could handle the massive volume of data while remaining focused on the end user. The agency envisioned a one-stop-shop platform designed with field condition data and user experience at its foundation. Such a system would organize data into a clear, actionable narrative and provide the same level of transparency and intuition provided by an electronic health record, which allows a doctor to see a patient's entire history at a glance, from birth (construction as-builts) through every check-up (inspection) and intervention (maintenance and rehabilitation).

The result is CDOT's System for Inspection and Management of Structural Assets (SIMSA), a new national benchmark for transportation safety and asset management. This custom-built, nonproprietary platform has transformed how CDOT manages its vast inventory of structural assets, including more than 3500 CDOT-owned bridges, which are among the 9000 state-owned and off-system bridges in Colorado. Prior to developing SIMSA, CDOT researched and used some available asset-management software products before determining that they would not fully meet the needs of the agency. Developed in partnership with the consulting firm HDR Inc. but fully owned by CDOT, SIMSA was built for long-term operational flexibility, allowing CDOT to adapt quickly to evolving industry requirements, including collaboration with the Federal Highway Administration (FHWA).

SIMSA, which aligns with FHWA's *Specifications for the National Bridge Inventory* (SNBI),[1] has been operational since the summer of 2025. The platform delivers a single, unified database of CDOT's structural assets, which promises to facilitate structural safety and efficient maintenance of Colorado's infrastructure for years to come.

Field First: User-Centric Development

While SIMSA leverages cutting-edge technology, its success is rooted in a "field-first" philosophy. From the outset, the development team prioritized the



The System for Inspection and Management of Structural Assets' map view offers inspectors a helpful interface. Photo: Stantec.

Inspectors can use the System for Inspection and Management of Structural Assets mobile version to tag photos to specific inspections, which saves time by streamlining report generation. Photo: Basis Partners.

needs of the inspectors working on site at bridges, tunnels, and culverts.

To ensure that the platform reflected real-world conditions, software developers visited asset sites alongside bridge engineers, maintenance crews, and inspectors to understand firsthand which data points were most critical for field operations. The result is a system that offers immediate, tangible benefits, including the following:

- Automated workflows: SIMSA's mobile app is accessed by CDOT and consultant bridge inspectors in the field on tablets. It allows inspectors to automatically tag photos to specific inspections and structures so that data-informed reports can be instantaneously generated. This feature eliminates the need for manual photo renaming and cumbersome logs.
- Data integrity: A well-defined backend schema has significantly reduced data cleanup and rework.
- Comprehensive scope: The platform provides actionable condition data for a wide array of assets beyond bridges, including tunnels, culverts, traffic signals, and highway signs. A subset of a structure's documents is automatically included in the sync for inspector's use in the field.

From Reactive to Predictive Management

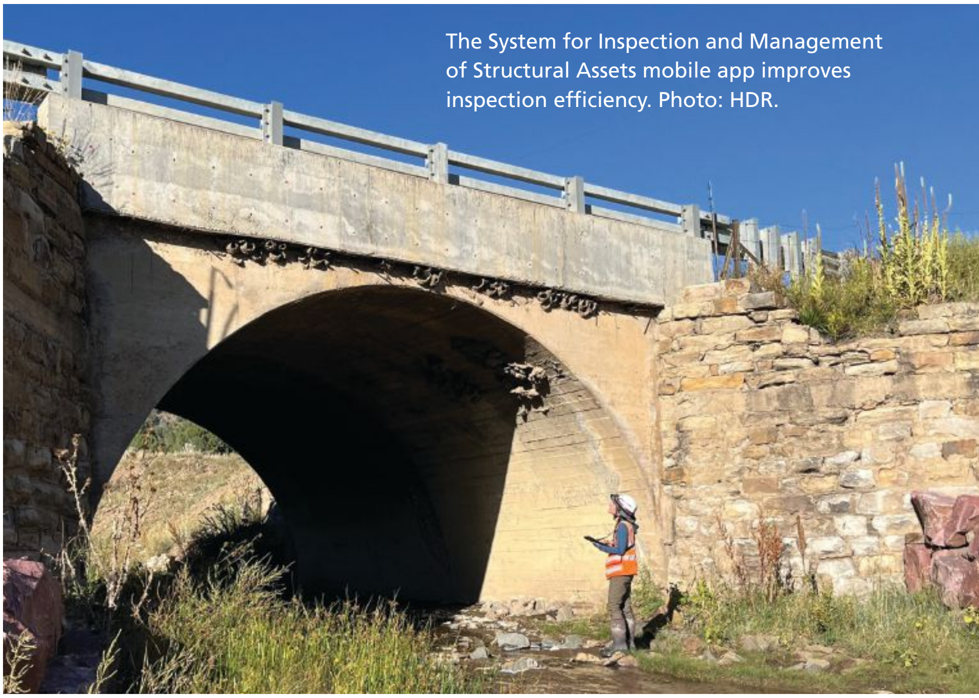
The shift to a centralized, comprehensive dataset has empowered CDOT to move away from a "worst first" maintenance mentality toward a proactive, life-cycle-based strategy. By leveraging precise analytics, CDOT can now implement the following:

- Preventive maintenance: Enhanced data scoring identifies concrete structures that are prime candidates for early intervention, such as deck seals or overlays, before costly rehabilitation is required.
- Strategic bundling: A unified database allows CDOT to bundle routine maintenance tasks within specific geographic project boundaries, maximizing resource efficiency.
- Deterioration modeling: SIMSA provides the bridge-element-level data necessary to predict future infrastructure health, allowing CDOT to optimize limited funding.

SIMSA's advanced filtering capabilities are an important benefit for stewardship. Users can instantly isolate specific categories—such as active, CDOT-owned, or federally-owned concrete bridges—and view them in both map and table formats. From there, managers can drill down into maintenance items, historical load ratings, streambed profiles, and vertical clearances.

Navigating the SNBI Transition

The resilience of the SIMSA platform was put to the test in 2022 when the FHWA published SNBI while SIMSA was being developed. This publication was not a minor update: SNBI represented a major shift for all nationwide state transportation agencies, as it requires an overhaul of bridge inspection data.



The System for Inspection and Management of Structural Assets mobile app improves inspection efficiency. Photo: HDR.

The System for Inspection and Management of Structural Assets dashboard. Photo: HDR.

FHWA's objective for SNBI is to transition from a rigid, "one-size-fits-all" time-based approach to a risk-based inspection model that prioritizes resources for bridges in the most critical condition. SNBI replaces outdated 1995 standards[2] with a modernized, bridge-element-level data structure that improves safety through better technology integration and more-precise condition reporting.

For CDOT, the timing of SNBI was particularly challenging. The agency was already in the midst of its own internal system transformation to SIMSA when the SNBI mandate required a nationwide revision of databases, update of procedures, and the comprehensive retraining of both inspection and bridge teams. Rather than pausing to rebuild the SIMSA system around the changing FHWA requirements, CDOT strategically completed the platform's original data schema first. This approach allowed the team to verify and clean legacy data in this single platform before initiating a comprehensive conversion to the new SNBI format.

The transition to SNBI introduced more than 50 new federally required data fields and complex many-to-one datasets for spans and substructures (see sidebar, "Implementing SNBI"). To prevent data bloat, the SIMSA team redesigned the user interface based on inspector feedback. Instead of flat, exhaustive lists, the system now uses organized, pull-down menus to keep data scannable and actionable.

To ensure compliance well ahead of the first federal submittal deadline for adopting the new SNBI schema (March 2026), CDOT employed a two-track collection strategy:

- Deskside collection: Starting in 2023, office teams coded new fields by reviewing historical plans and Geographic Information System imagery.
- Field collection: Beginning in early 2024, inspectors began capturing new SNBI measurements during regular inspection cycles.

Conclusion

SIMSA is a landmark achievement that redefines the relationship between a transportation agency and its data. By consolidating fragmented information into a single, transparent system, CDOT has pioneered a scalable path for structural asset management that extends beyond Colorado. Guided by an open-source philosophy, CDOT is actively working to share this robust tool with partnering agencies, elevating local excellence into a national best practice. As the industry looks toward a more resilient national bridge network, SIMSA stands as a testament to the power of user-focused, data-driven stewardship.

Implementing SNBI

Implementation of the *Specifications for the National Bridge Inventory* (SNBI)[1] has been proceeding as states begin to capture bridge inspection data in the revised format. This implementation will necessitate development of new database systems, updates to procedures, and training for inspectors and database managers, among other actions.

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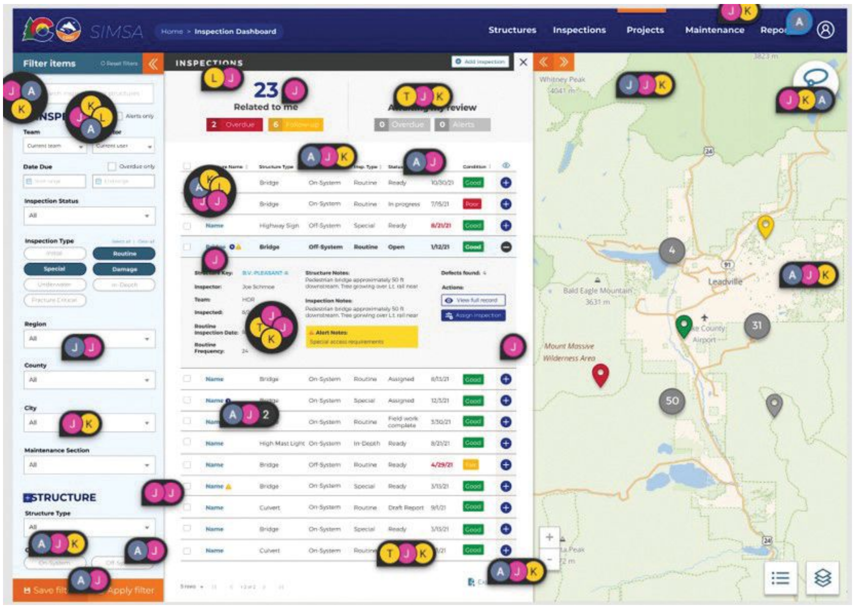
Jessica Terry is a bridge asset engineer, Berkeley Williams in an engineer-in-training in bridge asset management, Stephen Ranum is a data manager, and Nyssa Beach is a statewide bridge asset manager for the Colorado Department of Transportation in Denver.

User input for development of the System for Inspection and Management of Structural Assets. Figure: HDR.



Experimental Evaluation of Overheight Impact Damage and Repair Effectiveness in Prestressed Concrete Bridge Girders

by Dr. Mohamed A. ElGawady, Dr. Haitham AbdelMalek, Francis Ashun, Dr. Mohanad Abdulazeez, and Dr. Ahmed Gheni, Missouri University of Science and Technology, and Dr. Ahmed Ibrahim, University of Idaho

Between 2013 and 2018, more than 78,000 collisions between motor vehicles and fixed bridge objects were reported in the United States, with overheight truck impacts identified as the predominant accident type.[1] While many overheight collisions result in minor damage or cosmetic repairs, a smaller subset leads to severe structural damage, extended closures, and, in some cases, loss of load-carrying capacity that requires girder replacement.

