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SUMMER 2025

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The Elusive Art and Science of Leadership

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

Leadership always seems to be a topic of conversation. Everyone is looking for the "secret sauce" and contemplating the same questions: Why is good leadership so elusive? Is it art, science, or a bit of both? Why is it so challenging to get your head around it and put it into practice? I've talked with industry professionals and military officers about leadership, always attempting to find a few key points to develop my skills. What I've come to realize is that leadership is as unique as is the leader. Fortunately, we can learn from one another and improve.

Recently, I read an excerpt from *Extreme Ownership: How U.S. Navy SEALs Lead and Win* by Jocko Willink and Leif Babin, two former Navy SEALs.[1] Tyler DeVries provides a summary[2] of the book's key themes: take responsibility, lead with humility, make simple plans and communicate them, get comfortable making decisions without all the information, and lead and support your superiors. There are a couple of new ideas in that list that piqued my interest, and I have added *Extreme Ownership* to my reading list; you might find it interesting, too.

In the past 10 years or so, the topic of "work-life balance" is everywhere. Part of being a leader is finding one's own work-life balance and helping others find it, too. There is merit in finding the right mix for both personal and professional growth and development. A friend of mine blocks out "white space" on his calendar. This is time that cannot be filled without his approval. Sometimes, he uses the time for professional growth. Other times, he turns off his brain and reflects on things unrelated to specific projects or work in general. He also blocks out white space for his staff so they can engage in topics unrelated to their normal work. He told me that "forced time, paid time" away from normal office or project work provides huge benefits. It builds the connectivity of his team by supplying the most valuable resource—time.

Another key leadership concept is collaboration. The reason I feel so strongly about collaborative solutions is that I have witnessed how they benefit the parties involved, leading to long-term success and opportunities for future endeavors. One example is the collaborative effort with National Concrete Bridge Council members, the Federal Highway Administration, and the pooled-funding states in establishing the Concrete Bridge Engineering Institute at the University of Texas, Austin. That institute shows how collaboration benefits the training of our workforce, enhances students' academic journeys, and advances our concrete bridge industry.

I am a believer in sharing your discoveries, and I encourage you to share yours with the readers of *ASPIRE*®. A colleague uses the term "discovery learning" for the lessons learned when things don't go as planned. Another friend measures the complexity of a do-it-yourself home project by the number of trips to the store to deal with unanticipated "discoveries." I recognize that sharing may seem to run counter to the idea of ownership as it relates to, say, patenting a new product. That said, sharing an innovative technique, method, or solution to increase efficiency or improve safety or quality helps all of us avoid "discovery learning" situations.

In the Fall 2024 issue of *ASPIRE*, we bid farewell to several esteemed colleagues. How do we fill those holes in the ranks? We can't replace these experts, but how do we begin the journey of finding, recruiting, and developing the next group of industry leaders and professionals? The answer is simple: We stay interested, engaged, and relevant. Bridge people are a different breed. If construction, inspection, and engineering were easy, everyone would do it. We are set apart because we tackle the challenging issues facing our industry. Deep down, we are a very proud and competitive group. We find satisfaction in answering the demanding questions, solving the equations, and ultimately delivering resilient, quality concrete bridge solutions that benefit society. We're in a great profession at a great time—talk it up!

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Cover
Formwork and temporary shoring facilitate construction of the Bend Bridge's complex geometry in Toledo, Ohio. Photo: Kokosing Construction.

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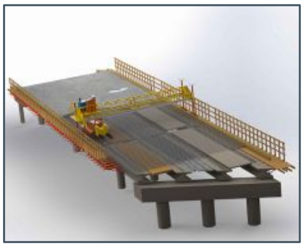

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Brian Toombs, PE, has 27 years of professional experience as an interchange specialist at Burgess & Niple Inc. in Columbus, Ohio. He has served on Burgess & Niple's culture committee for the past 4 years.

CONCRETE CALENDAR 2025–2026

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

July 13–16, 2025
International Bridge Conference
David L. Lawrence Convention Center
Pittsburgh, Pa.

August 19–20, 2025
Concrete Materials for Bridges
Concrete Bridge Engineering Institute
Austin, Tex.

August 19–20, 2025
NCBC Prestressed Concrete Bridge Seminar: Concepts for Extending Spans
Embassy Suites Downtown Convention Center
Denver, Colo.

September 8–12, 2025
PTI Certification Week
Marriott Phoenix Chandler
Phoenix, Ariz.

September 14–17, 2025
AREMA Annual Conference & Expo
Indiana Convention Center
Indianapolis, Ind.

September 16–20, 2025
PCI Committee Days Conference
Loews Chicago O'Hare
Chicago, Ill.

September 30–October 3, 2025
PTI Committee Days
Kempinski Hotel Cancun
Cancun, Mexico

October 2–5, 2025
Carbon Conscious Concrete (C3) Symposium
Westin Chicago River North
Chicago, Ill.

October 20–22, 2025
Midwest Bridge Preservation Partnership Meeting
Hyatt Regency Columbus
Columbus, Ohio

October 20–25, 2025
PTI Certification Week
Atlanta Marriott Northeast/Emory
Atlanta, Ga.

October 26–29, 2025
ASBI Annual Convention and Committee Meetings
Hyatt Regency
Bellevue, Wash.

October 26–29, 2025
ACI Concrete Convention
Hilton Baltimore and Marriott Baltimore Inner Harbor
Baltimore, Md.

November 2–5, 2025
CRSI Fall Business and Technical Meeting
Drake Hotel
Chicago, Ill.

November 17–22, 2025
PTI Certification Week
Hilton Austin Airport
Austin, Tex.

January 11–15, 2026
Transportation Research Board Annual Meeting
Walter E. Washington Convention Center
Washington, D.C.

NCBC
National Concrete Bridge Council

2025 NCBC Webinar Series

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

Schedule

July 23: Extending the Service Life of Concrete Using Lightweight Aggregates

August 20: Introducing Life-365 Service Life Model: A Web App for Estimating the Service Life of Concrete Mixture Designs and Corrosion Protection Systems

Other dates include:

September 10 October 22 November 19

Check our website for updates.

Certificates of attendance are available for these free 1-hour webinars.

Importance of Culture for Talent Retention

by Brian Toombs, Burgess & Niple Inc.

When I talk with colleagues about their main challenges in the construction industry, their responses vary, touching on financial goals, specific projects, client relations, management dynamics, or workload. However, today the primary concern seems to be hiring and retaining talented professionals. In the current competitive landscape with large volumes of infrastructure work available, many engineering firms are eager to secure talent. As part of their recruitment strategies, companies may promise higher pay, better benefits, and more opportunities, and such offers make it difficult for you as the current employer to retain employees. Therefore, creating a workplace culture that is positive, empowering, and supportive of professional growth is crucial to keeping your talent when recruiters come calling.

How Did We Get Here?

Most engineering and construction industry leaders recognize that we have a fundamental problem: there are not enough engineers to complete the current level of infrastructure work. The U.S. Bureau of Labor Statistics estimates a 6% increase in engineering positions over the next decade,[1] while enrollment in some university engineering programs is in decline. When the 2008 recession occurred, many engineers exited the industry, and some college students switched from engineering to majors with more stable career paths. As a result, we currently have a shortage of midlevel engineering talent. The COVID-19 pandemic had a similar downward effect in 2020 on the influx of new engineers into the workforce, which means that there is a deficit of engineers with 5 years of experience in the industry today. All these factors are driving companies to hire aggressively and making it increasingly difficult to retain talent.

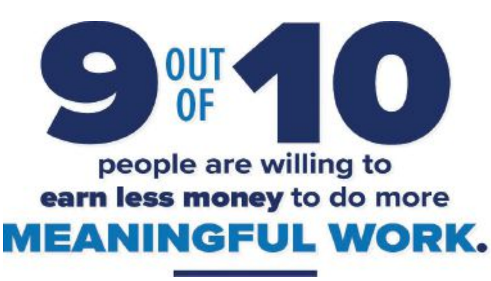
As we engage in this war for talent, it is critical to understand what is important to your employees. When I talk to my staff, I frequently hear that they want a better work-life balance; they want to be respected and challenged; and they hope to have a significant influence in the company, work on innovative projects, and make an impact in the community. Everyone has a desire to be part of an organization that wants them to succeed. The ability to provide that type of environment depends on your organizational culture.

The challenges of the architecture, engineering, and construction (AEC) industries. All Photos and Figures: Burgess & Niple Inc.

Culture as a Competitive Advantage

The Merriam-Webster dictionary defines organizational culture as "the set of shared attitudes, values, goals, and practices that characterize an institution or organization."[2] But I like to use a more anecdotal concept: "How do your employees' stomachs feel on Sunday night about work on Monday?" Culture is the vibe within your organization and influences how everyone experiences it. A strong, supportive culture has been shown to help employees feel a sense of purpose and adapt to change. It can also be a predictor of employee satisfaction, employee commitment, and the success of quality improvement initiatives.

A study of employee turnover in the United States during the "Great Resignation" of 2021 determined that organizational culture had 10 times the impact of compensation.[3] Similarly, a 2018 study published in the *Harvard*



The outside perspective of a third-party facilitator can be helpful for leadership and staff when it comes to defining organizational culture.

Business Review determined that 9 out of 10 employees would accept less money to do more meaningful work.[4] My own firm has faced the challenge of retaining talented employees, and we've come to the realization that compensation alone doesn't work. Historically, when an employee has received a job offer from another company, my firm has had about a 20% success rate of retaining them by offering more money. Furthermore, even if the employee initially accepts our counteroffer, they typically remain with us for less than 6 months. Faced with these facts, our leadership decided we needed to rethink our approach to retention, with greater emphasis on our workplace culture.

How to Improve Workplace Culture

The first step to improving workplace culture is to understand and define what you want your culture to be. An organization's leadership may not have the objectivity needed to see opportunities for improvement. Engaging an outside facilitator can help bring needed perspective and direction. During my firm's strategic planning process, we found that having a third-party coach to guide our leadership team and staff was critical to the success of our culture-shaping journey.

We began with the idea of treating our culture like a thermostat instead of a thermometer. A thermostat allows you to set the temperature you want, while a thermometer reads the current temperature. Simply reading the temperature of your culture, as you would read a thermometer, can lead to a reactionary response to the current environment. In contrast, if you set the desired temperature of your culture, as you would set a thermostat, you can be more intentional in defining what your culture should be and you have opportunities to adjust the settings as needed. For this method to be effective, it is important for organization leaders to listen with intention, seek to understand, and proactively make the required adjustments.[5]

At the same time that we set our cultural thermostat, we embraced a new leadership model. Traditional organizational hierarchy has leadership at the top and employees at the bottom, supported by customers. In this model, the leaders make the decisions, and the employees are expected to follow, while the customers, or clients, provide work to keep the business moving. We decided to replace that model with a "servant leadership" model, which creates a new paradigm in how we think about leadership and its impact on culture. In this model, leadership places their employees and customers at the top of the hierarchy and serves them by supporting their needs and empowering them.

In a servant leadership model, employees are recognized as the organization's greatest asset, which leads to more empowered, engaged, and productive teams. In my experience, organizations that shift the leadership's mindset from personal power to serving others are more likely to retain talent than those that follow the traditional hierarchy.

Calibrating Your Leadership Model

When adjusting your leadership model, it is important to calibrate your thinking to achieve buy-in from your team, which can be done using the following four steps:

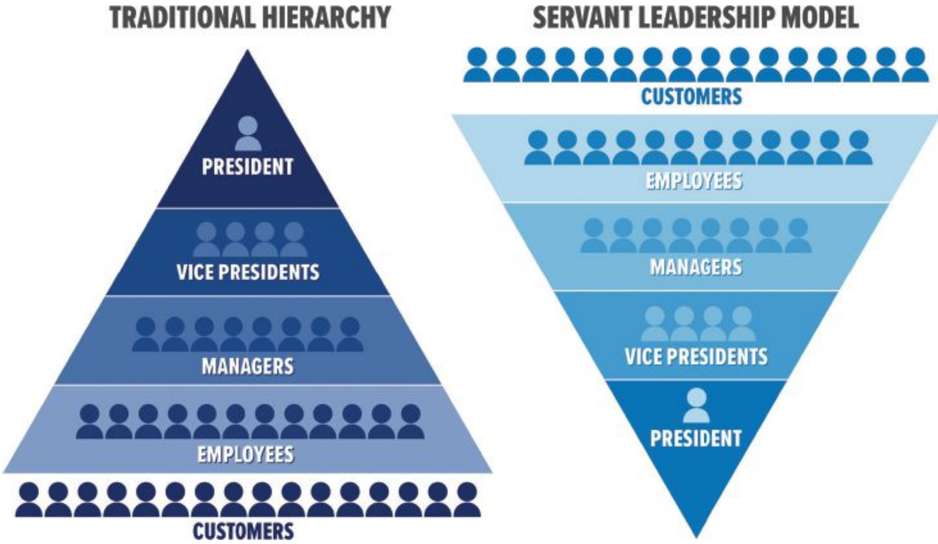
- **Step 1: Define the mission.** What is your leadership model? Should changes to that model be considered to support cultural changes? Involve your team in the development or adjustment of the mission. Seek feedback and clarify the mission. The development of a strategic plan to prioritize initiatives that you want to advance may be a key step in this process.

- **Step 2: Establish the vision.** Define where you want the organization to go. This is an opportunity to dream about what is possible. Engage staff to identify the collective vision of the team and everyone's role. Employees who help develop the mission and vision for the company will be invested and more likely to stay and build it.

- **Step 3: Define your values.** These values will be your compass. Many firms have core values posted on their walls, but those values are not in use daily. Core values should be integrated into everyday actions and be part of the formula used to measure progress on expectations. Revisit them periodically to ensure that they remain important to your team, your leadership, and your clients.

- **Step 4: Develop a strategy.** Bring the mission, vision, and values to life. This process could involve additional opportunities for feedback from staff and targeted conversations with leadership. We have created a culture committee, with representatives from the firm at all levels of the organizational structure, to ensure that every employee has a voice in setting our company's culture.

Proactively engaging your staff in these four steps promotes ownership of the process and demonstrates a transparent, clear, and authentic presence that your talent will gravitate toward.



A traditional hierarchy compared with the servant leadership model.