Overheight truck impacts impose a substantial financial burden even when bridges remain standing. A Washington State Department of Transportation (WSDOT) assessment reported repair costs exceeding $14,000 per linear foot of prestressed girder (2025 dollars).[2] For typical multigirder highway bridges, total repair expenditures can rival or exceed the costs of full replacement once traffic control, emergency contracting, and user delays are considered. In many cases, bridge owners are compelled to rapidly make repair-versus-replacement decisions with only limited information regarding the residual capacity and long-term performance of the damaged structure.

Despite the frequency, cost, and operational consequences of overheight vehicle impacts, the experimental literature addressing prestressed concrete bridge girders under impact loading is sparse. Most existing impact research focuses on small-span reinforced concrete beams tested under drop-weight or pendulum-type impactors. While those studies provide insights into local shear mechanisms and contact-force histories, their applicability to full-scale prestressed girders is limited due to differences in span length, prestressing-induced force states, composite deck interaction, restraint conditions, and system-level load redistribution. More importantly, small-scale vertical drop tests do not reliably capture the dominant mechanisms observed in overheight impacts in the field: partial strand rupture, asymmetric loss of prestress, torsional–flexural interaction, and progressive stiffness degradation.

This article presents the experimental component of a pooled-fund program developed to fill that gap.[3] Full-scale prestressed girders representative of department of transportation inventories were subjected to controlled impact loading, followed by systematic residual capacity testing and repair evaluation. When combined with the companion analytical study, described in the Winter 2026 issue of *ASPIRE*®, this work supports a unified postimpact decision framework for bridge owners that may reduce the number of unnecessary bridge and girder replacements while maintaining public safety.

Experimental Design

Fourteen 46-ft-long Missouri Department of Transportation (MoDOT) Type 2 prestressed concrete girders were fabricated for the study. Twelve girders contained twelve 0.5-in.-diameter low-relaxation Grade 270 strands and were detailed in accordance with the ninth edition of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*.[4] Two girders contained 16 strands to quantify the influence of prestressing level. Thirteen girders were detailed to achieve flexure-controlled behavior under design loading; one of the 12-strand girders included increased midspan shear reinforcement to investigate the mitigation of torsion-related cracking (**Fig. 1**).

The experimental program was executed in three operational phases. First, eight girders were subjected to lateral dynamic impacts under varying boundary conditions and impact intensities to replicate overheight vehicle collisions. Second, one girder was tested under quasi-static lateral loading to establish a baseline comparison between static and dynamic response and quantify the extent to which dynamic demands exceed static capacity. Third, 12 girders—2 as-built, 3 damaged, and 7 repaired—were tested in four-point bending to quantify residual flexural strength and to validate repair effectiveness.

Lateral Impact Simulator and Impact Scenarios

Lateral impacts were delivered using a purpose-built impact simulator consisting of a 60-ft-long elevated steel track, of which a 22-ft length was flat and 38 ft was inclined (**Fig. 2** and **3**). The impact bogie had an empty weight of 2800 lb and was designed to withstand the AASHTO LRFD specifications' equivalent static force of 600 kip. The steel frame—measuring 7 ft by 3 ft—was configured to accommodate removable concrete slabs for weight augementation. A steel bumper at the front contained load cells for measuring the impact force. During this experimental program, up to three 1400-lb concrete slabs were installed. With three slabs installed, the total bogie weight reached 7000 lb. The

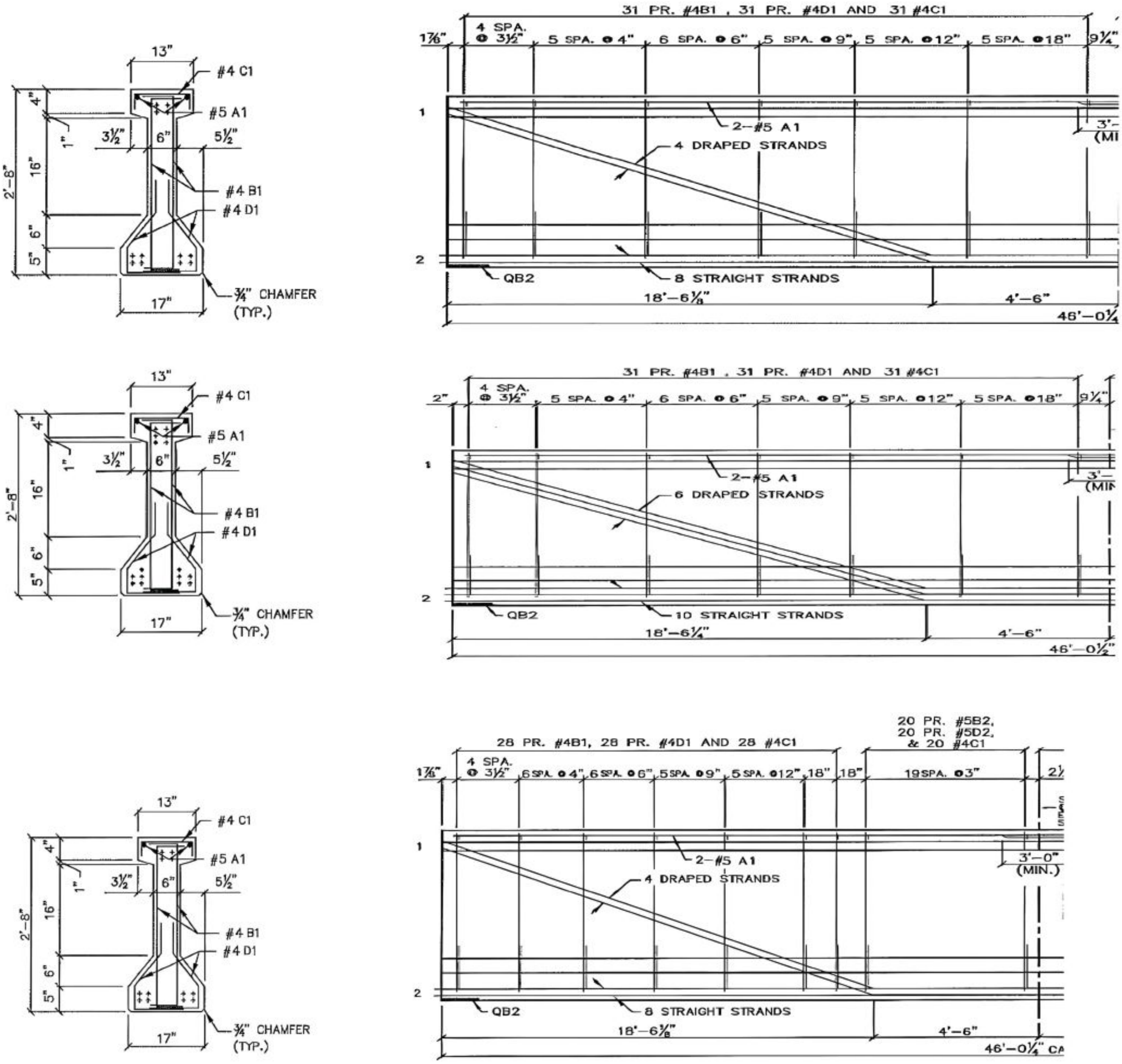


Figure 1. Test specimens' reinforcement details: 12 strands (top), 16 strands (center), and 12 strands with additional stirrups (bottom). All Photos and Figures: Missouri University of Science and Technology.

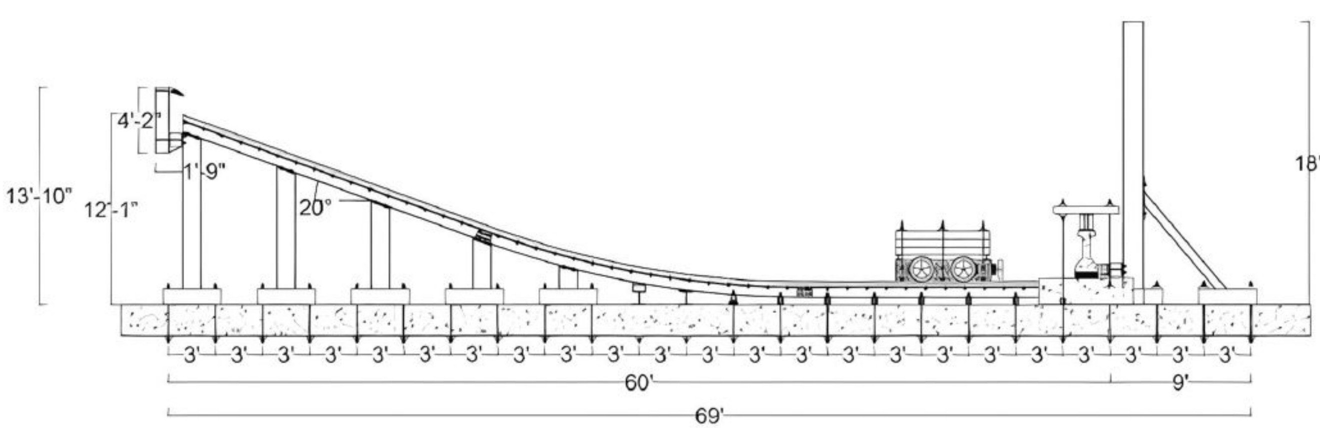


Figure 2. Elevation drawing of the impact testing setup with the 7000-lb steel impact bogie at the bottom of the ramp.

Figure 3. The impact bogie is ready to be released for impact testing of a girder specimen.

bogie, which was equipped with interchangeable impactor heads, served as the surrogate vehicle, providing repeatable kinetic energy under gravity-driven acceleration. A flat head simulated distributed contact consistent with trailer header-bar engagement, and a fork-shaped head reproduced concentrated, asymmetric contact representative of specialized equipment or offset strikes (**Fig. 4**). In all tests, the impactor head

Figure 4. Impactor shapes: flat head (top) and fork head (bottom). These impactors simulate typical tractor trailer and oversize equipment impacts, respectively.

targeted the bottom flange near midspan, consistent with documented strike locations in overheight vehicle incidents. This configuration enabled systematic variation of impact intensity and contact geometry while maintaining controlled repeatability.

Boundary Conditions and System Restraint

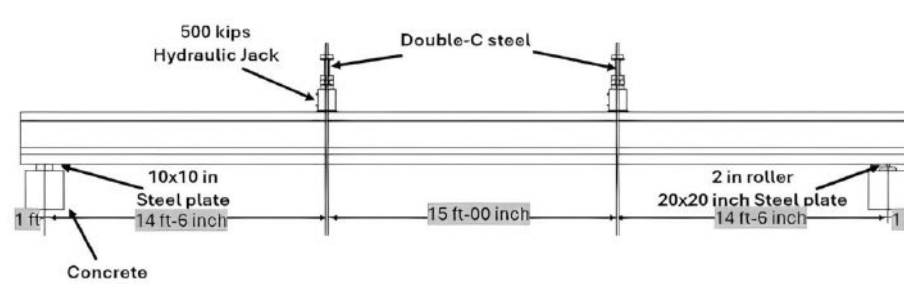
Because a girder in service is restrained by the bridge deck and diaphragms, two primary restraint configurations were employed. In the less-restrained configuration, girders were simply supported, with no top-flange restraint; this configuration allowed free lateral translation and rotation, which provided a lower-bound stiffness case. In the restrained configuration, a continuous steel restrainer was applied along the top flange to simulate the lateral-bracing effect of a reinforced concrete deck. In addition, two specimens were tested with intermediate steel diaphragms. Two built-up intermediate steel diaphrams were installed at the gider web, positioned at midspan with 9-ft spacing. The diaphragm locations are representative of those commonly used to distribute impact energy and stabilize load paths. Together, these testing configurations allowed investigators to quantify how deck restraint and diaphragms influence impact force transmission, lateral displacement demand, and damage localization.

Residual Flexural Strength of Impact-Damaged Girders

Following impact testing, the residual flexural capacities of three girders with approximately 25%, 50%, and 63% loss of prestressing strands on one side were evaluated. A four-point bending test (**Fig. 5**) with incremental monotonic loading and comprehensive instrumentation captured the force–deflection response, crack development, strand response, and failure modes of each girder.

The test results showed a strong, approximately proportionate relationship between loss of prestressing and

Figure 5. The four-point flexural test setup.



reduction of flexural strength. Girders with minor impact damage and no loss of prestress exhibited flexural capacities exceeding estimates based on AASHTO LRFD specifications by approximately 6% to 9%, consistent with typical conservatism in code-based predictions. Reductions of 25% to 63% strand loss resulted in approximately 25% to 62% reductions in flexural strength, respectively, a finding that confirms prestressing continuity as the controlling parameter for postimpact flexural capacity in these specimens (**Table 1**). The results also show that the AASHTO LRFD specifications accurately predict capacity if the damaged strands are discounted.

Repair Using Mechanical Strand Splicing and Carbon-Fiber-Reinforced Polymer Strengthening

Two repair strategies were experimentally evaluated for restoring flexural capacity after overheight impact–type damage: mechanical strand splicing using a proprietary system and externally bonded carbon-fiber-reinforced polymer (CFRP) composites. The strand-splicing program investigated four girders with strand losses between 17% and 33%. Repairs followed a consistent field-relevant sequence, including

- concrete removal and strand exposure,
- staggered splice installation,
- reintroduction of effective prestress by tensioning to approximately 70% of strand yield,
- patching with high-strength non-shrink grout with an average design compressive strength of 10,200 ksi, and
- epoxy injection into diagonal cracks along the web to restore stiffness and limit crack reopening (**Fig. 6**).

Each repaired specimen was then tested in four-point bending in the same manner as the undamaged control girders.

Table 1. Summary of the residual strength results

Girder number	Girder status	Strands		Girder compressive strength f'_c, ksi	Flexural strength, kip-ft		Test strength/ AASHTO estimate
		Design number of strands	Damaged, %		Estimated per AASHTO LRFD specifications	Test	
G02	Undamaged control	12	0	10.2	1053	1118	106%
G14	Undamaged control	16	0	10.5	1270	1377	108%
G03	Damaged	12	25	10.6	852	855	100%
G04	Damaged	12	50	11.0	565	526	93%
G06	Damaged	16	63	11.4	476	521	109%

Figure 6. After the girders were impact tested, these steps were taken in the strand splicing repair.

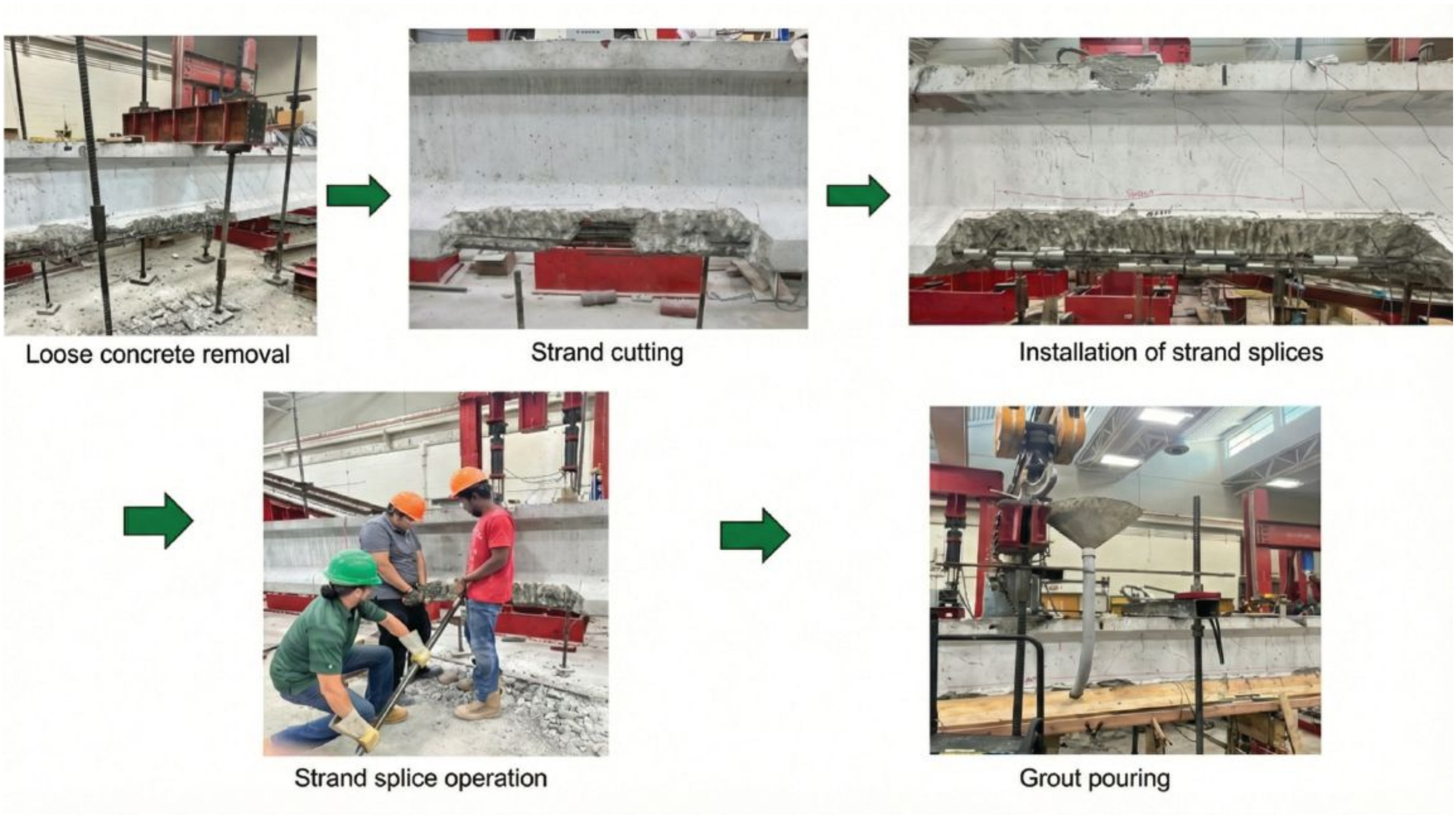


Figure 7. Failure of the carbon-fiber-reinforced polymer (CFRP) repaired girder; only the CFRP-repaired specimens were tested upside down.

Mechanical Strand Splicing

Testing results showed that mechanical splicing restored flexural strength effectively when strand loss is moderate and splice-region confinement is adequate. Girders with 17% strand replacement reached 84% of the control girder's strength and girders with 25% strand replacement reached approximately 95% of the control girder's strength, with initial stiffness and crack development comparable to the undamaged (control) specimen. As the percentage of spliced strands increased, performance became more sensitive to splice-region detailing and local stress concentrations. The impact-damaged girder repaired at 33% strand loss reached about 60% of the control girder's capacity. The relatively lower postrepair capacity for this girder was influenced by cumulative preexisting damage and local cracking at splice chuck locations. In contrast, a companion 33% strand-loss specimen that included added confinement around the splice region reached more than 91% of the control girder's strength, demonstrating that confinement reinforcement can mitigate premature splice-related failures and improve splice integration at higher damage levels. Instrumented tensioning of the spliced strand also showed that the manufacturer's torque-based tensioning procedure overestimated strand strain by approximately 18% on average, indicating the need for improved field calibration and quality-control checks when applying this method in practice.

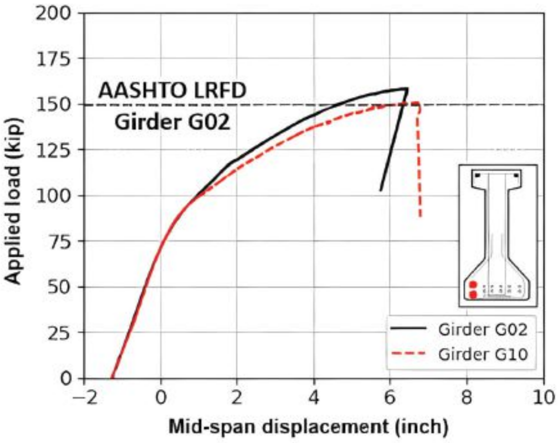
CFRP Strengthening

Externally bonded CFRP composites were evaluated as a flexural strengthening strategy for girders with 17% and 33% strand loss. No strand splicing was done as part of this work. A wet layup system designed in accordance with the American Concrete Institute's *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures* (ACI PRC-440.2-17)[5] was installed with longitudinal plies (individual layers of fiber reinforcement) on the soffit and U-wrap anchorage to mitigate debonding. Successful performance required thorough surface preparation, including crack injection, patching, corner rounding, and appropriate surface roughness. Pull-off testing[6] indicated excellent bond quality, with failure occurring within the concrete substrate rather than at the interface or within the CFRP system. Under four-point bending, both CFRP-repaired girders exhibited brittle failure initiated by U-wrap rupture followed by longitudinal ply debonding (**Fig. 7**). Despite the brittle failure mode, strength restoration was substantial: the 17% damage girder and the 33% damage girder exceeded the control girder's strength by 23% and 16%, respectively (**Fig. 8**). These results confirm that a CFRP system can restore and even increase flexural strength for moderate damage levels, provided that anchorage and bond quality are properly controlled; however, the repair may shift the failure mode from ductile flexure to anchorage-controlled brittle behavior.

Conclusion

This experimental program provides a practice-relevant dataset linking overheight impact–type damage states, loss of prestress due to strand breakage, and repair effectiveness in full-scale prestressed concrete girders. The testing confirms that postimpact flexural performance is governed primarily by the magnitude of prestress loss, with strength and stiffness reductions tracking strand damage in an approximately proportionate manner over the range studied. Minor impact damage without strand loss produced capacities slightly above AASHTO LRFD specifications' predictions, whereas asymmetric damage levels representative of overheight strikes reduced flexural capacity by

Figure 8. Force-deflection curve of repaired girder G08 (33% strand loss with carbon-fiber-reinforced polymer strengthening) and undamaged control girder G02.



approximately 25% to 62% for strand losses between 25% and 63%.

Repair tests establish that both mechanical splicing and CFRP strengthening can be effective, but their reliability depends on the damage mechanism caused by the overheight impact. Strand splicing is most appropriate when the restoration of prestressing continuity is essential; it can recover 84% to 95% of baseline strength for moderate strand loss, and it can recover more than 90% of baseline strength for higher strand losses when splice-region confinement is included. CFRP strengthening can restore and even exceed baseline flexural strength of girders with moderate strand loss.