Final Thoughts

With talent in high demand, building an intentional workplace culture is one of the smartest investments an organization can make. Developing a positive workplace culture requires continual effort, participation, and acceptance at all levels of the organization. At my firm, we saw small-scale immediate benefits within our teams, but it has taken about 3 years of targeted, intentional effort to achieve measurable results in employee retention and recruitment. A strong culture starts with knowing your history—but it thrives when you are open to new voices, willing to adapt, and focused on the future.

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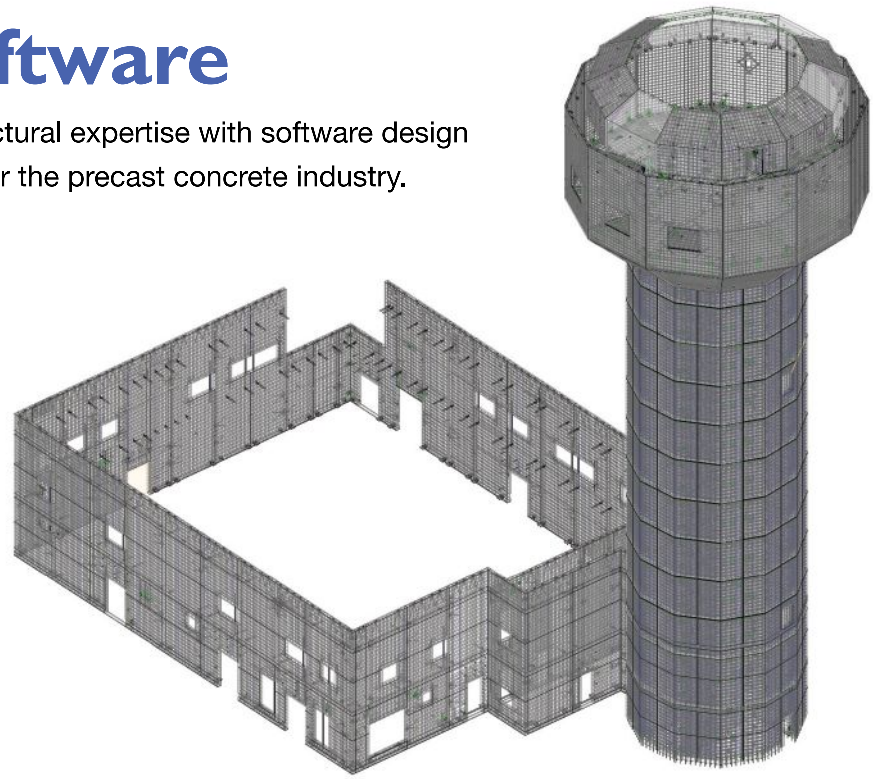
FOCUS

Eriksson Software

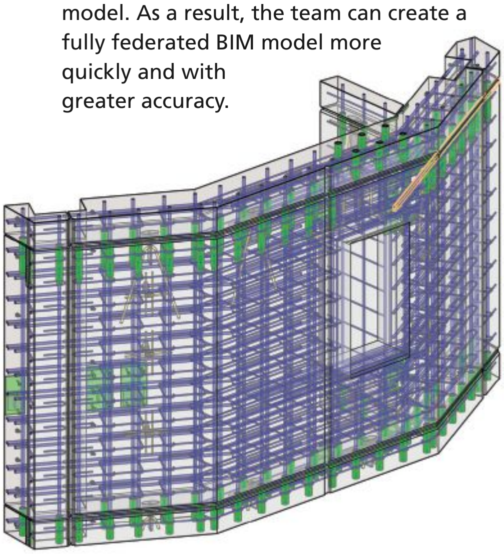
Eriksson Software combines structural expertise with software design knowledge to provide solutions for the precast concrete industry.

by Monica Schultes



The use of Eriksson Sync Software automation reduces the time it takes to complete a complex project. The Teterboro air traffic control tower project received a 2025 PCI Design Award for the use of building information modeling to perform highly accurate geometrical modeling. All Photos and Figures: Eriksson Software.

Like many successful entrepreneurs, Roy Eriksson, founder and principal of Eriksson Technologies and Eriksson Software, started his business from his home. In October 1998, he incorporated Eriksson Technologies, a firm that leverages his expertise in both structural and software design. Before starting his own enterprise, Eriksson worked for a large consulting firm, but it was his 10 years at LEAP Associates that he credits for igniting his passion to create software that improves engineering processes. He chose to name his firm Eriksson Technologies to signify the combination of both engineering and information technology in one company. The firm's first software product was a Windows-based program used to design precast, prestressed concrete highway bridge girders. "That was a significant undertaking at the time," Eriksson recalls. "It sold well, so I expanded into design and consulting work. As the company grew, we started doing more software and more engineering."



The use of Eriksson Sync Software places the necessary reinforcement directly into the model, eliminating the need for the BIM modeler to create them within the model. As a result, the team can create a fully federated BIM model more quickly and with greater accuracy.

In October 2012, Eriksson Software was spun off from Eriksson Technologies to become a separate company. Eriksson Technologies now employs 25 people, and Eriksson Software has 10 employees. Their headquarters are in Tampa, Fla., with a satellite office in Denver, Colo., and a half-dozen fully remote employees scattered across the United States.

Synergies

While Eriksson Software and Eriksson Technologies maintain separate offices and separate staff, the two companies work closely together. Roy Eriksson and his business partner, Brian Barngrover, are the common denominators. They oversee both operations, and, when necessary, employees of one firm will share their time and talent with the other company, which is invoiced for the work performed.

"The engineering services firm [Eriksson Technologies] is a power user of Eriksson Software products and tools. They assess the functionality and performance of the software and are early users of new products and generate a wellspring of ideas," explains Eriksson. "The strength of our operation stems from the symbiosis between the two firms. The engineering group benefits with the best tools available, and software developers get an inside look at the design process, so the two sister companies exist in harmony."

"The strength of our operation stems from the symbiosis between the two firms. The engineering group benefits with the best tools available, and software developers get an inside look at the design process, so the two sister companies exist in harmony."

A panel is erected on the Teterboro Airport air traffic control tower project. Eriksson Technologies used building information modeling (BIM) to perform construction engineering and prepare the detailed fabrication drawings for the project.

Industry Associations

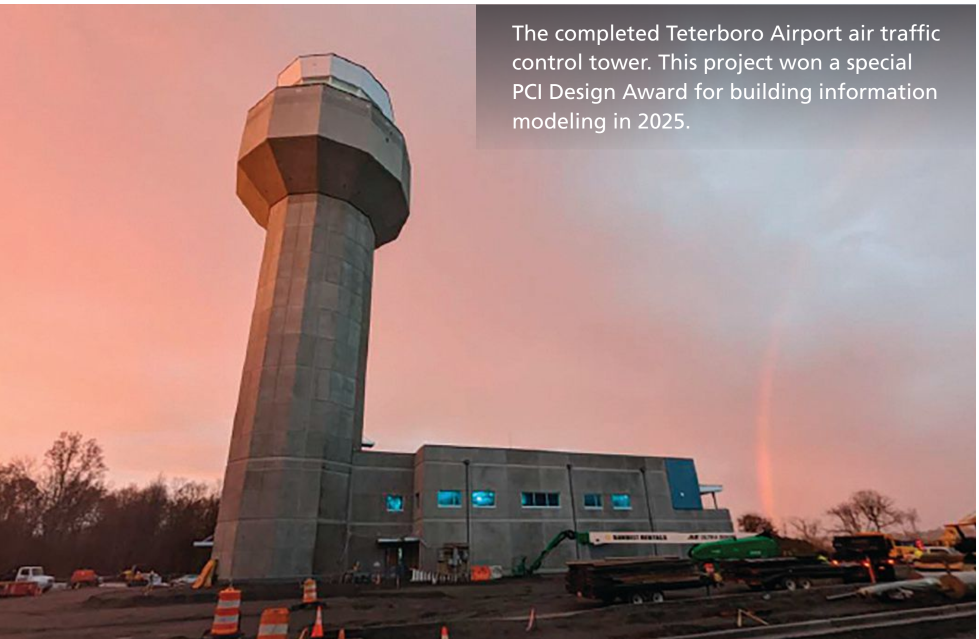
Eriksson has been a contributor and supporter of PCI for his entire career, and his team has followed suit by participating in PCI technical committee work and other volunteer efforts. This collaboration has been beneficial for PCI, the industry, and Eriksson personally. "PCI is particularly important to us, to our work, to our careers," he says. "Some of the best engineers and academicians serve on and contribute to technical committees alongside the precasters, consulting engineers, and public agencies." Eriksson employees also contribute to the industry by monitoring code-writing bodies such as the American Concrete Institute (ACI) and the American Association of State Highway and Transportation Officials (AASHTO).

Software for the Precast Concrete Industry

Eriksson Software focuses primarily on the development of engineering software for the analysis and design of precast concrete elements. End users of their software include fabricators, engineering firms, transportation agencies, and other owners. Clients who use Eriksson Software products can store their data anywhere; some clients have local servers, while others use cloud-based systems

Other Eriksson Software offerings in the bridge engineering market include Eriksson Pile, a program for the analysis and design of precast prestressed concrete piles; ETPier, software for the analysis and design of bridge substructures; and Eriksson Culvert, which is used for precast concrete or cast-in-place concrete culvert design. Future releases of all these programs will be compatible with building information modeling (BIM).

While the precast concrete industry encompasses both buildings and bridges, commercial clients tend to embrace new software technology more readily than those in the transportation sector. That latitude enabled Eriksson Software to develop a suite of software products for commercial structures, which includes products for beams, columns, wall panels, connections, and everything that goes into a precast concrete building. These products share a common data structure, which accelerates product development.



The completed Teterboro Airport air traffic control tower. This project won a special PCI Design Award for building information modeling in 2025.

Evolution from PSBeam to Eriksson Girder

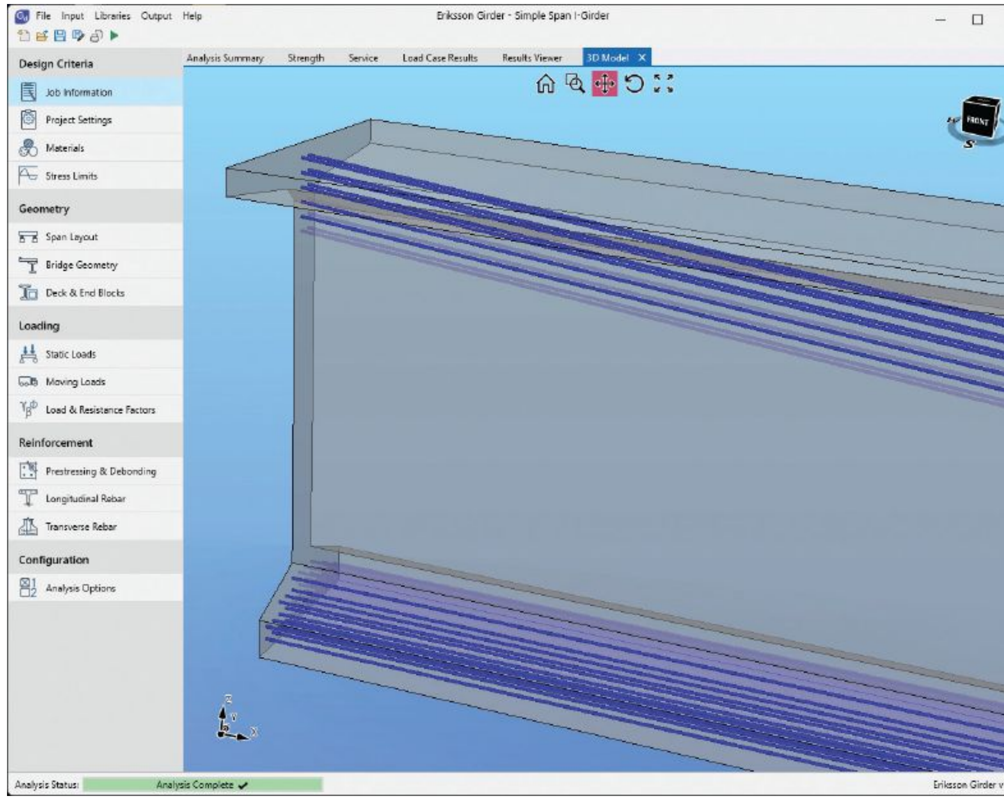
Roy Eriksson has been involved in software since the start of his career and has witnessed the transformation of computers from programmable calculators to laptops with high-resolution monitors and graphics packages. When he founded Eriksson Technologies, his first brainchild was PSBeam, a Windows-based program for the design and analysis of simple- or continuous-span pretensioned or post-tensioned precast concrete bridge girders. Fast forward to 2025: Eriksson Girder, a state-of-the-art Windows-based program, has been released and PSBeam is on its way to retirement.

Eriksson Girder sets a new standard for plant-cast prestressed concrete bridge girder design with a new architecture and data structure that supports current engineering workflows while paving the way for expanded use of building information modeling. Eriksson Girder will also support precast concrete plant automation and project delivery. "This is a bittersweet moment, because I wrote PSBeam but it's time for a new generation of software," says Eriksson.

Customer Support

Software firms use a metric called the "churn" rate, which measures attrition or lost subscriptions when users switch to different software products. Eriksson Software enjoys little churn, which indicates that their customers are satisfied with the products and the support that Eriksson offers.

In addition to offering standardized training, tutorials, and a video library, Eriksson Software stays in touch with customers through an efficient tech support team. "We listen very carefully to



Rendering of the strand pattern for a prestressed concrete girder from Eriksson Girder, a new Eriksson Software program to facilitate the design and analysis of precast, pretensioned concrete bridge girders.

our users and have developed an internal knowledge base. If a feature needs improvement to make it more intuitive, we work with the end user to improve it and provide the best customer support we can," Eriksson says.

The firm maintains a high level of quality control for their software and is up to date with the recently published 10th edition of *AASHTO LRFD Bridge Design Specifications* and other codes. A test suite of problems is routinely performed to evaluate every version and revision. "With complex software, something frequently arises that you do not foresee, like a structural design code interpretation, but we pride ourselves on our quick response," says Eriksson.