Most importantly, the experimental evidence closes a long-standing gap in bridge engineering practice by providing full-scale confirmation of how overheight impact–type damage and asymmetric loss of pretress affect residual flexural strength and repair outcomes.

Acknowledgments

Special thanks go to the Transportation Pooled Fund Study TPF-5(462), with contributors from the departments of transportation in Missouri, Alaska, Idaho, Mississippi, Ohio, and Texas, along with the Federal Highway Administration. The financial support, vision, and commitment of the study contributors have been instrumental in pushing the boundaries of current practices and advancing innovative concepts. Additional support was provided through the Missouri Center for Transportation Innovation, whose involvement helped facilitate coordination of the project. The authors also acknowledge the in-kind support and technical contributions of industry partners Sika USA, Simpson Strong-Tie, General Technologies (GTI), and Nucor-Yamato Steel, whose materials, expertise, and collaboration were essential to the successful execution of the experimental program.

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Evaluation and Health Monitoring of the Veterans' Glass City Skyway Stay Cables

by Dr. Serhan Guner, University of Toledo, Ohio

Stay cables are the most critical and vulnerable components of cable-stayed structures. While corrosion is typically the primary driver of deterioration of internal steel strands, other risks to strands include section loss, wire breakage, and moisture infiltration.

Opened to traffic in 2007, the Interstate 280 Veterans' Glass City Skyway (VGCS) in Toledo, Ohio, employs a special stay-cable design to mitigate these risks. However, the unique design features present challenges for standard nondestructive evaluation (NDE) strategies. This article reports on a study to identify practical NDE methods to monitor these stay cables and establish a long-term assessment plan.[1,2]

Unique Design Features and Challenges

The VGCS stay-cable system incorporates several layers of protection that influence inspection protocols and dictate the selection of NDE technologies. These protection layers include the following:

- Epoxy-coated strands with interstices filled with epoxy: This strand type provides a high level of corrosion resistance by blocking moisture travel into and within the strand itself (**Fig. 1**). However, the coating is vulnerable to "nicking" (abrasive damage) during installation or through interstrand contact over the bridge's service life. Such localized damage can lead to coating delamination, potentially exposing bare steel to corrosive elements.
- Large-diameter stainless steel sheathing: The stay cables encase between 82 to 156 strands in a 0.25-in.-thick, 18-in.-diameter stainless steel skin that serves as the primary external barrier. The sheathing provides superior protection against the environment; however, due to the significant diameter of the sheathing and the resulting standoff distance, many traditional electromagnetic NDE methods designed for smaller, more compact cables are technically inadequate or do not provide sufficiently accurate data.
- Ungrouted configuration and cheese plates: Instead of traditional grouted stay cables, the VGCS has an ungrouted design using "cheese plates" (**Fig. 2**) to separate individual strands. This configuration is advantageous because it allows for the extraction and replacement of individual strands. However, while laboratory testing of an extracted strand provides definitive data, removing a strand is a destructive and logistically intensive process reserved for high-concern scenarios where NDE methods have already indicated significant section loss.

A Risk-Based Nondestructive Evaluation Strategy

Deterioration in cable-stayed bridges typically initiates at or near the anchorage zones, where moisture tends to collect and stresses are concentrated, rather than along the midlengths of the stay cables. Consequently, investigators for the study described herein implemented a tiered, risk-based approach: evaluating high-risk, accessible areas first, then deploying advanced methods for the midlengths of the stay cables if evidence of significant deterioration was detected.

Advanced Borescope Visual Inspection

Visual inspection remains a high-reliability method for internal assessment when access is available. The research team used existing drain holes in the sheathing at the lower ends of the stay cables as access ports for visual inspection (**Fig. 3**).

- Methodology: The team deployed high-definition dual-camera borescopes with four-way articulation to navigate the internal stay-cable environment. This method allowed for a 360-degree view of the strands and the inner surface of the sheathing.
- Findings: Across 14 inspected stay cables, more than 250 photographs

Figure 1. Cross section of a seven-wire stay-cable strand featuring epoxy coating and filled interstices designed to prevent moisture migration and corrosion. All Photos and Figures: Dr. Serhan Guner.

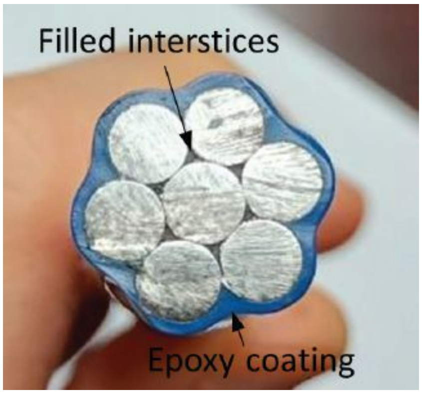

Figure 2. The "cheese plate" assembly maintains the required separation of the ungrouted, noncompacted strands.

Figure 3. Dual-camera borescopes with four-way articulation are used for internal visual inspection through existing stay-cable drain holes.

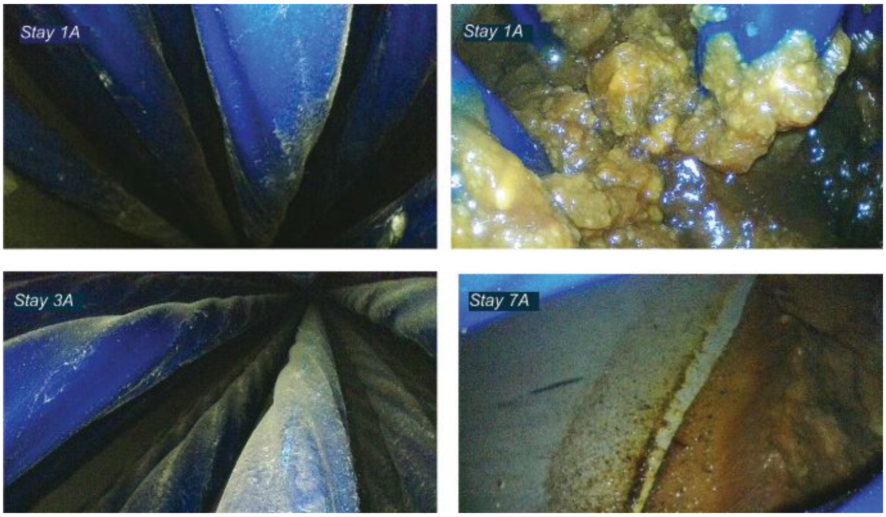

Figure 4. Representative borescope images show the internal condition of the strands. The presence of dust and lack of moisture indicate a dry, healthy internal environment. The upper-right image shows the grease used at the lower ends as a corrosion protection method during the original bridge construction.

confirmed the strands were in excellent condition (**Fig. 4**). The presence of fine dust rather than rust residue or mineral deposits indicated dry internal conditions. Furthermore, the specialized grease at the lower ends appeared stable and showed no signs of water emulsification or contamination.

Environmental Microclimate Monitoring

Corrosion is an electrochemical process that requires moisture and oxygen. By monitoring the microclimate inside the stay cables, engineers can predict the susceptibility of strands to corrosion.

- Sensor deployment: For this investigation, probe-based sensors were installed through drain holes (**Fig. 5**) and coupled to external data loggers attached to the bridge's concrete pylons and shielded from solar radiation to capture ambient conditions.
- Dew point analysis: The study prioritized dew point—an absolute measure of moisture—rather than relative humidity, which varies with temperature. Data (**Fig. 6**) indicated that while internal conditions were slightly more humid than ambient levels, there was no evidence of excessive moisture accumulation or sweating (condensation) on the internal stainless steel surfaces.

Global Health Assessment via Laser Vibrometry

Consistency in stay-cable tension is a primary indicator of both stay-cable health and overall superstructure integrity. However, in ungrouted systems, the sheathing and strands may not vibrate as a unified mass

Figure 5. An internal humidity and temperature probe has been installed at the lower stay-cable termination through an existing drain hole to monitor the internal microclimate.

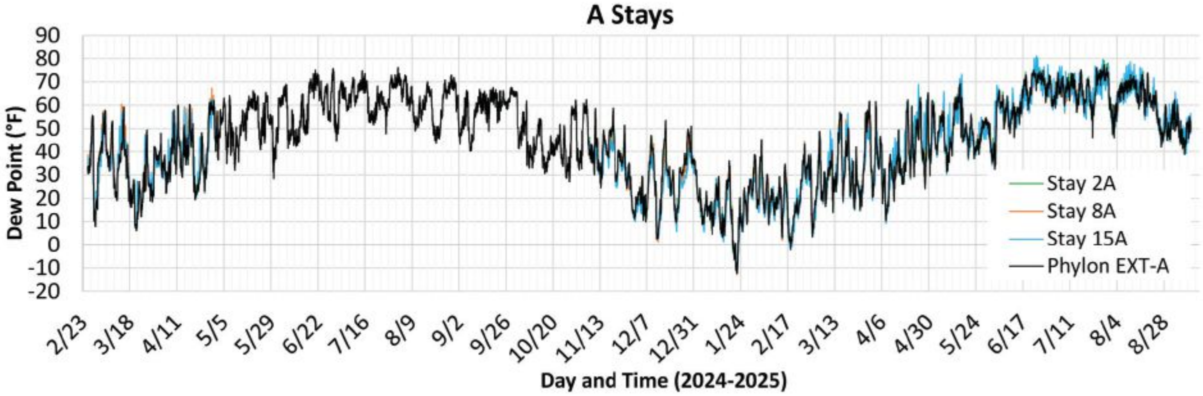

Figure 6. Comparison of internal and external dew point data used as an absolute measure to assess moisture levels and corrosion susceptibility inside the A stay cables, which are located on the north side of the structure.

Figure 7. A laser vibrometer is used to collect data from the Veterans' Glass City Skyway deck during a coordinated lane closure to eliminate traffic-induced vibration noise.

at the lower ends, and that makes traditional accelerometers attached to the sheathing unreliable. Therefore, the team employed laser vibrometry to obtain measurements from the middle third of the stay cables, effectively bypassing the nonunified vibrations present at the lower ends. Laser vibrometry uses a laser to measure the natural frequency (vibration) of a cable; a simple equation is then used to calculate the force in the cable.

- Field testing: To test the laser vibrometry setup, a pilot study was performed from an adjacent bridge that was temporarily closed to traffic. However, to the surprise of the researchers, the measured frequencies corresponded well with the baseline (liftoff) values. During the VGCS evaluation, the Ohio Department of Transportation provided lane closures to eliminate high-pressure shockwaves and deck vibrations associated with heavy truck traffic (**Fig. 7**).
- Results: The temperature-adjusted stay-cable forces showed a deviation of less than 10% from the original liftoff forces recorded during construction. This finding is well within the expected margin for a healthy structure, indicating that no significant section loss or strand breakage had occurred.

Conclusion and Recommendations

Based on the integrated NDE findings, no credible concerns were detected regarding the health of the VGCS stay cables. The multilayered protection system of epoxy coating, internal grease, and stainless steel sheathing appears to be performing as designed. To maintain this status, the following long-term monitoring plan for the stay cables is recommended:

- Routine NDE integration: Formalize borescope use and monitoring of moisture within the sheathing as part of the bridge's regular inspection program.
- Cyclic tension testing: Establish a four-year measurement cycle using laser vibrometry to monitor global force distribution.
- Conditional advanced testing: If localized deterioration is detected, deploy advanced methods such as magnetic flux leakage or electrochemical impedance spectroscopy for the free lengths.

By adopting repeatable NDE protocols, bridge owners can move toward a more predictive health-assessment model, ensuring that structures such as VGCS meet their service-life goals through informed, incremental monitoring.

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Serhan Guner, PhD, PE, is an associate professor in the Department of Civil and Environmental Engineering at the University of Toledo in Toledo, Ohio. His research focuses on bridge engineering, computational and strut-and-tie modeling, and nondestructive testing.


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CBEI SERIES

Immersive, Hands-On Training for Inspection of Bridge Deck Construction

By Dr. Oguzhan Bayrak, Greg Hunsicker, and Doug Beer, Concrete Bridge Engineering Institute

Workforce development has become one of the most pressing challenges facing transportation agencies today. Departments of transportation across the United States are navigating staffing shortages, a growing number of retirements, and limited opportunities for knowledge transfer; at the same time, the demand for safe, durable, and efficiently delivered bridge projects continues to increase. One of the most challenging issues to resolve is how to prepare new construction inspectors and early career engineers to confidently perform bridge deck construction inspection.

Historically, inspectors gained competence through long-term mentorship. Novices often spent months, and sometimes years, shadowing experienced engineers, area managers, or construction project leaders before being assigned primary inspection responsibilities. This approach worked because it provided repeated exposure to real construction conditions and a steady transfer of practical field judgment. However, time and mentorship capacity are now in short supply. With fewer experienced staff available to train and support incoming personnel, agencies must develop new approaches to inspection training.

At the same time, construction inspection is not a profession that can be learned solely in a classroom. Most inspectors and construction engineers learn best through hands-on training, which provides a practical application of connecting plans, specifications, and standards directly to what is happening in the field. For this workforce, traditional lecture-only training can be limited in effectiveness. Field-ready inspectors must interpret construction documents, identify noncompliance, understand intent, communicate corrective actions, and document decisions, often while under pressure. These skills require practical repetition in real conditions.

A Bridge Built for Training

To help address this need, the Concrete Bridge Engineering Institute (CBEI) launched the Bridge Deck Construction Inspection Training Program, which blends classroom learning and field experience in a manner structured to accelerate participants' field readiness for bridge deck inspection.

The program is centered on a purpose-built, three-span bridge training facility constructed to represent multiple phases of deck construction. This training structure provides a unique environment: participants can walk through the deck construction process across three spans, each representing different construction stages, and learn the inspection requirements associated with forming; reinforcement placement; verification before concrete placement; concrete placement, finishing, and curing; and final inspection. The facility allows participants to connect construction details directly back to drawings, notes, and specifications in real time.

Unlike an active construction project, the facility removes many of the pressures that can limit effective learning in the field. There is no contractor production schedule to

Participants at the first Bridge Deck Construction Inspection Training Program at the Concrete Bridge Engineering Institute, held in September 2025, use a full-scale, three-span training bridge to represent phases of deck construction and with built-in problems frequently encountered in the field. All Photos: Concrete Bridge Engineering Institute.

disrupt, no traffic-control risks, and no project-related urgency (such as impatient supervisors) to prevent participants from slowing down and studying details. These conditions create an ideal instructional environment—one that is technically realistic but optimized for learning.

Linking Specifications to Construction Reality

The Bridge Deck Construction Inspection Training Program is designed so participants become familiar with construction documents and inspection expectations in the classroom and then immediately apply that learning in the field. This rapid "paper-to-practice" connection is a significant benefit in the training process. Participants can study reinforcement placement, clear cover, chairing systems, forming alignment, bracing, deck drainage features, blockouts, and other critical details, and discuss inspection priorities while standing over the work.

One of the most powerful outcomes of this approach is the immediate clarity it provides. A question about a plan note or detail can be answered not only with reference to a drawing but also by physically seeing the detail and understanding the constructability implications. This process accelerates learning and builds lasting confidence.

The Final Aspect of Inspection: Communication and Corrective Action

Bridge deck inspection is not just about identifying issues; it is also about how those issues are communicated and resolved. Inspectors must provide timely decisions, apply requirements consistently, and maintain professional fairness. These behaviors influence construction quality and define working relationships on every project.

A principle frequently shared among experienced construction leaders still holds true: contractors want timely, consistent, and fair inspection. When inspection is responsive, uniform, and professionally applied, the entire project benefits, quality improves, rework decreases, and schedule risk is reduced.

Recognizing that communication is one of the most difficult aspects of inspection training, the Bridge Deck Construction Inspection Training Program includes a focused "what's next" component. After a concern is identified, how should it be addressed? What documentation is needed? What is the priority level? When is escalation appropriate? How should corrective action be communicated to facilitate both compliance and production?

To support these discussions, the training bridge includes intentional defects, such as misplaced reinforcing steel and concrete defects in the slab, with varying levels of severity and urgency. These scenarios provide structured opportunities for participants to practice inspection judgment and professional communication in a controlled setting.

Upcoming dates for the Bridge Deck Construction Inspection Training Program in Austin, Tex., are as follows:

May 19–21, 2026
June 9–11, 2026
July 7–9, 2026

The Bridge Deck Construction Inspection Program (DCIP-1001) is open for registration. To register, go to Training and Certification section of the CBEI website (https://cbei.engr.utexas.edu/training-certification). Fees are noted on the registration page.

The Concrete Bridge Engineering Institute's Bridge Deck Construction Inspection Program addresses the critical need for hands-on training experience. Participants are performing dry-run operations prior to concrete placement, including checking the steel cover and concrete slab depth.

The Concrete Bridge Engineering Institute uses training modules that cover every stage of deck construction—from bracing and forming to concrete placement and finishing.

Calibration of Rating Factors for Segmental Bridges

by Dr. Andrzej Nowak, Auburn University, and Dr. Karina Popok, Gresham Smith

Concrete segmental bridges in the United States have evolved from the pioneering structures of the 1970s into a mainstream solution for medium- to long-span crossings (**Fig. 1**). However, many are still load rated using procedures calibrated primarily for girder-type bridges. Using these load-rating methodologies for concrete segmental bridges can produce overly conservative load ratings, especially at service limit states, which typically govern the design of concrete segmental systems. The overarching objective of National Cooperative Highway Research Program (NCHRP) Project 12-123, "Proposed AASHTO Guideline for Load Rating of Segmental Bridges" was to conduct a reliability-based calibration of load and resistance factors specifically for the Service III limit state. The work refined the design and evaluation of concrete segmental bridges by aligning inventory and operating load ratings for concrete segmental bridges with rational target reliability indices.[1] The results of this project have been published as NCHRP Report 1128.[2]

Challenges

The original calibration of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[3] was applied to strength limit states in girder bridges. That calibration is documented in NCHRP Report 368, *Calibration of LRFD Bridge Design Code*.[4] The load components in NCHRP Project 12-33 included live load, dynamic load, and three components of dead load.

Figure 1. A concrete segmental bridge structure is constructed using precast concrete segments. Figure: Datai Crane.

The statistical parameters used in NCHRP Report 368 were based on a very limited truck survey from Ontario, Canada, in 1978. In particular, resistance parameters were very limited and required verification. Furthermore, the parameters were, to a large extent, based on the engineering judgment of experts, and the supporting material had a large degree of variation.

In NCHRP Project 12-123, the research team had access to new data on materials and loads in addition to advanced reliability analysis procedures. However, calibration of load and resistance factors for design and evaluation of concrete segmental bridges involved other challenges, including selection of representative bridges; structural analysis (two-dimensional versus three-dimensional [3-D]); live-load distribution; multilane reduction factors for three or four lanes; striped lanes versus design lanes; loss of prestress (which required a separate study); temperature effects (uniform temperature [TU] and thermal gradient [TG]); condition factor, and system factor. The representative concrete segmental bridges were selected out of a considerable variety of structural types and spans. Therefore, a larger number of bridges had to be considered to capture a representative sample. The structural analysis required 3-D models to consider longitudinal and transverse distribution of loads. Segments are typically one- or two-cell boxes; therefore, the load distribution is very different than that in girder bridges.

Calibration Procedure

The calibration performed in NCHRP Project 12-123 followed a classical reliability framework adapted to concrete segmental bridges, as follows:

1. Selection of representative structures and limit states. The selected

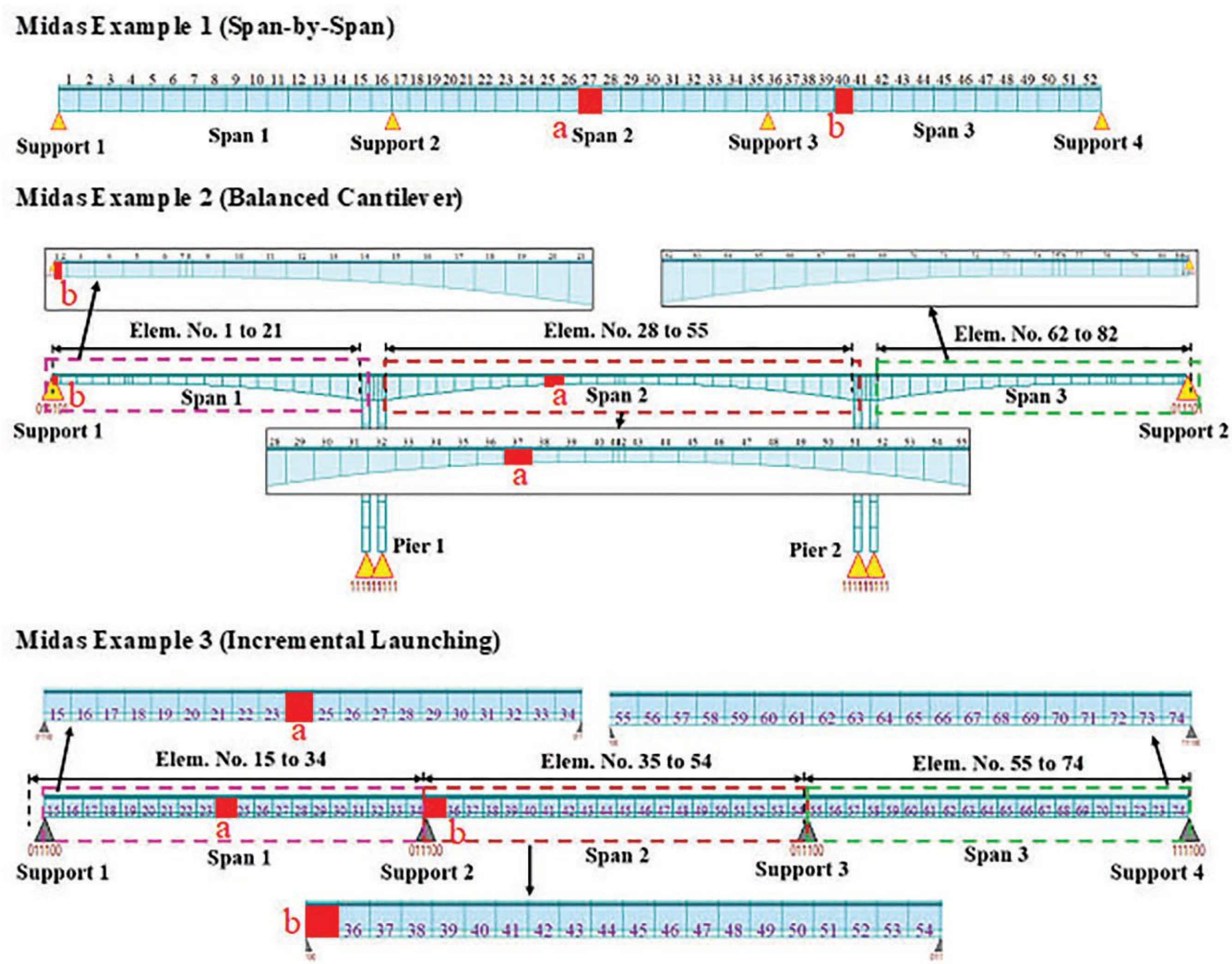
structures included span-by-span, balanced-cantilever, and incremental launching configurations (**Fig. 2**). Four limit states were chosen: service flexure (tensile stress), service principal tension in webs, flexural strength, and shear strength.