Building Information Modeling

Recently, the Eriksson teams of professional programmers and engineers have focused on developing and leveraging BIM. Using BIM software developed by Eriksson Software, Eriksson Technologies has improved accuracy and reduced the time to design and produce fabrication drawings by 25% to 50%. Many U.S. precast concrete producers also use the Eriksson commercial software platform for shop drawings. "We anticipate that they will be on board with BIM," predicts Eriksson.

Eriksson Sync software establishes two-way connectivity between design software and BIM models. This software generates the reinforcing objects from a design and places them within a BIM model. The reinforcement in a BIM model can be edited directly within the model or within the design software. Separate from Sync is BIMpak, software that runs within a BIM platform and provides tools to detail precast, prestressed concrete structural elements within a BIM model. BIMpak also provides the capability to output the design of the structural elements in the form of piece tickets.

Teterboro Air Traffic Control Tower Project

In 2025, Eriksson Technologies was part of the team that won a PCI Design Award in the BIM category for the Teterboro Airport air traffic control tower (ATCT). Eriksson Technologies performed construction engineering and prepared the detailed fabrication drawings. The total–precast concrete ATCT features precast concrete panels that form a cylinder at the base and a sphere at the top level of the tower. As the Eriksson team generated the construction drawings, BIM modeling helped them meet tight tolerances in the field and carefully check for clashes and alignment of reinforcing bar couplers. "For complex projects like this one, BIM is excellent at 3-D [three-dimensional] geometry and modeling accuracy," says Eriksson. "This project was a great fit for BIM, where we maximized the capabilities of the computer to perform highly accurate geometrical modeling."

The use of Eriksson Sync played a key role in the successs of this project because it precisely placed the reinforcement from the structural design directly into the BIM model, eliminating human error. The wall panels in the Teterboro model had a great deal of reinforcement that was interrupted at the joints of the tower. Continuity of the reinforcement across the joints was established with the help of reinforcing bar couplers, which required meticulous alignment for proper fit-up. The use of Sync ensured that bars were placed exactly as required so that when the panels were stacked upon each other the protruding reinforcing bars aligned precisely with the reinforcing bar couplers.

Eriksson Sync is an innovative technology that provides a critical two-way interface between engineering design software and BIM models. Its use profoundly changes the normal design workflow for detailing precast, prestressed concrete structures, as it allows the BIM modeler and the structural engineer to work together as a team. The BIM modeler breaks the project into discrete precast concrete elements, and then the structural engineer interfaces with the model to analyze, design, and generate the reinforcement and structural embedments for each precast concrete element in the model. The design software places the necessary reinforcement directly into the model via Sync, eliminating the need for the BIM modeler to create them within the model. As a result, the team can create a fully federated BIM model more quickly and with greater accuracy.

BIM for Bridges

BIM is slowly being adopted by the transportation industry, and the technology for BIM modeling that Eriksson Software has developed for use in the commercial sector is directly applicable to the bridge sector. To accommodate the use of BIM in transportation projects, Eriksson Software's bridge engineering library has undergone a complete rewrite. After two years of development and beta testing, Eriksson Software launched a new product called Eriksson Girder in March 2025. This new product for the bridge market is a Windows-based program for the design and analysis of simple- or continuous-span precast, pretensioned concrete bridge girders. Eriksson Girder will soon be ready to connect to BIM and is expected to be widely adopted by the bridge market.

A powerful feature of Girder is its load rating capability. It computes inventory and operating load ratings for flexure, shear, and concrete stresses. A future release will permit the user to transfer load rating data for a girder to the National Bridge Inventory database.

The stability of girders during lifting, handling, and hauling is addressed within Girder. The methods used within the program are consistent with those recommended by PCI. However, some of the methods used within Girder are capable of more accurate calculations. Currently, stability is checked up to the point of placement of a girder on the bridge.

"Eriksson Girder replaces PSBeam, and the license extends to our current PSBeam users. They will be able to use both products and eventually we will sunset PSBeam," says Eriksson. He believes that "if we could consolidate the disjointed design-bid-build project delivery method used in the bridge market, the BIM workflow would be more efficient." Design-build projects have the greatest potential for early adoption of BIM for bridges.

Transportation agencies are trending toward using BIM models for project delivery. As state agencies and the industry become more familiar with BIM and its capabilities, two-dimensional drawings will become a thing of the past.

Artificial Intelligence

Eriksson Software builds the capability to fully analyze seismic, wind, and other loads into their software products. "If you have the design criteria, materials, and applied loads, we try to make the applications intuitive for the end user. We incorporate the required load combinations and load factors and so forth into the software," says Eriksson.

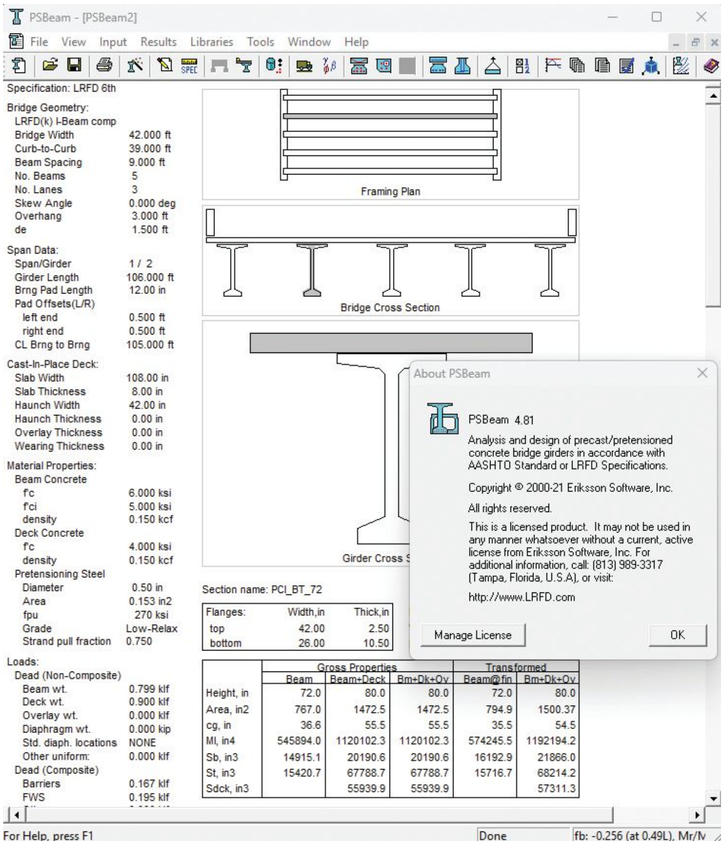
One current and future priority in software development is the appropriate use of artificial intelligence (AI) in the analysis of given loads and constraints to produce the optimal design. Large language models are capable of scanning the web for every usable data source. However, to determine the optimal design of a structure, the value of the data captured by these models is limited because some projects do not comply with current specifications or code.

Instead of scanning copious amounts of data, the Eriksson platform synthesizes only the data that are relevant. "We believe this concept has the potential to help designers accelerate the process even further. That is our version of AI," says Eriksson. "Our algorithms that design precast concrete elements save time by creating a concept plan that is very close to a final design without too much trial and error."

"Our algorithms that design precast concrete elements save time by creating a concept plan that is very close to a final design without too much trial and error."

From its early days as a pioneer in precast concrete software through the recent introduction of Eriksson Sync and Eriksson Girder, Eriksson Software has led the precast concrete industry toward a digital future in which BIM, AI, and other technologies help teams design, build, and maintain bridges more efficiently.

Screen capture of PSBeam output for the design of a sample precast prestressed concrete bridge. The new Eriksson Girder software generates similar data analysis.



PROJECT

Evans Mill Pond Pedestrian Bridge Replacement

by William J. Castle, Childs Engineering


View of the original bridge superstructure looking west, with the concrete spillway to the left and the pond to the right. All Photos: Richard Parisi.

On June 20, 2020, the Camden County Department of Parks in New Jersey retained a bridge consultant to perform an inspection of the pedestrian bridge over Evans Pond at Challenge Grove Park in the borough of Haddonfield, N.J. Above- and underwater inspections were conducted to assess the condition of the structure and determine the necessary repairs or reconstruction required to support pedestrians and park vehicles.

The existing pedestrian bridge was constructed circa 1913. Although there are no records available, it is assumed that the adjacent earthen dam, concrete spillway, and steel sheet-pile wall were built at the same time. The pedestrian structure was a 20-ft-wide, 33-ft-long single-span bridge with 10 steel I-beams and a 3-in.-thick timber deck. The superstructure was supported by concrete abutments with adjacent steel sheet-pile retaining walls on the downstream embankments and a concrete weir on the upstream side. There was also a concrete slab between the abutments for scour control from the upstream dam.

Inspection

Inspectors found that the concrete substructure was in good condition, with only minor abrasion and random hairline cracking. However, the superstructure was in critical condition, with moderate to severe corrosion, and holes in the webs and flanges at various locations on all of the steel beams. The bridge was closed due to the severe deterioration of the beams. The timber deck, parapets, and railing were found to be in overall poor to serious condition. Based on the inspection findings, it was recommended that the bridge remain closed until

New prestressed, precast concrete slab beams are installed.


profile

EVANS MILL POND PEDESTRIAN BRIDGE / HADDONFIELD, NEW JERSEY

BRIDGE DESIGN ENGINEER: Childs Engineering, Hainesport, N.J.

PRIME CONTRACTOR: Walters Marine, Ocean View, N.J.

CONCRETE SUPPLIER: L&L Redi Mix Concrete Inc., Southampton, N.J.

PRECASTER: Jersey Precast, Hamilton, N.J.—a PCI-certified producer

OTHER MATERIAL SUPPLIERS: Deck reinforcement: J.M. Ahle, South River, N.J.; stone facing: Quarry Cuts, Parker Ford, Pa.; solar-powered bollards: First Light Technologies, Victoria, BC, Canada; railing: Susan R. Bauer Inc., Ringwood, N.J.

The completed superstructure replacement features a stamped concrete deck, decorative bridge railings, and lighted bollards. The concrete features a gray cobblestone pattern on the deck flanked by a boardwalk pattern on the sidewalks.

extensive repairs or rebuilding of the superstructure could be completed.

Project Design

The engineering analysis and rehabilitation design of the Evans Mill Pond Bridge began in mid-2021, the completed plans were sent out to bid in July 2021, and construction was completed in early 2022. The bridge design consultant reviewed several different design approaches and decided to remove the existing superstructure down to the concrete abutments. The concrete parapets on the abutments were to be partially removed; otherwise, the abutments were in good condition and required only superficial repairs. The top of the abutments would need to be cut down to accommodate the new superstructure, and the new bridge seat would be coated with epoxy waterproofing.

A dam safety permit was required because portions of the substructure and adjacent retaining walls are considered to be part of the dam. Given the presence of the dam, maintaining the horizontal and vertical clearances was an important design priority. The existing concrete abutments were to remain in place and the underclearance elevation was to remain the same; these decisions maintained the overall waterway opening, which simplified the approval process.

Construction had to progress with no interruption of the waterflow at the dam and no impact to the two overflow pipes at the secondary spillway just past the wingwalls on the west side of the bridge. Construction staging and access also had to be carefully considered to avoid damage to the dam. Adjacent prestressed concrete slab beams were chosen for the superstructure because they could maintain the required clearances and offered the advantages of durability, low maintenance costs, and simplicity in erecting the beams over the waterway with access from the adjacent roads.

New Superstructure

The design criteria for the project were based on the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[1] and the current edition of the New Jersey Department of Transportation's *Standard Specifications for Road and Bridge Construction*.[2] The new superstructure is composed of adjacent prestressed concrete slab beams with a stamped, minimum 3-in.-thick cast-in-place concrete deck and sidewalk. The single-span bridge consists of a 12-ft-wide roadway and 4-ft 1-in.-wide sidewalks with steel bridge railings on both sides. The length of the new superstructure is the same as the previous superstructure, and is supported by new elastomeric bearings on the existing abutments with only minor modifications. Anchor dowels are

Elevation view of new bridge taken from the pond side. Decorative stone facing runs along the existing parapets on the bridge approaches and extends down the outside face of the abutments and wingwalls.

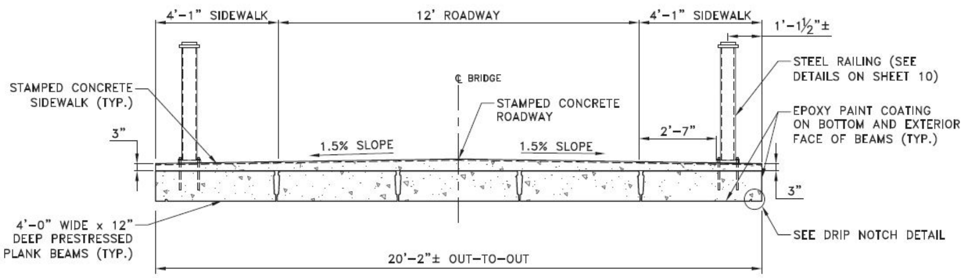
CAMDEN COUNTY PARKS AND RECREATION, OWNER

BRIDGE DESCRIPTION: Single-span, 32-ft 9-in.-long bridge constructed of five adjacent precast, prestressed concrete slab beams with a 12-ft-wide roadway and 4-ft 1-in.-wide sidewalks on both sides.

STRUCTURAL COMPONENTS: Existing reinforced concrete abutments; five 4-ft-wide, 12-in.-deep precast, prestressed concrete slab beams; and a 3-in.-thick, noncomposite cast-in-place reinforced concrete deck.

BRIDGE CONSTRUCTION COST: $310,000

General elevation view of new bridge taken from the pond side, looking southwest. Decorative stone facing runs along the existing parapets on the bridge approaches and extends down the outside face of the abutments and wingwalls.



Bridge typical section showing the 12-ft-wide roadway and 4-ft 1-in.-wide sidewalks with steel bridge railing on both sides. All Figures: William J. Castle.

used for the beam connections to the substructure, with one end considered fixed and one expansion.