2. Development of statistical models for load and resistance. Two key parameters are the bias factor λ, which is the ratio of the mean to the nominal value, and the coefficient of variation *V*. Live-load parameters were derived from the weigh-in-motion data and adapted to segmental bridge configurations, whereas TG models accounted for climatic zones and structural depth effects unique to box-girder systems. For resistance, statistical parameters were obtained from material properties, fabrication variability, and analytical model factors.
3. Reliability analysis. The reliability indices were calculated for representative structures and limit states. The wide spectrum of obtained results indicated reliability indices for already-constructed concrete segmental bridges.
4. Selection of the target reliability index for the considered limit states. It was agreed that the existing concrete segmental bridges have acceptable reliability. The mean values were taken as a base, then adjusted based on specific target reliability index criteria. Therefore, the target reliability index can be considered close to the center of the obtained spectrum.
5. Selection of the limit state–specific load and resistance factors. Candidates for load and resistance factors were limited because the values were rounded to the nearest 0.05. For each set of load and resistance candidates, representative structures were redesigned accordingly and the resulting reliability values were compared with the target.

Findings and Recommendations

Concrete segmental bridge–specific reliability calibration is critical at the Service III limit state. Applying girder-based calibration to segmental bridges misrepresents their reliability performance. The proposed calibration methodology, including updated load and resistance factors, achieves

Figure 2. Locations where reliability index β is critical for load ratings for the service flexure limit state: (a) in the bottom fiber; (b) in the top fiber.[2]



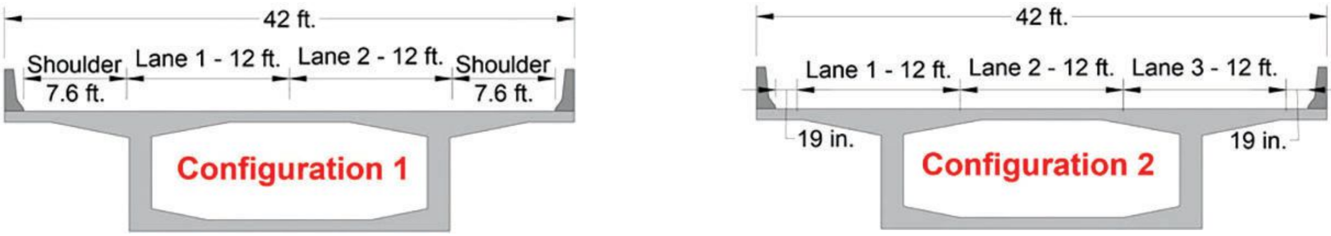


Figure 3. Lane configurations for a box-girder cross section of a concrete segmental bridge: (a) two-lane striped configuration; (b) three-lane design configuration.[2]

uniform reliability while accounting for the quality, continuity, and redundancy of concrete segmental construction. Specific recommendations are as follows:

- Retain the use of design lanes for inventory load ratings, and use design lanes instead of striped lanes, (**Fig. 3**) with reduced live-load factors for Service III limit state operating load ratings in AASHTO's *Manual for Bridge Evaluation*.[5] This approach rationally reduces live-load demand and aligns with reliability targets for service limit states (β = 1.0 for inventory load rating, β = 0 for operating load rating).
- Retain LRFD strength-level live-load factors for concrete segmental boxes. Strength limit states remain governed by γ_{LL} = 1.75 for inventory load ratings and γ_{LL} = 1.35 for operating load ratings, ensuring consistency with LRFD calibration lineage. Maintain γ_{LL} = 0.8 for Service III inventory load ratings with resistance factor of ϕ = 1.0. Apply reduced live-load factor of γ_{LL} = 0.65 for Service III operating ratings since reliability analysis confirms alignment with β targets (**Fig. 4**), with a resistance factor of ϕ = 1.0.
- Include TG in Service III limit state checks. The calibrated load combinations 0.5 TG with live load and 1.0 TG without live load are essential for accurately capturing joint and web stresses that often control the load ratings of concrete segmental bridges.
- Adopt updated resistance statistics and fabrication factors. The research introduces refined models for material properties, fabrication variability, and professional factors, including updated fabrication-factor distributions specific to precast, prestressed concrete construction, which is typical for concrete segmental bridges. These models improve reliability predictions for flexural capacity and shear strength, particularly in systems with a mix of bonded and unbonded tendons.

A reliability-based calibration was performed for load and resistance factors specifically for the Service III limit state. It refined the design and evaluation of concrete segmental bridges by aligning inventory and operating load ratings for concrete segmental bridges with rational target reliability indices (Fig. 4).

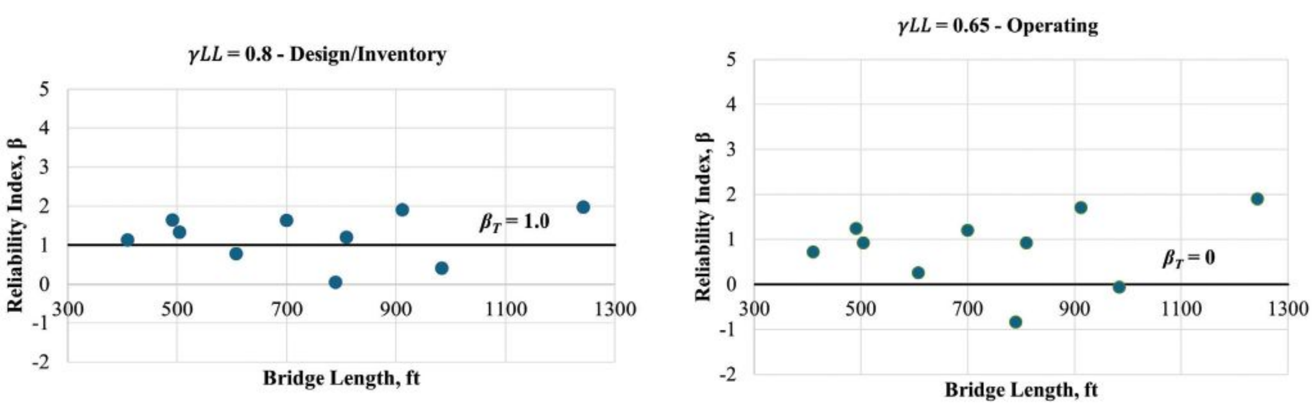
Acknowledgments

The authors acknowledge the contributions of the research team, including Hani Nassif, Chris Eamon, Fatmir Menkulasi, John Corven, Ben Morris, and Robert Barnes.

References

1. Popok, K. 2024. "Reliability-Based Calibration for the Design and Load Rating of Segmental Bridge." PhD diss., Auburn University.
2. Popok, K., A. Nowak, H. Nassif, et al. 2024. *Load Rating of Segmental Bridges*. National Cooperative Highway Research Project (NCHRP) Report 1128. Washington, DC: National Academies Press. https://doi.org/10.17226/28597.
3. American Association of State Highway and Transportation Officials (AASHTO). 2020. *AASHTO LRFD Bridge Design Specifications*. 9th ed. Washington, DC: AASHTO.
4. Nowak, A. S. 1999. *Calibration of LRFD Bridge Design Code*. NCHRP Report 368. Washington, DC: NCHRP. https://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_368.pdf.
5. AASHTO. 2020. *Manual for Bridge Evaluation*. 3rd ed. Washington, DC: AASHTO.

Figure 4. Computed minimum reliability indices β in the representative bridges (service flexure limit state): (a) using γ_{LL} = 0.8 (inventory load rating); (b) using γ_{LL} = 0.65 (operating load rating).[2]



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

IN THIS ISSUE

https://www.pci.org/workforce
Workforce development is a hot topic in the construction industry. This is a link to PCI's workforce development resources webpage, which offers many resources related to worker recruitment, retention, and wellness. The Focus article on page 5 answers questions about how to drive change in workforce development.

https://www.pa.gov/agencies/penndot/projects-near-you/district-1-projects/route-6-bridge-replacement-project-near-saegertown
The French Creek Parkway Bridge no. 3 project near Hayfield and Woodcock Townships, Pa., is noted in the Perspective article on page 8. This is a link to the project website, which provides a project overview and a project presentation.

https://www.buildingsmart.org
Open data standards developed by buildingSMART International, which are referenced in the Perspective article on page 11, can be found at this link. The website includes the buildingSMART Data Dictionary, Information Delivery Specification, and BIM Collaboration Format.

https://cvent.utexas.edu/event/DCIP-1001/details
This is the link to the registration page for the Concrete Bridge Engineering Institute's (CBEI's) Bridge Deck Construction Inspection Program, which is described in the CBEI article on page 50. The program is open to the public.

https://www.aspirebridge.com/magazine/2015Summer/ASPIRESupplementSummer2015.pdf
The Polk Creek bridges discussed in the Concrete Bridge Technology article on page 23 use an innovative solution that incorporates curved precast, post-tensioned concrete U-girders. A supplement to the Summer 2015 issue of *ASPIRE*®, found at this link, tracks the development of the U-girder.

https://www.aspirebridge.com/magazine/2007Summer/veterans_glass_sum07.pdf
Monitoring of the Interstate 280 Veterans' Glass City Skyway (VGCS) is presented in the Safety and Serviceability article on page 46. Shortly after construction of the VGCS was completed, the bridge was featured in the Summer 2007 issue of *ASPIRE*. This link takes the reader to the Project profile of VGCS in that issue.

https://www.fhwa.dot.gov/pavement/concrete/asr/pubs/hif12022.pdf
This is a link to the *Alkali-Silica Reactivity Field Identification Handbook*, published by the Federal Highway Administration. The handbook offers an explanation of the chemistry behind alkali-silica reactivity (ASR) and many photos and guidelines to help identify ASR-related deterioration in the field. The Concrete Bridge Technology article on page 30, which is the latest article in a series about cracking, covers the structural evaluation of ASR-affected concrete bridges.

https://www.aspirebridge.com/magazine/2021Spring/CBT-LoadRatingSegmentalBridges.pdf
The Professor's Perspective on page 52 discusses the calibration of load ratings for segmental bridges. The Concrete Bridge Technology article in the Spring 2021 issue of *ASPIRE*, found at this link, gives useful background on the challenges associated with load rating segmental bridges.

https://www.codot.gov/projects/i70westvailauxiliarylanes
The Interstate 70 West Vail Pass auxiliary lanes project website can be found at this link. As part of the project to enhance safety and reduce congestion through the steep mountain pass, the Colorado Department of Transportation is replacing the bridge structures over Polk Creek in Summit County, Colo., near the town of Vail. The bridge replacements are the focus of the Concrete Bridge Technology article on page 23.

https://www.nationalacademies.org/publications/28597
This research report from the National Cooperative Highway Research Project (NCHRP) on load rating of segmental bridges, was the focus of the Professor's Perspective on page 52.

AASHTO LRFD Bridge Design Specifications: *Strain Compatibility Analysis and Reinforcement Properties*

by Dr. Oguzhan Bayrak, University of Texas at Austin

This article provides an overview of three agenda items that update the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*:[1] agenda item 34 (working agenda item [WAI] 235): Strain Compatibility; agenda item 37 (WAI 238): Reinforcement Properties; and agenda item 38 (WAI 234): ASTM A615 Updates. While these agenda items were developed separately, it is logical to discuss them together because strain compatibility analyses involve the use of mechanical properties of reinforcement.

In estimating the flexural capacity of reinforced concrete elements, it is customary to assume that the rupture strain of typical ASTM A615[2] or ASTM A706[3] reinforcing bars is sufficiently large. That is to say, as crushing of concrete on the flexural compression side takes place, the reinforcement on the flexural tension side of a member yields but does not reach its rupture strain. This assumption can easily be verified by establishing the strain profile at strength limit strength. However, it can be problematic to extend the logic of the assumption to prestressed concrete components and members in which reinforcing materials with lower rupture strains, such as stainless steel or carbon-fiber reinforced polymers, are used. In such cases, rupture of reinforcement may take place before concrete on the flexural compression side of an element is crushed. In addition, the closed form equations of Article 5.6.3 of the AASHTO LRFD specifications can significantly underpredict the flexural resistance of flanged precast, prestressed concrete girders in cases when the neutral axis falls outside of the flange. (See articles on strain compatibility and girders with composite decks in the Fall 2024 and Winter 2025 issues of *ASPIRE*®.) In all of the cases described in this paragraph, strain compatibility analyses provide estimates of flexural capacity that are more accurate than the customary assumption. In this context, agenda item 34 offers guidance for performing strain compatibility analyses, and agenda item 37 provides detailed information on the mechanical properties of reinforcing steels. Since agenda item 34 relies on information provided in agenda item 37, let us start our discussion with agenda item 37.

Agenda Item 37

This agenda item adds a new table (**Table 5.4.3.1-1**) that provides a summary of steel reinforcement.

In addition to the table, this agenda item adds the following language after the second paragraph of C5.4.3.1, across from the new Table 5.4.3.1-1, as follows:

> *The values listed in Table 5.4.3.1-1 represent the minimum values that materials must achieve to meet the requirements of the ASTM specification for a given grade. Designers should use these minimum values in design equations unless material specific information is available.*
>
> *There is no value provided for elongation/tensile strain in ASTM A1064; therefore, it was not included in the table. Designers should review available research data should they need to find a minimum tensile strain for welded wire reinforcement that complies with ASTM A1064.*
>
> *Other materials that are not included in Table 5.4.3.1-1 may be used as reinforcement. For data on those materials, designers should consult applicable ASTM standards, available research, or material suppliers.*

Agenda item 37 also revises **Table 5.4.4.1-1** by adding the specified minimum tensile strains and including 0.7-in.-diameter strands.

Finally, this agenda item adds a new commentary article, C5.4.4.1, as follows:

> *The values for tensile strength and minimum tensile strain listed in Table 5.4.4.1-1 represent the minimum values that materials must achieve to meet the requirements of the ASTM specification for a given grade or type. Designers should use these minimum values in design equations unless material specific information is available.*

Agenda Item 34

This agenda item adds a new (15th) bullet point to Article 5.6.2.1, as follows:

- *The minimum strain limits of reinforcing steel specified in Table 5.4.3.1-1 and prestressing steel specified in Table 5.4.4.1-1 shall be considered.*

A new paragraph within the commentary (C5.6.2.1) and directly across from the new bullet point in Article 5.6.2.1 is added, as follows:

> *The values listed in Table 5.4.3.1-1 and Table 5.4.4.1-1 represent the minimum values that materials must achieve to meet the requirements of the ASTM specification for a given grade. Designs should not rely on values that exceed these minimums unless material specific information is available.*

In addition, this agenda item revises Article C5.6.3.2.2 by adding a new second paragraph to read as follows:

> *Seguirant, Brice, and Khaleghi (2004)[8] demonstrate that the*

Table 5.4.3.1-1. Material Standards for Reinforcing Steel

Standard	Grade of Reinforcement	Specified Minimum Yield Strength (ksi)	Specified Minimum Tensile Strength (ksi)	Specified Minimum Tensile Strain
ASTM A615	Grade 60	60	80	0.07
	Grade 80	80	100	0.06
	Grade 100	100	115	0.06
ASTM A706	Grade 60	60	80	0.10
	Grade 80	80	100	0.10
	Grade 100	100	117	0.10
ASTM A955[4]	Grade 60	60	90	0.20
	Grade 75	75	100	0.20
	Grade 80	80	100	0.16
ASTM A1035[5]	Grade 100	100	150	0.06
ASTM A1064[6]	Grade 56	56	70	Not included in ASTM
	Grade 65	65	75	
	Grade 70	70	80	
	Grade 72.5	72.5	82.5	
	Grade 75	75	85	
	Grade 77.5	77.5	87.5	
	Grade 80	80	90	

Note: Minimum tensile strain for each grade is shown. Value is dependent on bar size within grade. See ASTM standard for more details. For components designed using the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*,[7] the reduced ultimate tensile strain values shall be used in lieu of the values in the table above.

Table 5.4.4.1-1. Limits on Prestressed Steel

Material	Grade or Type	Diameter (in.)	Tensile Strength, f_{pu} (ksi)	Yield Strength, f_{py} (ksi)	Specified Minimum Tensile Strain
Strand	270 ksi	0.375 to 0.7	270	90% of f_{pu} = 243	0.035
Bar	Type 1, Plain	0.75 to 1.375	150	85% of f_{pu} = 127.5	0.04
	Type 2, Deformed	0.625 to 2.5	150	80% of f_{pu} = 120	

Note: For components designed using the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, the reduced ultimate tensile strain values in the Guide Specifications shall be used in lieu of the values in the table above.

flexural resistance predicted by Eq. 5.6.3.2.2-1 becomes increasingly conservative relative to strain compatibility analysis when the neutral axis is below the reinforced concrete deck. The degree of conservatism magnifies with increasing girder concrete strength. Neglecting the compression force contribution of beam flanges and large chamfers at box section web-flange intersections also contributes to the conservatism. The higher strength concrete in the prestressed concrete girder is not considered due to limitations of the approximation flexural resistance equations. As the depth of the compression block increases to balance the tension force in the reinforcement, the internal moment arm decreases leading to a lower predicted capacity. This in turn leads to sections that are predicted by Eq. 5.5.4.2-1 and Eq. 5.5.4.2-2 to be in the compression-controlled or transition region with reduced resistance factors. In these cases, strain compatibility analysis can provide a more accurate estimate of flexural resistance.

Article 5.6.3.2.5 is also revised as follows:

The strain compatibility approach is recommended when more precise calculations are required especially for flanged prestressed concrete sections. The appropriate provisions of Article 5.6.2.1 shall apply.

The stress and corresponding strain in any given layer of the *section is taken from any appropriate stress-strain formula or graph of the material in the layer. Minimum specified properties shall be used in the stress-strain formula or graph for non-prestressed reinforcement with yield strengths between 75.0 and 100 ksi.*

A new commentary article, C5.6.3.2.5, is added, as follows:

Strain compatibility analysis is a method of analysis that often involves subdividing the cross section into discrete layers or fiber elements, estimating the strain in each element based on the assumption of plane sections, evaluating the stress using stress-strain relationships, and integrating the stress field to determine the flexural strength of the section. This analysis is typically iterative by varying the curvature of the section until conditions of equilibrium are satisfied.

Strain compatibility analysis uses stress-strain relationships for the reinforcement and the concrete elements. Composite girder sections with different concrete compressive strengths in the deck and girder are more accurately modeled with this approach. Non-rectangular sections and multiple layered reinforcing with limited ductility can also be accurately analyzed. The analysis can account for reinforcement at any position in the cross section.

Stress-strain relationships for concrete compression are typically non-linear or bilinear curves. Several stress-strain concrete curves have been developed and may be used to determine the stress-strain relationship to model the behavior of the concrete in compression. See Hognestad (1951)[9] and Desayi and Krishnan (1964).[10] Seguirant, Brice, and Khaleghi (2004)[8] use a non-linear stress-strain curve based on Collins and Mitchell (1991).[11] Figure C5.6.3.2.5-1 shows a bilinear concrete compression stress-strain relationship based on French specifications (2016)[12] and used by El-Helou (2022)[13] and Tadros (2021).[14] The bilinear stress-strain relationship is expressed in Eqn. C5.6.3.2.5-1 and produces results similar to nonlinear models.

$$f_c(\varepsilon_c) = \begin{Bmatrix} E_c\varepsilon_c \text{ for } \varepsilon_c < \dfrac{0.85f'_c}{E_c} \\ 0.85f'_c \text{ for } \varepsilon_c \geq \dfrac{0.85f'_c}{E_c} \end{Bmatrix} \qquad \text{(C5.6.3.2.5-1)}$$

where:

ε_c = *compressive strain in concrete (in./in.)*

f_c = *compressive stress corresponding to the strain* ε_c *(ksi)*

E_c = *modulus of elasticity of concrete (ksi)*

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

The Menegotto-Pinto model (Menegotto, 1973),[15] also known as the "power formula" (Mattock 1979,[16] Skogman 1988,[17] Devalapura 1992[18]), is often used to idealize the tensile stress-strain behavior of steel reinforcement. The power formula is expressed by Equation C5.6.3.2.5-2. Figure C5.6.3.2.5-2 shows the stress-strain curve for different reinforcement types.

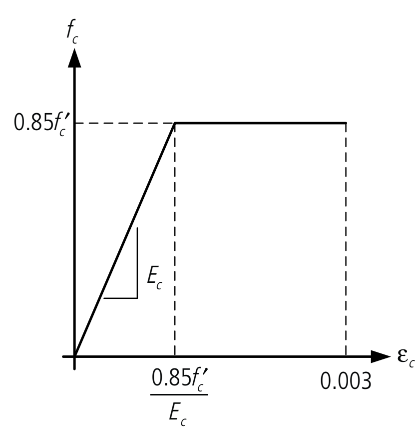


Figure C5.6.3.2.5-1. Bilinear concrete compression stress-strain relationship.

$$f(\varepsilon_s) = E\varepsilon_s \left\{ Q + \frac{1-Q}{\left[1+\left(\dfrac{E\varepsilon_s}{Kf_y}\right)^R\right]^{\frac{1}{R}}} \right\} \leq f_u \qquad \text{(C5.6.3.2.5-2)}$$

where:

ε_s = *strain in reinforcement (in./in.)*

E = *reinforcement modulus of elasticity;* E_s *for steel reinforcing,* E_{ps} *for prestressing steel (ksi)*

f_y = *reinforcement yield stress;* f_y *for steel reinforcing,* f_{py} *for prestressing steel (ksi)*

f_u = *minimum tensile strength of reinforcement;* f_{pu} *for prestressing steel (ksi)*

K, Q, *and* R = *curve fitting parameters*

Power formula curve fitting parameters for prestressing strand and reinforcing bars are given in Table C5.6.3.2.5-1. Common properties of reinforcement are listed in Table 5.4.3.1-1 and Table 5.4.4.1-1. Steel reinforcement stress-strain curves using the curve fitting parameters are shown in Figure C5.6.3.2.5-2.