Each of the five precast, prestressed concrete slab beams is 4 ft 0 in. wide, 32 ft 9 in. long, and 12 in. deep. The beams are connected using grouted shear keys and transverse tie rods. The precast concrete slab beams are designed to accommodate an H10 truck live load for park vehicles and a pedestrian live load of 75 lb/ft^2, as required by the owner. The shallow-depth, adjacent prestressed concrete slab beams were designed with a minimum concrete compressive strength of 6800 psi at transfer and 8000 psi at 28 days, and seven-wire Grade 270 prestressing strands—14 strands for the interior beams and 20 strands for the fascia beams. The noncomposite cast-in-place concrete deck is 32 ft 9 in. long and 20 ft 2 in. wide. The deck is 3-in. thick, required a minimum concrete compressive strength of 4000 psi, and is reinforced with welded-wire reinforcement.

The texture and color of the stamped concrete deck and sidewalks were approved by Camden County, which wanted the bridge aesthetics to blend into the park environment. The concrete features a gray cobblestone pattern on the deck flanked by a dark gray boardwalk pattern on the sidewalks. The steel bridge railings are coated in a black matte zinc coating and anchored into the concrete deck and slab beams. Solar-powered bollards are located on both ends of the new structure. New decorative stone facing was installed along the existing parapets on the bridge approaches and extends down the outside face of the abutments and wingwalls.

Conclusion

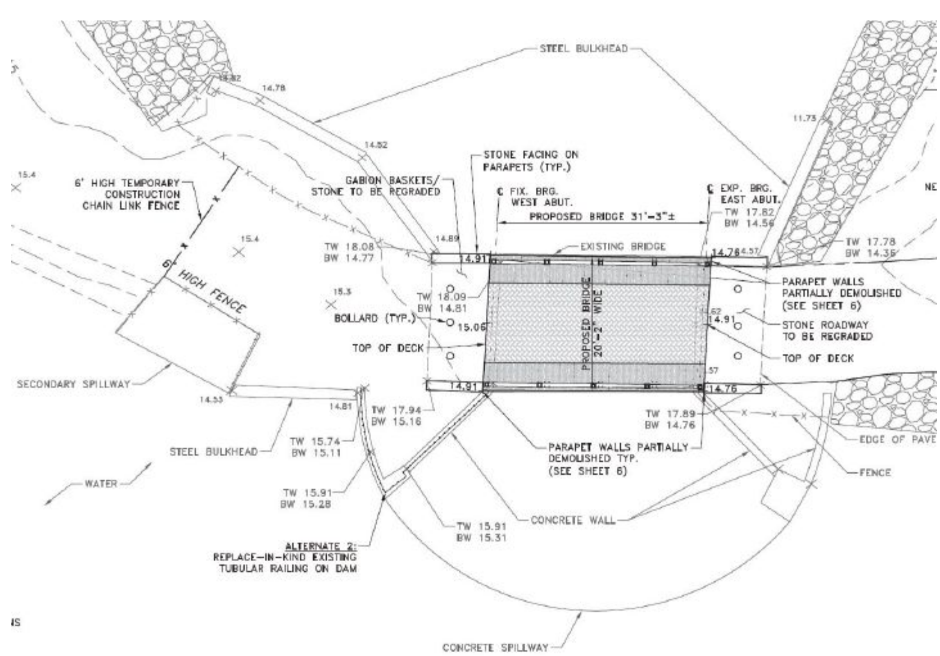
The contractor had to ensure that there were no adverse effects to the dam during construction and no operational or flow disruptions. Using prestressed concrete beams simplified the reconstruction of the bridge superstructure and helped control the final costs of the project. In addition, the aesthetics of the bridge complement the surrounding park area. The bridge was completed within time and on budget and opened for use in May 2022.

References

1. American Association of State Highway and Transportation Officials (AASHTO). 2020. *AASTHO LRFD Bridge Design Specifications*. 9th ed. Washington, DC: AASHTO.
2. New Jersey Department of Transportation (NJDOT). 2019. *Standard Specifications for Road and Bridge Construction*. Trenton: NJDOT. https://www.nj.gov/transportation/eng/specs/2019/pdf/StandSpecRoadBridge_20190528.pdf.

William J. Castle is a licensed professional engineer with Childs Engineering in Hainesport, N.J., and a recognized leader in marine structural engineering.

The Evans Mill Pond Pedestrian Bridge site plan showing the adjacent earthen dam, concrete spillway, and steel sheet-pile walls.



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PROJECT

When Engineering Meets Architecture: The Story of the Bend Bridge

by Mario J. Quagliata and Matthew C. Wagner, Colliers Engineering & Design

Metroparks Toledo is reshaping the Toledo, Ohio, riverfront with an exciting vision to develop 300 acres of vibrant green space. As part of its dedication to fostering community connections, Metroparks Toledo is committed to creating dynamic public spaces where visitors can engage with nature and each other. The Glass City Riverwalk will feature more than 5 miles of new mixed-use trails and paths to link communities on both sides of the Maumee River. The $57 million project also includes a wetland walks, a refurbished lighthouse, docks, and natural playgrounds, among other amenities. A standout feature is the new Bend Bridge. This architectural marvel provides pedestrian access from the Glass City Riverwalk to the Martin Luther King Jr. Bridge (MLK Bridge), which crosses the Maumee River and connects to parkland on the east side of the river.

View of the Bend Bridge looking toward the connection at the Martin Luther King Jr.(MLK) Bridge. Photo: Metroparks Toledo.

Superstructure

The project team—consisting of the owner, engineer, architect, and contractor—selected cast-in-place reinforced concrete for its flexibility in shaping complex geometry. The bridge features back-to-back horizontal curves: a tight, 36-ft radius curve followed by a more gradual 290-ft radius curve leading to the MLK Bridge. This distinctive alignment inspired the bridge's nickname, the Bend Bridge.

The structure is 303-ft-long and consists of seven continuous spans, varying from 41 to 55 ft, with one 6-ft cantilever span that abuts the MLK Bridge. The deck provides a 15 ft 9 in. clear width for pedestrians and bicycle users and provides access from the riverfront parkland to the MLK Bridge. The slab superstructure of the Bend Bridge has a curved bottom surface with thickness ranging from 12 in. on the outside to 27 in. at the center. Glass seeding was used on the top surface to enhance aesthetics and recall Toledo's history as the Glass City. To create the glass-seeding finish, the contractor broadcast a mix of tumbled glass (blue, white, and gray colors) on the fresh concrete surface, just after it was finished. The next day, the

profile

BEND BRIDGE, TOLEDO, OHIO

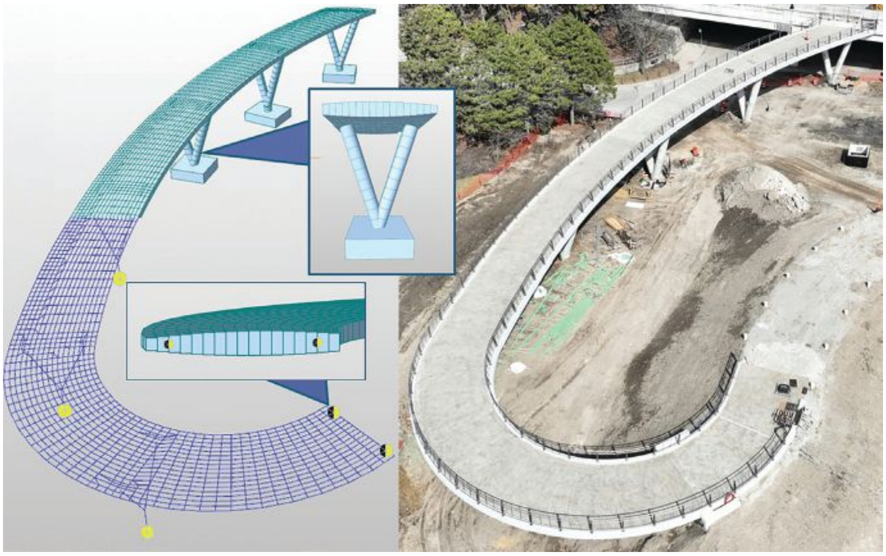
BRIDGE DESIGN ENGINEER: Colliers Engineering & Design, Toledo, Ohio

Other Consultants: Architect: WXY, New York, N.Y.

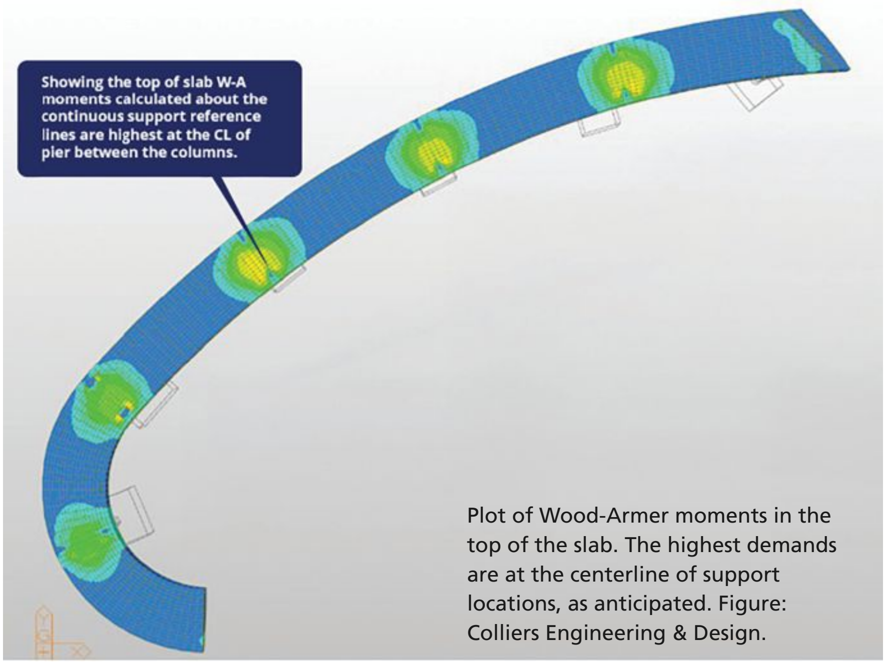
PRIME CONTRACTOR: Kokosing Construction, Westerville, Ohio

CONCRETE SUPPLIER: Kuhlman Corporation, Toledo, Ohio

OTHER MATERIAL SUPPLIERS: Custom metal formwork (piers): CFC Manufacturing, Carbondale, Pa.; custom foam inserts (piers): Global Foams, Dayton, Ohio; reinforcing bar supplier: CMC Rebar, Muncie, Ind.; reinforcing bar installer: Flatrock Bridge Group, Maumee, Ohio; stainless steel railing: Forms+Surfaces, Pittsburgh, Pa.



Three-dimensional finite element mesh (left); an aerial view of the actual bridge layout (right); and model renderings of the V-shaped piers and curved slab superstructure (inset). Figures and Photo: Metroparks Toledo.



Plot of Wood-Armer moments in the top of the slab. The highest demands are at the centerline of support locations, as anticipated. Figure: Colliers Engineering & Design.

contractor power-washed the surface to remove the paste from the glass. This treatment, which was used on the deck surface and on all the path sidewalks in the park, gives the concrete a pop of color and an interesting look. Stainless steel hand railings are mounted on 1 ft wide × 6 in. tall concrete curbs that also house conduit for embedded LED lighting. A 1-in.-wide open joint with a sliding cover plate at the MLK Bridge interface allows independent movement between the two structures while providing compliance with the Americans with Disabilities Act accessibility requirements.

Substructures

The project team collaboratively designed the piers to serve both structural and architectural roles. Cylindrical or flared columns were initially considered for simplicity, but the project architect proposed a V-shaped pier design with varying oval cross sections to provide a more distinctive sculptural form. The legs range in transverse width from 3 ft at the base to about 4 ft at the top and are 2½ ft wide in the longitudinal direction.

The contractor developed custom forms with foam inserts at the base to create a smooth saddle (or fillet) where the legs converge. The bridge design team ran preliminary analyses, which confirmed that the elements would withstand the anticipated applied loads and internal forces. The piers are fully integral with the superstructure. Advanced analysis and design were required to produce the visually seamless form. The piers vary between 10 and 22 ft in height, with grouped pier heights that allowed the reuse of formwork. Each pier is supported by vertical H-piles driven to bedrock. Compared with battered piles, vertical H-piles provide more lateral flexibility in the foundations, so the superstructure can more easily accommodate creep and shrinkage over the long term.

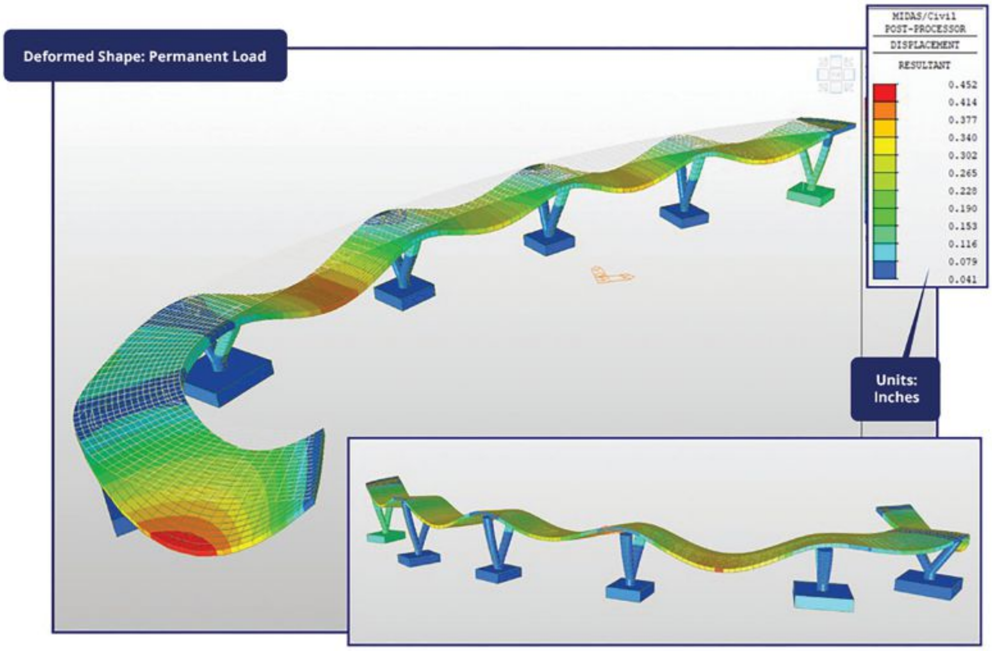
The abutment at the low end of the bridge features a more standard semi-integral design. The abutment is supported on vertical and battered piles, with elastomeric bearing pads supporting and accommodating the movement of the superstructure. The elastomeric bearings sit on top of the abutment beam seat. They are vulcanized to galvanized steel sole plates that are embedded into the superstructure concrete with shear studs. An approach slab and sleeper slab were also used to provide a smooth transition onto the bridge.