Table C5.6.3.2.5-1. Power Formula Curve Fitting Parameters

Reinforcement Type	K	Q	R
ASTM A416[19] Grade 270 Strand	1.040	0.031	7.36
ASTM A615, A706 Grade 60	1.096	0.000	100.0
ASTM A615, A706 Grade 80	0.980	0.028	100.0
ASTM A615 Grade 100	0.980	0.022	4.50
ASTM A1035 Grade 100	1.140	0.050	1.71

Stress-strain relationships for steel reinforcement by Thompson and Park (1980)[20] and prestressing steel according to PCI Design Aid 15.2.3 (PCI, 2017)[21] may be used.

Agenda Item 38

This agenda item corrects the ratios of minimum yield strength to ultimate tensile strength for AASHTO M 31 (ASTM A615) Grade 60 and Grade 80 reinforcing bars. The correction will increase the cracking moment M_{cr}, which would increase the minimum amount of flexural reinforcement required in cases where M_{cr} is less than 1.33 times the ultimate moment M_u.

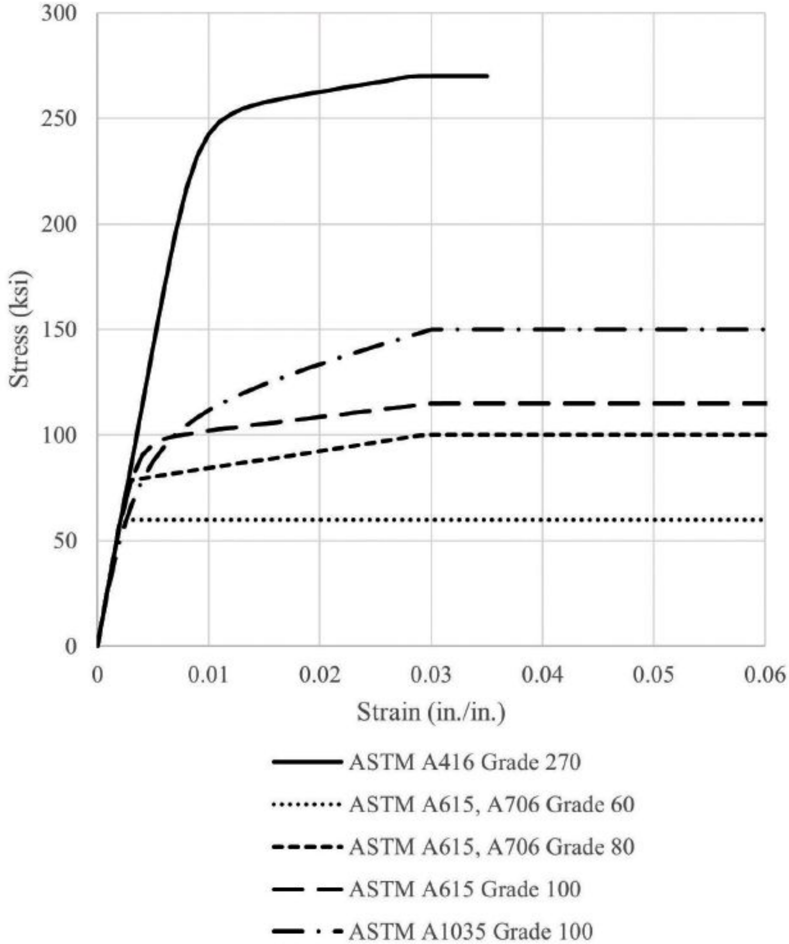


Figure C5.6.3.2.5-2. Stress-strain curves for several types of steel reinforcement.

Additionally, agenda item 38 adds the ratios for AASHTO M 31 (ASTM A615) Grade 100 reinforcing bar, ASTM A706 Grade 100 reinforcing bar, and ASTM A955 Grades 60, 75, and 80 reinforcing bars. It also provides values for minimum tensile strength of reinforcing bars to use when determining spacing of noncontact lap splices of longitudinal reinforcement that go from columns and anchors in oversized shafts.

This agenda item also refers users to the new Table 5.4.3.1-1 that is being developed under WAI 238.

Furthermore, this agenda item revises the notation $f_{u\ell}$ in Article 5.3 as follows:

$f_{u\ell}$ = *specified minimum tensile strength of column longitudinal reinforcement (ksi), (5.10.8.4.2a)*

The third paragraph of Article 5.6.3.3 is revised as follows:

γ_3 = *ratio of specified minimum yield strength to minimum tensile strength of the non-prestressed reinforcement*

= *0.75 for AASHTO M 31 (ASTM A615), Grade 60 reinforcement*

= *0.80 for AASHTO M 31 (ASTM A615), Grade 80 reinforcement*

= *0.87 for AASHTO M 31 (ASTM A615), Grade 100 reinforcement*

= *0.75 for ASTM A706, Grade 60 reinforcement*

= *0.80 for ASTM A706, Grade 80 reinforcement*

= *0.85 for ASTM A706, Grade 100 reinforcement*

= *0.67 for ASTM A955, Grade 60 reinforcement*

= *0.75 for ASTM A955, Grade 75 reinforcement*

= *0.80 for ASTM A955, Grade 80 reinforcement*

= *0.67 for AASHTO M 334 (ASTM A1035), Grade 100 reinforcement*

Finally, this agenda item revises the definition of $f_{u\ell}$ in Article 5.10.8.4.2a as follows:

$f_{u\ell}$ = *specified minimum tensile strength of column longitudinal reinforcement (ksi) from Table 5.4.3.1-1*

Fundamentally, the three agenda items covered in this article provide the much-needed clarification on calculating flexural capacity of prestressed concrete elements in bridges, by taking into account all potential failure modes such as strand rupture and crushing of concrete on the flexural compression side of an element. In addition, the inclusion of minimum specified tensile strain values in Chapter 5 of the AASHTO LRFD specifications, will prove to be useful in design.

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5. ASTM International. 2024. *Standard Specification for Deformed and Plain, Low-Carbon, Chromium, Steel Bars for Concrete Reinforcement*. ASTM A1035/A1035M-24. West Conshohocken, PA: ASTM International.
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2026 TRANS-IPIC UTC Workshop

April 14: Big Ten Conference Center, Rosemont, IL

The Transportation Infrastructure Precast Innovation Center (TRANS-IPIC) is happy to announce its 2026 UTC Workshop. TRANS-IPIC is a research consortium, funded through the USDOT's University Transportation Centers (UTC) Program, and led by the University of Illinois Urbana-Champaign. Our research focuses on Precast Concrete (PC) and related technologies. Register for the 2026 TRANS-IPIC UTC Workshop to hear how PC and its related technologies can be utilized to boost durability and safety of various transportation modes in the U.S. through infrastructure repair and reconstruction.

8:00-8:25 a.m. Breakfast/Networking

8:25-8:30 Welcome and Summary Remarks
Bassem Andrawes, TRANS-IPIC Director

8:30-9:00 TDOT's Use of Precast Bridge Elements & Systems in ABC Projects
Presenter: Ted A. Kniazewycz, Tennessee Department of Transportation

9:00-9:30 Innovative Design and Monitoring Technologies for Transportation Infrastructure
PI and Presenter: Bassem Andrawes, University of Illinois Urbana-Champaign

9:30-10:00 Practical Lessons Learned from Application of Precast Elements Over the Years
Presenter: Tony Shkurti, HNTB

10:00-10:15 Morning Break

10:15-10:45 Integrating 3D Printing and Smart Sensing into Permanent Formwork for Next-Gen Precast Concrete Structures
PI: Yen-Fang Su, Louisiana State University
Presenter: Khalilullah Taj

10:45-11:15 Hollow Precast Inverted Tees in Bridge Substructures
Presenters: Sam Keske and John Kintz, WJE

11:15-11:45 3D Printed Advanced Materials to Mitigate Prestressed Concrete Girder End Cracks
PI: Ravi Ranade, University at Buffalo
Presenter: Pranay Singh

11:45-12:30 p.m. Poster Session & Live Demo

12:30-1:15 Lunch

1:15-1:45 Precast Concrete in Texas Bridges: Design Innovations and Durability Strategies
Presenter: Robert D. Owens, Texas Department of Transportation

1:45-2:15 Advanced Curing Methods and Nanomodification for Accelerated Production of Durable Precast Elements: Opportunities and Challenges
PI and Presenter: Mirian Velay-Lizancos, Purdue University

2:15-2:45 Precast Segmental in Urban Transit – CTA Red-Purple Modernization Case Study
Presenter: Ben Soule, Systra International Bridge Technologies

2:45-3:00 Afternoon Break

3:00-3:30 Can Road Users Play a Role in Structural Health Inspection? Continuous Inspection of Precast Concrete Bridges using Connected Automated Vehicles
PI and Presenter: Chaozhe He, University at Buffalo

3:30-4:00 Manufacturing Techniques: Continual Improvement in Design and Construction
Presenter: Donald S. Dardis, Dukane Precast

4:00-4:30 Gaze-directed UAV-UGV Coordination Framework for Onsite Quality Inspection of Precast Bridge Construction
PI and Presenter: Jiannan Cai, University of Texas at San Antonio

[Click Here to Register](#)

PCI Offers Transportation eLearning Modules

Courses on Design and Fabrication of Precast, Prestressed Concrete Bridge Beams

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

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

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

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

PCI eLearning Precast, Prestressed Concrete Bridge Girder Series

Preliminary Precast, Prestressed Concrete Design (T110)

Materials and Manufacturing of Precast, Prestressed Concrete (T115)

Design Loads and Load Distribution (T120)

Flexure Service (T125)

Flexure Strength (T130)

Prestressed Losses (T135)

Shear (T145)

End Zone Design (T160)

Extending Spans (T310)

Bearing Pads (T450) and (T455)

In-Service Analysis Load Rating (T710)

Full-Depth Precast Concrete Deck Panels Series

Introduction on Full-Depth Panel Precast Concrete Deck System and Its Advantages (T210)

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

Production and Construction Details of Full-Depth Precast Concrete Deck Panels (T220)

Case Studies and Emerging Developments of Full-Depth Precast Concrete Deck Panels (T225)

Lateral Stability of Precast, Prestressed Concrete Bridge Girders Series

Introductory Material and Hanging Girders (T520)

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Seated Girders and Stability Issues from Bed to Bridge (T525)

Stability Calculations and Sensitivity Analysis (T527)

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