METROPARKS TOLEDO, OWNER

BRIDGE DESCRIPTION: 303-ft-long, continuous cast-in-place concrete slab structure supported on six integral V-shaped concrete piers. The bridge has seven 41- to 55-ft-long spans plus a 6-ft cantilever span. The deck provides a 15 ft 9 in. clear width for pedestrians and bicycle users.

STRUCTURAL COMPONENTS: Cast-in-place concrete slab superstructure with a curved bottom surface that varies in thickness from 12 to 27 in. The V-shaped piers vary in height from 13 to 23 ft with oval-shaped legs that vary in width from 3 ft at the bottom to about 4 ft at the top. Each pier is supported on nine vertical HP12x53 H-piles that are driven to bedrock.

BRIDGE CONSTRUCTION COST: $2.6 million total ($475/ft^2)



Plot of exaggerated deflections due to application of permanent loads. Both the pier column and slab formwork elevations were intentionally overbuilt to allow for short- and long-term displacements. Figure: Colliers Engineering & Design.

Funding

The project was primarily funded by a federal Better Utilizing Investments to Leverage Development (BUILD) grant, which was secured for the bridge and the broader riverfront parkland development. The grant was tied to an aggressive timeline to start construction, which added pressure to complete the design phase of the project efficiently. Design was completed in 8 months between January and August 2022.

Analysis and Design

To meet the project's structural and aesthetic goals, the bridge design team developed a detailed finite element analysis (FEA) model to evaluate structural demands and deformations. This approach enabled accurate simulation of complex structural behaviors and facilitated precise reinforcement detailing and construction planning. The modeling began with creating a mesh in computer-aided design software, aligning nodes orthogonally to the bridge's curved centerline to match the direction of primary transverse reinforcement.

Quadrilateral elements were selected for their accuracy and computational efficiency. A critical parameter in model development is the element aspect ratio (AR), and while elements with an AR close to 1 yield high accuracy, they significantly increase computational runtime and complexity, resulting in unnecessary model troubleshooting challenges. The final FEA model consisted of 2506 nodes, 136 beam elements representing structural members such as columns and pile caps, and 2263 shell elements modeling the concrete slab. This complex geometry was managed through 28 analytical domains, each grouping elements by similar local bending axes. These domains were essential for efficiently applying Wood-Armer moment calculations, which were pivotal in the slab reinforcement design. (Those calculations are described in the next section of this article.)

An important and common question in finite element modeling is "How accurate is accurate enough?" For the Bend Bridge design, targeting an AR of less than 5 struck a practical balance between model accuracy and computational runtime.

Pile foundations and elastomeric bearings were represented through point springs and elastic links. Horizontal stiffness from foundations was captured using springs and was adjusted based on geotechnical data. Substructure components such as columns and pile caps were modeled explicitly as beam elements, whereas elastomeric bearings at abutments were modeled as equivalent springs derived from their physical properties. The deck slab was modeled using shell elements divided transversely into 1-ft-wide strips, which were each assigned an average thickness based on the slab's cross-sectional variation. This method provided an accurate representation of bending and torsional moments in the slab.

Wood-Armer Method

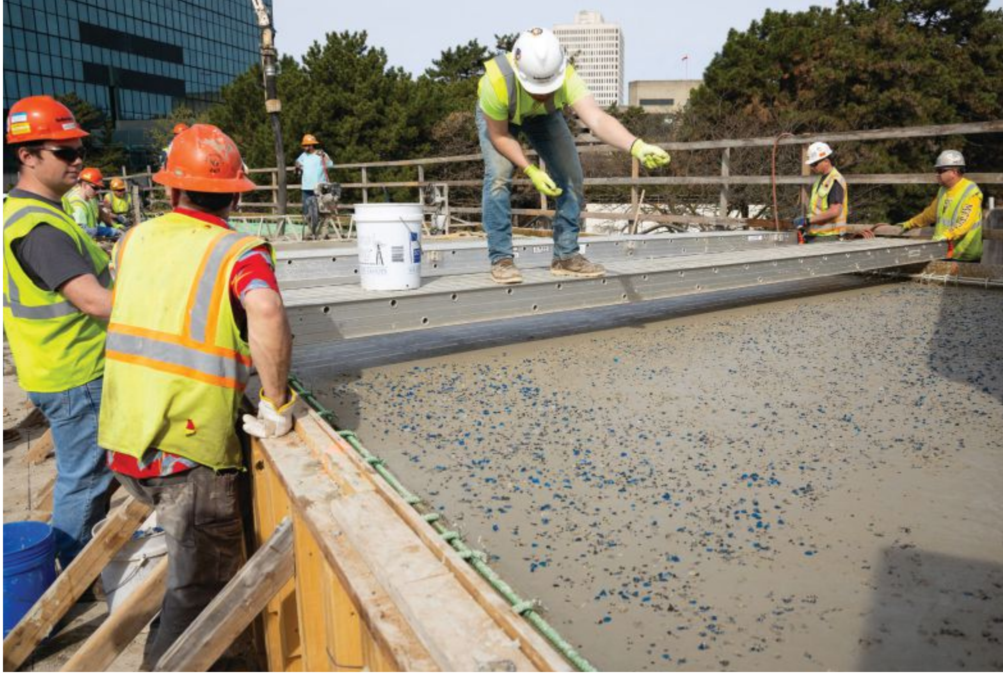
Given the Bend Bridge's unique curved geometry and V-shaped integral piers, the traditional strip method design approach for the slab would have been inadequate as it would result in underestimating the design stresses. The Wood-Armer method—developed for plate and shell elements—was adopted to accurately capture combined bending and torsional stresses, which are especially prominent near slab edges and support regions. Because traditional, slab strip methods primarily address direct bending moments M_x and M_y but often neglect twisting moments M_{xy}, which occur in plates, such methods may underestimate reinforcement requirements and compromise structural performance.

The Wood-Armer equations explicitly account for twisting moments, combining them with direct bending to calculate equivalent reinforcement moments in longitudinal and transverse directions. The Bend Bridge design leveraged analytical domains to ensure that alignment of the reinforcement accurately reflected principal-stress directions.

Results from the Wood-Armer analysis provided clearly enveloped demands

Elevation view showing custom forms used to construct the V-shaped piers. Photo: Kokosing Construction.

Elevation view of one of the finished V-shaped piers showing the architectural form, including the smooth fillet area between the column legs. Photo: Colliers Engineering & Design.



Workers perform the first step in creating a glass-seeding finish by applying a mix of tumbled glass (blue, white, and gray colors) on the fresh concrete surface, just after it was finished. The glass seeding enhances the bridge aesthetics and recalls Toledo's history as the Glass City. Photo: Colliers Engineering & Design.

and maximum stresses. Critical areas requiring additional reinforcement were identified by the design team and addressed in the plan details. Slab-to-pier interfaces, which experience significant stress concentrations, received special attention. Elastic links were strategically used to represent the load transfer between the columns and slab nodes without artificially increasing stiffness, thus avoiding locked-in stresses, modeling singularities, or other common inaccuracies that may occur when simplified boundary conditions are used.

Consideration of Time-Dependent Deformations

The analysis also explicitly considered time-dependent deformations—primarily concrete creep and shrinkage—to evaluate long-term structural integrity and inform decisions about reinforcement detailing. Concrete creep, defined as the increase in strain under sustained loading, gradually alters internal stress distributions. Shrinkage results from moisture loss, inducing internal tensile stresses in restrained elements. These phenomena significantly influence long, multispan reinforced concrete structures such as the Bend Bridge, and they could potentially cause distress, increased deflections, and compromised serviceability if ignored. The design team incorporated analyses of both creep and shrinkage, providing accurate predictions of their long-term impact on the structure.

The detailed evaluation of creep and shrinkage used equations from the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[1] and time-step analysis considering the age of concrete for each component based on the anticipated construction schedule. This evaluation directly influenced key design decisions, including the following:

- Staged construction. The design accounted for phased construction, recognizing time-dependent changes in geometry and stresses with specific temporary shoring requirements incorporated into the contract documents to achieve the desired profile grade.
- Formwork and shoring. The V-shaped columns, which are inherently flexible, were intentionally built slightly higher than final profile elevations to counter anticipated creep- and shrinkage-induced displacements. Although this approach initially seemed counterintuitive, it ensured correct final elevations and improved ride quality.

The analysis results directly informed predictions about both short- and long-term displacements from sustained loads during and after construction. Coordination of these details, including intentional overbuilding of pier column and slab formwork elevations, significantly mitigated field issues, ensuring precise shoring construction, accurate reinforcement placement, and adherence to design tolerances. This approach ultimately will improve ride quality and structural performance throughout the bridge's service life.

Conclusion

The Glass City Riverwalk Bend Bridge exemplifies modern structural analysis and design practices, demonstrating how advanced analytical methods such as finite element modeling and Wood-Armer reinforcement design can effectively facilitate the realization of complex architectural visions. By prioritizing meticulous modeling, precise detailing, and proactive consideration of long-term behavior of the structural material, the team delivered a structure that is functional and durable, and will serve as a striking landmark bridge within the Toledo Metroparks System.

Reference

1. American Association of State Highway and Transportation Officials (AASHTO). 2020. *AASHTO LRFD Bridge Design Specifications*. 9th ed. Washington, DC: AASHTO.

Mario J. Quagliata is regional discipline leader: bridges and structures, and Matthew C. Wagner is geographic discipline leader: bridges and structures for Colliers Engineering & Design in Lansing, Mich.

PROJECT

Innovative Post-Tensioned Concrete Slab Superstructure Elevates New Landmark Florida Bridge

by Rafal Wuttrich and George C. Patton, H&H

Aerial view of the completed State Route 80/Southern Boulevard Bridge. Photo: Johnson Brothers Corp.

The Southern Boulevard Bridge is the southernmost of three drawbridges that connect West Palm Beach, Fla., across Lake Worth Lagoon and the Atlantic Intracoastal Waterway, to the barrier island town of Palm Beach. This bridge has served as an important transportation link since its original construction in 1950. After conducting a project development and environment study, the Florida Department of Transportation (FDOT) concluded that the existing drawbridge needed replacement to address the poor structural condition from corrosive deterioration and the substandard roadway section. The new drawbridge serves one of Florida's most influential communities, so it was important that it be a landmark structure consistent with the Royal Park and Flagler Memorial drawbridges to the north.

The new two-lane structure is 948 ft long and consists of a 228-ft-long, double-leaf rolling bascule main span with five 72-ft-long approach spans on each side. The overall width of the bridge varies from approximately 60 ft for the east approach spans and the moveable span to 72 ft for the west approach spans. The additional width for the west approach spans accommodates a left turn lane. The new bridge was constructed on the same straight alignment as the existing bridge, while traffic was maintained on a temporary bridge, with a vertical lift span, on an offset alignment. The design and construction of the approach

profile

STATE ROUTE 80/SOUTHERN BOULEVARD BRIDGE, PALM BEACH/WEST PALM BEACH, FLORIDA

BRIDGE DESIGN ENGINEER: H&H, Tampa, Fla. (specialty construction engineer and approach-span redesign engineer of record)

OTHER CONSULTANT: Primary engineer of record: AECOM, Tampa, Fla.

PRIME CONTRACTOR: Johnson Brothers Corp., a Southland Company, Orlando, Fla.

CONCRETE SUPPLIER: Titan Florida LLC, Deerfield Beach, Fla.

PRECASTER: Capitals for approach span support: Johnson Brothers Corp., a Southland Company, Orlando, Fla.

POST-TENSIONING CONTRACTOR: Freyssinet USA, Sterling, Va.

The completed east approach spans with the drawbridge visible in the background. Photo: H&H.

span superstructures are highlighted in this article.

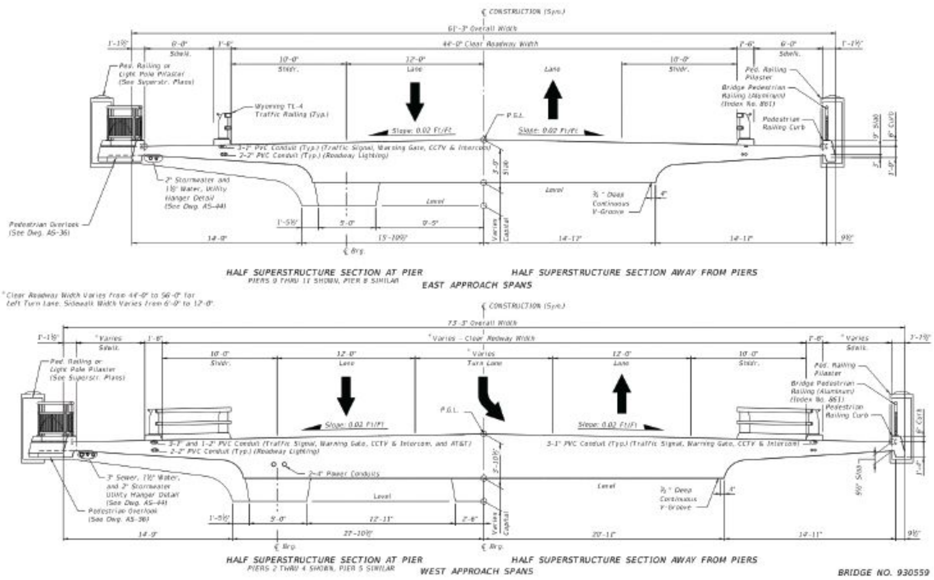
Design Constraints and Considerations

The new bridge had to meet current U.S. Coast Guard navigation requirements, including navigation clearances of 125 ft horizontal and 21 ft vertical. Those new clearances were, respectively, 45 ft and 11 ft greater than the horizontal and vertical clearances of the existing bridge. To achieve the increased clearances, the roadway vertical profile had to be raised by using steeper, 5% grades. The vertical profile was constrained by the adjacent intersection with Flagler Drive, only 100 ft west of the west bridge end. Even with a higher profile, the approach spans' lowest members are less than 12 ft above mean high water, which is the minimum height required by FDOT *Structures Design Guidelines* to minimize exposure of the superstructure to salt water. The new approach spans are also below the maximum wave crest elevation of 18.5 ft, which subjects the superstructure to wave loading during major coastal storm events. To achieve the 75-year design life considering the profile constraints, the superstructure included a combination of 3-in. bottom cover and FDOT Class V concrete with a 6.5-ksi 28-day minimum compressive strength, Type II (MH) cement, fly ash, and silica fume.

For this site, FDOT recognized the advantages of using continuous, cast-in-place concrete slabs—longitudinally and transversely post-tensioned—rather than conventional precast, prestressed concrete girders. The slab cross section is similar to that of a typical segmental concrete box girder, with long, slender cantilever wings on each side, but with a shallow solid core instead of an open trapezoidal core. The wings are approximately 15 ft long and 9.5 in. deep at the cantilevered end, and they taper at a slope of 1:16. The core for the west approach spans is approximately 42 ft wide, with a maximum depth of 3.88 ft at the center, while the core for the east approach spans is approximately 30 ft wide, with a maximum depth of 3.75 ft at the center. The slabs include integral tapered capitals at the piers that align with the pier columns. The advantages of the solid slab compared with conventional precast, prestressed concrete girders at this site include the following:

- A shallower structure depth that maximizes clearance above the salt water.
- A more durable structure with less surface area exposed to salt water.
- A shallower profile that significantly reduces wave forces, which improves the likelihood the bridge will survive a major coastal storm event. (The slab was

Typical cross sections of the approach spans. The west approach span is 12 ft wider than the rest of the bridge to accommodate a left-turn lane. Both sections share a common wing geometry. Figure: H&H.



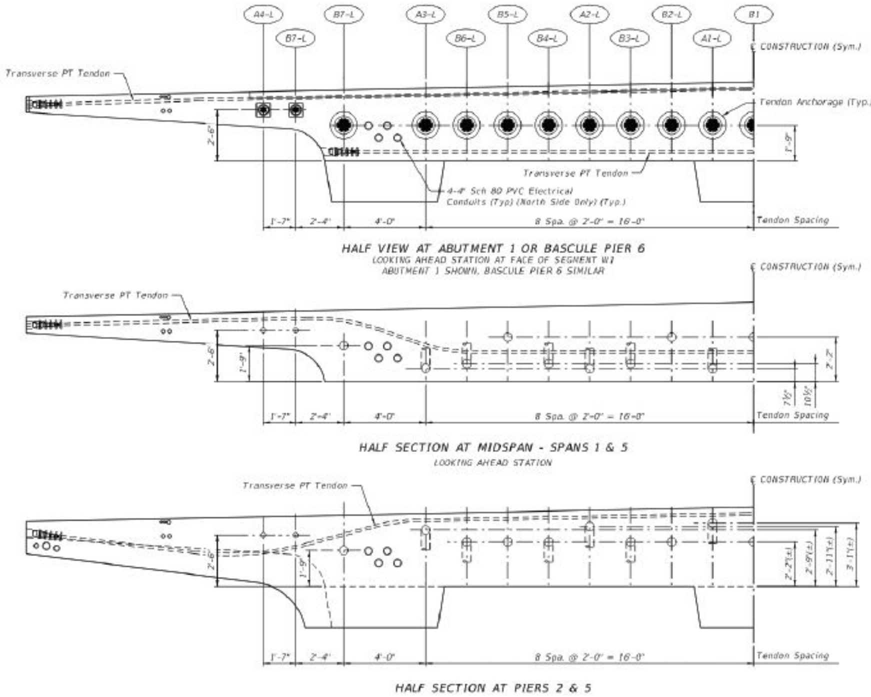
FLORIDA DEPARTMENT OF TRANSPORTATION, DISTRICT 4, OWNER

OTHER MATERIAL SUPPLIERS: Falsework fabrication: Wheelblast Inc., Zephyrhills, Fla.; formwork design and fabrication: EFCO, Orlando, Fla.

BRIDGE DESCRIPTION: 948-ft-long bridge with 228-ft-long, double-leaf rolling-bascule moveable span and 360-ft-long, five-span continuous concrete approach units on each side (72 ft per span)

STRUCTURAL COMPONENTS: Approach spans: longitudinally and transversely post-tensioned concrete slab superstructure on reinforced concrete piers with drilled shaft foundations

BRIDGE CONSTRUCTION COST: $93 million (estimated total contract value for construction of the main span, the approach spans, tide-relief bridge, and associated roadway work)



Half sections of the west approach spans showing longitudinal and transverse post-tensioning. Figure: H&H.

the only alternative that could reasonably meet an "extremely critical" importance level and "service immediate" performance level at the strength limit state as described in the American Association of State Highway and Transportation Officials' *AASHTO Guide Specifications for Bridges Vulnerable to Coastal Storms*.[1])

- A greater amount of prestressing force, designed to provide net compression across the construction joints under service loading, and bonded reinforcing steel across the construction joints for crack control.
- Improved aesthetics with a visually slender superstructure, clean and uncluttered soffit, no dark shadows between beams, and full aesthetic integration with substructure.

The approach spans were originally designed to be built using the incremental launching construction method, with 36-ft-long segments (equal to half of one span) cast on temporary casting beds in the first and last spans of the bridge and then jacked uphill toward the bascule pier on launching beams. The contractor saw an advantage in constructing 72-ft-long segments (the length of a one span) cast on falsework over water. The contractor proposed this change as a contractor savings initiative (CSI). Although the CSI required a large amount of temporary falsework, it offered the following advantages:

- It maintained all major design features, criteria, and restrictions in the contract documents.
- It reduced the amount of longitudinal post-tensioning through more-efficient design and replacement of bar tendons with more-efficient strand tendons.
- It eliminated the casting beds, including approximately ninety-five 30-in.-diameter driven steel pipe piles, the casting bed concrete slab, and steel beams.
- It eliminated temporary intermediate pile bents and corresponding lateral bracing.
- It eliminated incremental launching equipment and structures.
- It simplified segment formwork.
- It reduced the number of transverse post-tensioning tendons with modified duct routing to achieve greater efficiency.
- It moved construction staging further from active traffic.
- It will reduce future maintenance requirements by having fewer transverse construction joints and post-tensioning hardware.
- It employed a more commonly used construction method.

AESTHETICS **COMMENTARY**

by Frederick Gottemoeller

The civic scene in Palm Beach, Fla., was influenced by Addison Mizner, Florida's leading architect of the 1920s. His Spanish Mediterranean revival style became the architectural signature of the place, creating the ambience that truly transformed the landscape of South Florida. In other words, he was a hard act to follow. Nevertheless, the designers of the Southern Boulevard Bridge were asked to do just that—and they had to do it in the few feet available between the deck of the highway grade and the crests of the salt waves below. They rose to the challenge with both innovative aesthetic features and innovative engineering.

Examining the aesthetic features from the top down, we see that the overlooks offer comfortable locations to pause, rest, and take in the civic scene, while the railing is transparent to drivers and pedestrians alike. Looking below the roadway, the structural system is clear, simple, and shaped to reflect the forces on it. The soffit is flat and reflective, with no dark recesses to harbor birds or debris. The pier columns have a pilaster on their outer faces that reduces their visual mass and makes them look thinner. The column tops flare to accept the bearings, and that flare continues into the slab to visually spread the bearing reaction into the slab. The column pilasters continue onto the slab as brackets that reinforce the overlooks. Even though there is a clear joint between the piers and the slab, they look as if they were conceived as a single shape.

Getting all of that to work in an extremely thin structure, and then building it over water, required numerous engineering innovations. The designers and contractors are to be congratulated on the ingenuity that resulted.

It's probably a stretch to say that Addison Mizner himself would have been proud of the Southern Boulevard Bridge, but I think he well might have been.

Spans 1 and 2 before concrete placement. Photo: Florida Department of Transportation.

Post-Tensioned Slab

The structure is post-tensioned using 0.6-in.-diameter low-relaxation strands in both longitudinal and transverse directions. The following post-tensioning systems were used:

- Primary longitudinal 29- and 30-strand tendons located in the slab core:
  - The primary tendons in the core perform much of the work of carrying the weight of the cantilever wings to the piers, which are located within the core section.
  - East approach: 10 five-span tendons and two groups of 5 two-span tendons. (The shorter two-span tendons were used to accommodate the casting sequence, described later.)
  - West approach: 13 five-span tendons and two groups of 6 two-span tendons.
- Secondary longitudinal 7-strand tendons located in the slab wings:
  - The secondary tendons in the wings provide the necessary supplemental longitudinal precompression of the wings.
  - East and west approaches: 2 five-span tendons and 2 two-span tendons.
- Four-strand transverse tendons (per five-span approach unit):
  - 128 draped tendons over the full width of the slabs, anchored at the wing tips.
  - 50 straight tendons over the width of the core near the bottom of the slab, anchored at the edges of the core.

Modified Span-by-Span Slab Construction

The slab placement sequence was developed to minimize the amount of falsework and to optimize the longitudinal post-tensioning. Placing the full five-span unit at one time would have been prohibitively expensive because of the extensive amount of falsework required. Therefore, an innovative span-by-span construction scheme was proposed to allow reuse of the falsework and formwork. After the substructure construction was complete, each five-span superstructure unit was constructed in three major stages:

- Stage 1: Construct the two-span unit near the abutment.
- Stage 2: Construct the two-span unit near the bascule pier.
- Stage 3: Construct the center closure span.

The first two stages included only the portion of the total post-tensioning required to support the self-weight of the two-span continuous slabs, and the corresponding temporary construction loads—including staging of construction equipment on the previously constructed unit. After the center closure span was placed, full-length continuity tendons were installed to create the continuous five-span unit. This construction scheme resulted in offsetting moments that significantly reduced the total amount of longitudinal post-tensioning.

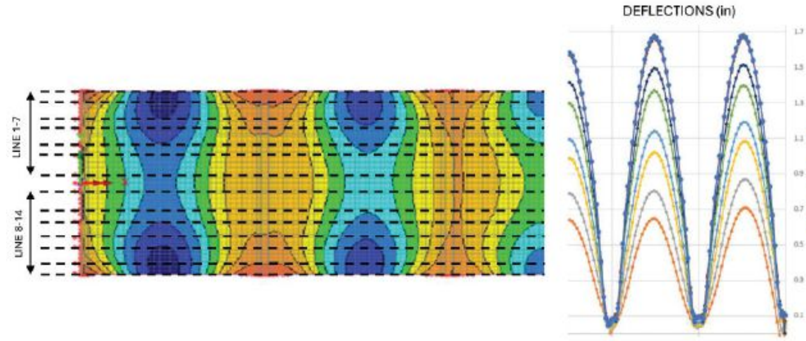
Modular Falsework Design

The falsework consisted of a series of longitudinal steel beams that spanned from pier to pier. W36x262 beams were used for the heavier core sections, and W36x150 beams were used for the lighter slab wings. For the core section, the beams were supported on top of the abutment footings and on HP14x117 columns that were supported on top of the waterline footings at the intermediate piers, anchored to the pier columns for stability. For the cantilevered wings, the beams were supported on steel towers, consisting of HP14x117 columns with cross bracing, supported on driven temporary steel pipe piles. The columns were designed to accommodate the variation in height at each pier with a series of bolted column extensions.

The falsework beams, diaphragms, and formwork system for each span were preassembled and floated into position on a barge. Next, a lifting system on top of the piers, consisting of strongback beams, hydraulic jacks, and threaded bars, was used to lift the spans to their specified heights. The falsework beams were then bolted to the falsework columns. The process was reversed to lower the falsework and float it to the next span to be constructed. Two spans of falsework were used in stages 1 and 2, and one span of falsework was used in stage 3. Each falsework component was used multiple times (between two and six times) during the construction. This modular approach provided significant savings in the cost of the falsework.

Completed spans 1 and 2 support a concrete pump truck used for concrete placement on spans 4 and 5. Photo: Florida Department of Transportation.

View of a typical formwork system supported by the steel falsework. Shoring posts are individually adjusted to provide three-dimensional camber. Photo: Florida Department of Transportation.



Typical slab deflection data from three-dimensional finite-element analysis. Deflection contours for half of the five-span unit are shown on the left. Deflection data for half of the five-span unit are shown on the right. The deflection data are used to set formwork elevations to achieve the final deck profile. Figure: H&H.

Geometry Control

Post-tensioned segmental concrete structures require detailed construction engineering and planning to achieve correct final vertical roadway geometry upon completion. Due to the nearly square superstructure (span-to-width ratio), significant weight, and relatively shallow depth, the slabs used for this project created geometric-control conditions unlike those typically found in segmental concrete box-girder construction, as the structure behavior is three-dimensional. The project team performed three-dimensional structural deformation analysis, including time-dependent concrete effects, longitudinal and transverse post-tensioning force effects, and flexibility of the formwork and falsework supports. The "fresh concrete" stage was modeled using a fictitious low-modulus-of-elasticity material to calculate deflection of the falsework. The "fresh concrete" deflections were superimposed with those of the hardened post-tensioned concrete slab to provide a single set of transverse and longitudinal camber input points.

Each of the east approach spans required 224 adjustable shoring posts supported on top of the falsework beams that support the formwork, while each of the west approach spans required 256 shoring posts. Top elevations of the shoring posts provided camber adjustment points for the formwork. Calculations also included required cambered screed clearances to the top of the top mat of reinforcement at key locations to offset deflections occurring during concrete placement. Top-of-deck elevations were recorded after concrete placement, post-tensioning, and form removal to provide ongoing camber validation and adjustment in the following spans where necessary. The calculations proved sufficiently accurate so that only minor diamond grinding of the roadway was required after completion.

Aesthetics

The use of post-tensioned, cast-in-place concrete permitted the designers to develop a unique custom visual form for this project. The long cantilever wings resulted in a slender fascia and permitted the piers to be tucked in away from the edges, which gives the impression of a lighter and more graceful structure. The shallow slab and smooth underside provide a clean and uncluttered appearance with no dark shadows, which makes the underside seem open and bright, despite the limited height above the water. The slab fully integrates with the piers, providing visual continuity. Tapered capitals below the slab core and cantilever ribs below the slab wings align with the tapered cruciform shape of pier columns and diaphragms between the pier columns. The tapered edges of the thickened slab core harmoniously align with the outside face of the pier columns. The wider west approach spans use the same pier shapes as the narrower east approach spans but with three columns instead of two. The front face of the abutments and back face of the bascule piers include matching pier shapes integrated into the walls. Steel post and beam traffic railings and custom aluminum pedestrian picket railings maximize viewing opportunities of the waterway for motorists and pedestrians. Semicircular overlooks integrated into the cantilever wings at each pier provide respite for pedestrians to take in the natural environment. The octagonal domed control house and trellis structures are inspired by architectural themes throughout the historic town of Palm Beach.

Conclusion

The use of a longitudinally and transversely post-tensioned concrete slab superstructure for the Southern Boulevard Bridge was an innovative solution to address the site-specific geometric constraints. The result is a durable and resilient structure suitable for an aggressive saltwater environment where there is a high risk of significant wave forces from major coastal storms. The CSI for the approach-span superstructure and the supporting specialty engineering were instrumental in the construction of the new landmark drawbridge. The first falsework component was erected in November 2019, the final post-tensioning operation was completed in May 2021, and the bridge was opened to traffic in September 2022.

Reference

1. American Association of State Highway and Transportation Officials (AASHTO). 2008. *AASHTO Guide Specifications for Bridges Vulnerable to Coastal Storms*. Washington, DC: AASHTO.

Rafal Wuttrich is principal bridge engineer and senior associate, and George C. Patton is a chief engineer and principal associate, with H&H in Tampa, Fla.

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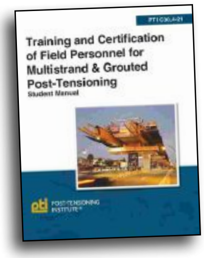

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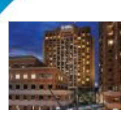

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Interstate 5 over 26th Avenue

A creative solution results in added value for the Oregon Department of Transportation and the public

by Joel Tubbs, David Evans and Associates Inc., and Robert DeVassie, Oregon Department of Transportation

Only a single 56-hour closure was required for replacement of the Interstate 5 bridge over SW 26th Avenue in Portland, Ore. Photo: David Evans and Associates Inc.

After decades of deck performance issues and a recent realization that a seismic retrofit would be cost prohibitive due to the need for full replacement of the timber pile foundations, the Oregon of Department of Transportation (ODOT) determined that the Interstate 5 (I-5) bridge over SW 26th Avenue in Portland, Ore., was reaching the end of its service life. ODOT then partnered with a consultant design team to find an innovative solution that would improve long-term safety for the traveling public. ODOT decided that the best option would be to replace the existing, maintenance-intensive, and seismically insufficient, three-span bridge with a new, single-span, steel-pile-supported structure designed to current static and seismic standards. To mitigate freight mobility concerns and avoid scheduling conflicts with other large infrastructure projects, construction completion was targeted for summer of 2024.

Several factors—limited right-of-way; residential, commercial, and environmental elements; and topographic constraints—would have made it challenging and expensive to use traditional bridge-replacement methods. The original concept included an on-site detour structure and five construction stages. ODOT anticipated that this concept would cost more than $19 million and affect I-5 traffic for two years. The temporary detour alignment would require easements in adjacent parcels and conflict with a critical, existing stormwater treatment facility. Any impact on the performance of that facility would require redesigning it to current federal standards. The staged construction, right-of-way process, and stormwater design and permits of this original concept posed significant cost, schedule, and safety risks for the project.

To determine the best use of public funds, ODOT implemented a value-engineering (VE) process for the project. The VE team recommended a concept that incorporated a new, single-span structure erected under the in-service, existing I-5 bridge, without shifting the alignment and without affecting I-5 traffic. The adopted VE alternative required closing both directions of I-5 for a 56-hour window so the existing superstructure could be demolished, and a new flexible pavement section could be constructed. This solution provided not only cost and schedule savings but was better aligned with ODOT's goals to minimize impacts to the traveling public, adjacent properties, and the environment. Throughout the design process, the design team worked as partners with ODOT staff to develop solutions to the unique and complex issues the project presented.

Replacement Structure

The constructed bridge has a 55-ft span length, is constructed with twenty-eight 30-in.-deep, 48-in.-wide prestressed concrete voided slabs that accommodate a maximum roadway width of approximately 105 ft. The bridge is buried under about 4 to 6 ft of roadway embankment fill and asphalt concrete pavement. ODOT standard prestressed concrete voided slabs were selected because they have a proven

profile

INTERSTATE 5 OVER SW 26TH AVENUE BRIDGE REPLACEMENT / PORTLAND, OREGON

BRIDGE DESIGN ENGINEER: David Evans and Associates Inc., Portland, Ore.

GEOTECHNICAL ENGINEER: Shannon and Wilson Inc., Lake Oswego, Ore.

PRIME CONTRACTOR: HP Civil Inc., Salem, Ore.

CONCRETE SUPPLIER: Wilsonville Concrete Products Inc., Wilsonville, Ore.

PRECASTER: Knife River, Harrisburg, Ore.—a PCI-certified producer

OTHER MATERIAL SUPPLIER: Low-density cellular concrete: The Conco Companies, Concord, Calif.

Because vertical access is limited, workers use specialized, low-overhead equipment to construct micropiles. Photo: Shannon and Wilson Inc.

history of performance and strict quality control standards, and could maximize the clearances across the full width of SW 26th Avenue.

Multipronged Strategy for Success

To successfully realize the benefits of the VE concept for all users and affected parties, the team implemented a combination of key design and construction strategies.

Accelerated Bridge Construction

The project team leveraged accelerated bridge construction methods, such as the use of prefabricated bridge elements, lateral translation technology, and pressure-grouted concrete connections, to facilitate construction of the bridge superstructure without traditional vertical (that is, overhead) construction access. A key element of this strategy was a scheme to perform the staged lateral translation of the prestressed concrete voided slabs to mitigate issues related to space and right-of-way constraints. With only about 50 ft of space available for temporary supports on the southbound side of the existing bridge, a maximum of 10 slabs per stage could be set at one time. For each stage, each slab was placed onto the lateral slide system, which was supported on temporary falsework; then, each slab was connected to the previous slab with tie rods, the keyways between slabs were grouted, and waterproofing membrane was applied. All slabs that had been connected by the end of each stage were moved under the existing bridge to accommodate setting and connecting the next stage of slabs. The first and second stages connected 10 slabs each, and the third stage connected 8 slabs. Therefore, the first translation included only the 10 slabs connected in the first stage, the second translation involved 20 slabs, and the final translation was for all 28 slabs and ended with the slabs in their final positions. The lateral slide system included the jacking beam that directly supported the slabs, rollers that accommodated the translation from pulling on the jacking beam, and a slide track that provided the smooth surface the rollers ran on.

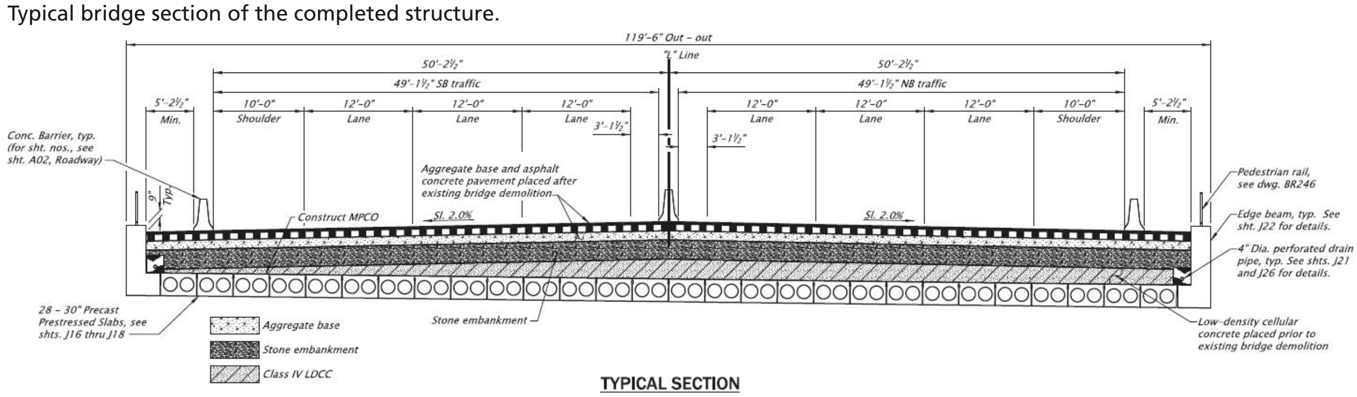
Specialized Equipment

Because of the depth of soft soils at the site, the existing and new bridges were required to be supported on deep foundations. To install deep foundations, crews typically need vertical access for pile driving or drilled shaft equipment. However, with the existing bridge overhead, this type of access was not an option. Additionally, the required horizontal clearance under the new bridge and the presence of existing foundations imposed constraints on the available width of the new bridge foundation. To address these issues, micropiles—built with specialized, low-overhead equipment, anchored into the underlying rock, and with narrow pile caps—were constructed.

Innovative Materials

Low-density cellular concrete (LDCC) was used behind the new abutments as a self-leveling, self-consolidating, retained fill under the existing bridge end spans. To mitigate settlement issues due to the weight of the fill under these end spans, the LDCC and associated

Typical bridge section of the completed structure.



OREGON DEPARTMENT OF TRANSPORTATION, OWNER

BRIDGE DESCRIPTION: 55-ft single-span, prestressed concrete slab bridge

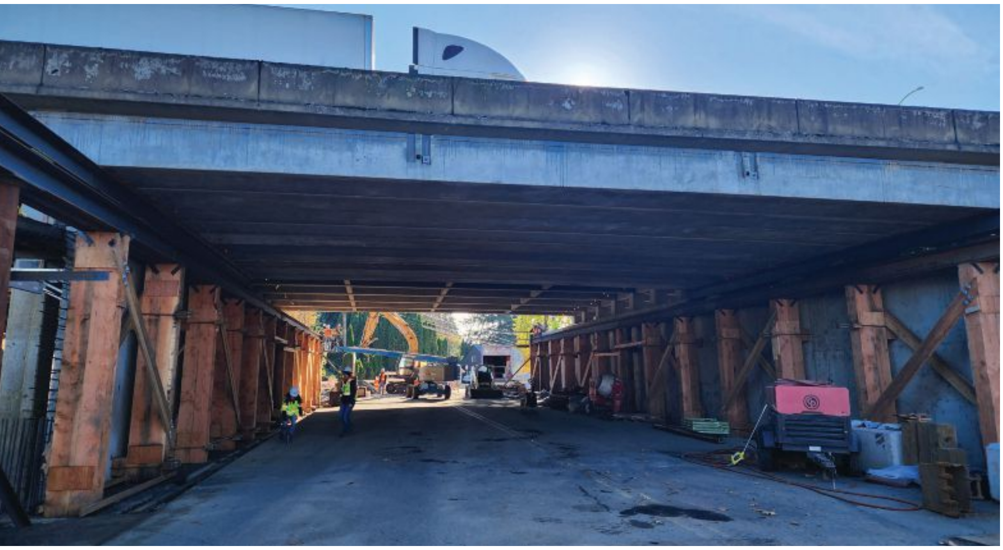
STRUCTURAL COMPONENTS: Twenty-eight 30-in.-deep × 48-in.-wide prestressed concrete voided slabs, cast-in-place concrete portal frame, abutment walls, pile footings

BRIDGE CONSTRUCTION COST: $4.75 million

AWARDS: 2025 PCI Design Award: Bridge with a Main Span Up to 75 Feet; 2025 American Council of Engineering Companies (ACEC) Oregon Project of the Year; 2024 Institute of Transportation Engineers (ITE) Oregon Project of the Year; 2025 ITE Western District Project of the Year; 2025 ACEC National Recognition Award

First group of prestressed concrete slabs are ready for lateral translation. Photo: David Evans and Associates Inc.

Translation of the first stage of precast concrete slabs under the existing bridge. Photo: David Evans and Associates, Inc.



The first stage slabs have been moved under the bridge and the temporary falsework and lateral translation system are ready for setting the second stage of prestressed concrete slabs. Photo: David Evans and Associates Inc.

excavation were designed to achieve a near net-zero overburden pressure on the excavated surface. ODOT had not previously used LDCC as the reinforced backfill for mechanically stabilized earth bridge walls on a project of this scale, so special authorization was required.

Industry Review and Input

To ensure confidence in the feasibility of the design and construction costs, ODOT led a contractor constructability review workshop when about 60% of the design was completed. The workshop included one-on-one sessions with any interested contractor. Ultimately, seven contractors reviewed and commented on in-progress design plans and exhibits. To ensure a fair and transparent bidding process, notes from each of the sessions were posted on ODOT's Electronic Bidding Information Distribution website, which all contractors can view and anyone is free to join.

Sustainability Enhancements

To improve sustainability on the project, the general contractor and ODOT agreed to reuse the demolished bridge waste as roadway fill during the closure. Using the demolished material kept material out of the landfill, reduced costs, and eliminated truck emissions associated with importing fill and transporting waste to the landfill.

Collaborative Closure Planning

Planning for the weekend closure of the full-width of I-5, which was critical to the success of the entire project, was a lengthy and complex process. During the design phase, ODOT and the design team secured buy-in for the full closure from the ODOT-led Mobility Advisory Committee, which includes representatives from the trucking industry, mobile home industry, oversize load freight, automobile users, general contractors, paving contractors, bicycle users, and pedestrians. ODOT initiated early coordination with interested parties two years in advance of the start of construction. Also, during the design phase, the team used risk-based analysis to provide flexibility in the contract to help minimize contractor risk. The execution of the 56-hour weekend closure then required careful planning, hour-by-hour scheduling, backup resources, and extensive coordination among ODOT, the design team, and the contractor.

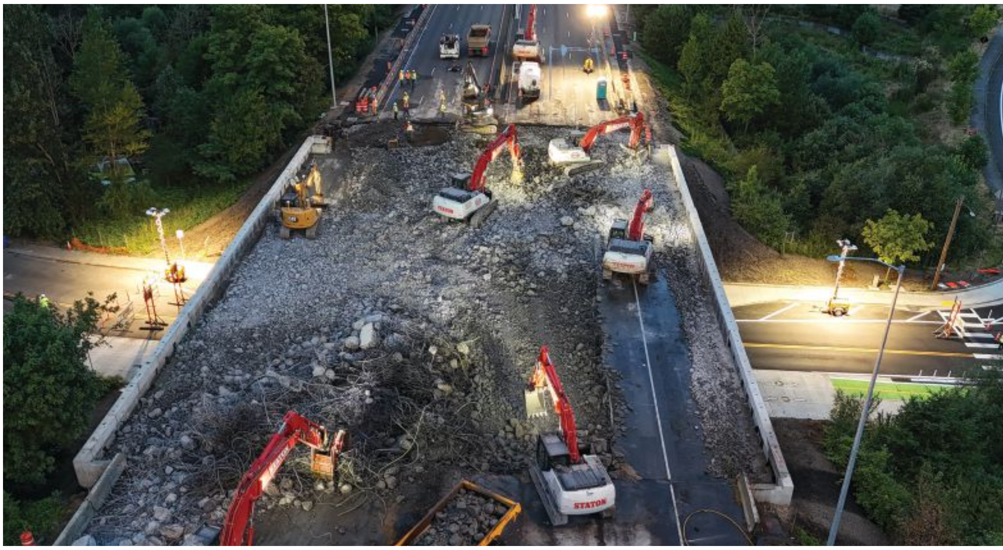
Effective Outreach

The project team's public outreach strategy sought to not only convey the project's cost- and time-saving benefits but also generate awareness of the unique engineering concepts being applied to replace the deteriorating structure. By telling the story of not just the "what" but also the "why" and the "how," the team helped people relate to the project and understand the benefits that engineering can bring to their communities. Social media was key for communicating with audiences who are usually not reached by typical advertising and outreach methods, and it also enabled live updates during the closure.

Low-density cellular concrete (LDCC) backfill is placed behind the new abutments. LDCC backfill was used to achieve a near net-zero overburden pressure on the excavated surface. Photo: David Evans and Associates Inc.

Balanced Multimodal Impacts

The project's impact on local vehicular, bicycle, and pedestrian traffic on SW 26th Avenue was a crucial consideration. The local vehicular detour was relatively short, but it was still too long for bicycle and pedestrian traffic. Also, the Portland Bureau of Transportation (PBOT) was ready to construct a multiuse path between neighborhoods on either side of the bridge, and it was important to ensure that the bridge project would not unnecessarily impede pedestrian and bicycle traffic along that path. Fortunately, the project team was able to maintain bike and pedestrian access except during the weekend closure.

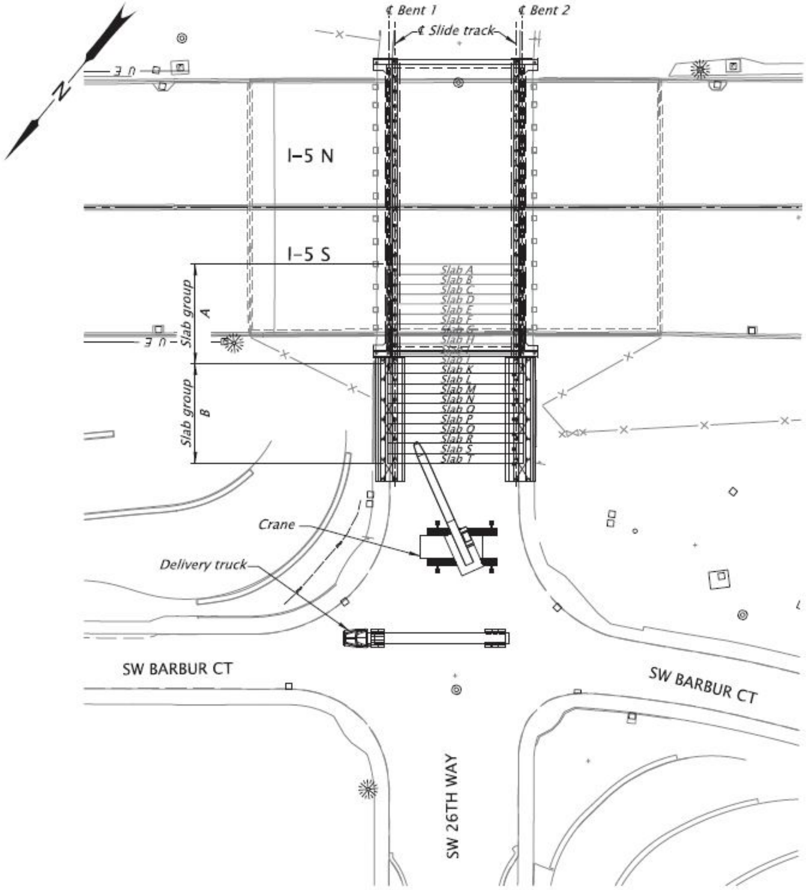


Demolition of the existing superstructure during the weekend closure. The debris was used as roadway fill, which kept the material out of the landfill. Photo: Oregon Department of Transportation.

Efficient Contracting

The design of PBOT's multiuse path under the bridge and through the adjacent intersection with SW Barbur Court included enhanced striping, bicycle lanes, and accessible curb ramps. ODOT, PBOT, and the design team worked together to optimize multimodal connectivity along SW 26th Avenue under I-5 in the following ways: coordinating design details between the two design projects, implementing temporary bicycle and pedestrian routes that tied into PBOT's neighborhood improvements, and incorporating the construction of the multiuse path into the project's ODOT contract. This streamlined approach avoided a lengthy permitting process for both agencies, eliminated potential interface risks associated with multiple contractors working in the same area, and removed the potential need to reconstruct PBOT-completed elements on SW 26th Avenue

Aerial view of the completed structure showing the multimodal improvements. Photo: Oregon Department of Transportation.



SLAB ERECTION - STEP 2

The 10 slab beams of the second stage are placed on the lateral translation system after the first stage beams have been moved under the existing bridge.

after the bridge construction was complete.

Conclusion

The combination of design and construction strategies helped solve several complex issues surrounding the project. The closure was successful, finishing two hours early with no reported incidents on the detour routes. Compared with the original concept, the project eliminated two years of work-zone impacts to more than 100,000 commuting, freight, and transit vehicles per day on I-5. Because bridge construction occurred completely on SW 26th Avenue and bridge demolition and highway construction happened during the weekend I-5 closure, freeway traffic never needed to travel through an active I-5 work-zone.

The success of the project and the lessons learned during implementation demonstrate that variations of this concept could be readily scaled to solve different problems. Local agencies with limited funds could use this concept to save on costs for smaller scale, but still expensive bridge replacements, or to rapidly perform emergency repairs.

The I-5 over SW 26th Avenue bridge replacement project required a collaborative team that could work together to adapt new techniques and applications, address high-risk elements, and ultimately provide a project that maximizes the return on investment for taxpayers. ODOT and the design team collaborated to find solutions through frequent meetings between agency and consultant subject matter experts, including experts in structures, materials, geotechnical, traffic, and risk management, leading to a final product that accomplished the original project goals with added value. The bridge design was delivered approximately 20% under budget and was constructed by its target completion date of summer of 2024. The team achieved nearly 30% savings relative to the original estimate based on traditional bridge construction.

Joel Tubbs is a senior bridge engineer and project manager for David Evans and Associates, Inc., in Portland, Ore. Robert DeVassie is a resident engineer and project manager for the Oregon Department of Transportation in Portland.

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.erikssonsoftware.com/sync
The Focus article on page 8 is about Eriksson Technologies and discusses the use of Eriksson Software's Sync technology for the Teterboro Airport air traffic control tower project. This is a link to an overview of Eriksson Sync—an innovative technology that provides a critical two-way interface between engineering design software and building information modeling models.

https://www.gcrtoledo.com/build
https://www.dot.state.oh.us/OTEC/Documents/2024OTECPresentations/48_QuagliataWagner.pdf
The first link leads to a website that provides information about the Glass City Riverwalk project to revitalize a section of riverfront in Toledo, Ohio. The second link leads to a presentation about the project from the 2024 Ohio Transportation Engineering Conference. The Project article on page 16 is about the new Bend Bridge, an integral part of the project that provides pedestrian access from the Glass City Riverwalk to the Martin Luther King Jr. Bridge, linking both sides of the Maumee River.

https://www.southernblvdbridge.com
The Florida Department of Transportation website for the Southern Boulevard Bridge can be found at this link. The Project article on page 20 discusses the design and construction of the continuous, cast-in-place, longitudinally and transversely post-tensioned slabs that form the approach spans for this bridge replacement project connecting West Palm Beach, Fla., across Lake Worth Lagoon and the Atlantic Intracoastal Waterway, to the barrier island town of Palm Beach.

https://www.youtube.com/watch?v=1k6JdbGju3U
The project to replace the Interstate 5 bridge over SW 26th Avenue in Portland, Ore., used an innovative structure type and construction method to minimize disruptions to the traveling public and enhance safety. The Project article on page 26 describes how the value-engineering concept for the bridge incorporated a new, single-span, buried structure under the in-service, existing bridge, without shifting the alignment and without affecting interstate traffic. This is a link to a time-lapse video of the weekend closure that brought the replacement structure in service.

https://ctr.utexas.edu/wp-content/uploads/pubs/0_5253_1.pdf
https://library.ctr.utexas.edu/ctr-publications/0-6416-1.pdf
The Concrete Bridge Technology article on page 35 discusses shear cracking and presents tools for informed decision-making regarding the capacity of concrete bent caps. These two links lead to reports about the underlying research. The reports also include the tools described in the article for estimating component capacity based on crack width and reinforcement ratios.

https://www.epoxyinterestgroup.org/quality-certification
This is a link to the Epoxy Interest Group's (EIG's) webpage about the Concrete Reinforcing Steel Institute's Epoxy-Coating Plant Certification and the Certification for Fabrication of Epoxy-Coated Reinforcement. The NCBC Member Spotlight article on page 44 highlights these programs in relation to EIG's commitment to improve the quality and performance of epoxy-coated reinforcement.

https://www.fhwa.dot.gov/resourcecenter/teams/structures-geotechnical-hydraulics/Structural_Design_UHPC_Workshop_Manual.pdf

A concrete bridge technology guidance document on the selective uses of ultra-high-performance concrete (UHPC) to enhance durability and extend the service lives of concrete bridges can be found at the link above. The Federal Highway Administration's *Structural Design with UHPC Workshop Manual* is available at this link.

https://doi.org/10.17226/27029
https://onlinepubs.trb.org/onlinepubs/webinars/230920.pdf
The Concrete Bridge Technology article on page 32 summarizes the key findings and recommendations from National Cooperative Highway Research Program (NCHRP) Research Report 1026, *Guidelines for Adjacent Precast Concrete Box Beam Bridge Systems*, which can be downloaded using the first link. The report discusses shear key solutions for adjacent prestressed concrete box-beam bridges that can enhance the durability and service life of these structures. The slides from a webinar on the same topic are available at the second link.

Guidelines for Adjacent Precast Concrete Box-Beam Bridge Systems: Addressing Performance of Shear Keys

by Dr. Abdullah Haroon, University of Minnesota – Duluth

Adjacent precast concrete box-beam bridge systems offer an efficient solution for short-span bridges, particularly in cases where vertical clearance is a concern. These systems rely on load transfer between adjacent beams through grouted shear keys. However, a persistent challenge has been the cracking and subsequent leakage of these shear keys, which can lead to the corrosion of prestressing strands and reinforcing steel, ultimately compromising the bridge's structural integrity and service life. The National Cooperative Highway Research Program (NCHRP) Research Report 1026, *Guidelines for Adjacent Precast Concrete Box Beam Bridge Systems*,[1] provides a comprehensive investigation into this problem and proposes revised design and construction guidelines to enhance the performance of connections and extend bridge service lives. This article summarizes the key findings and recommendations of NCHRP Research Report 1026, offering valuable insights for structural engineers and transportation agencies.

Understanding Shear-Key Cracking

Previous research, summarized in the NCHRP Report, has shown that thermal movements are the primary cause of the cracking in shear-key joints. Live loads do not typically initiate cracks in intact shear keys, but they can cause existing temperature-induced cracks to propagate. The thermal gradient within the concrete beams, which is particularly pronounced over the top 4 in. due to solar heating, induces expansion-and-contraction cycles. Shear keys are often placed during the day when the beams are in an expanded state. After placement, as the temperature drops at night, the beams contract and tensile stresses develop within the shear-key material and at the interface with the beams. The literature review in the NCHRP report also indicates that cracking often occurs within the first few weeks after placing grout in the shear keys. The cracks initiate near the ends of the beams, possibly due to the restraint in the transverse direction offered by bearing pads.

The Research Approach

To investigate complexities of shear-key performance, the research team employed a twofold approach: detailed analytical modeling using finite element analysis (FEA), followed by full-scale testing.

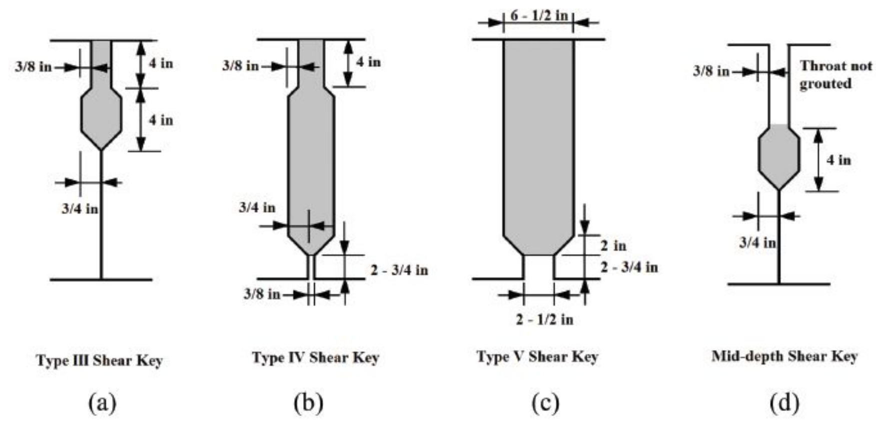
Analytical Modeling

The researchers developed three-dimensional FEA models to simulate the behavior of adjacent box-beam bridges under various loading and environmental conditions. These models incorporated various shear-key configurations, including the commonly used partial-depth Type III, the proposed full-depth Types IV and V, and a mid-depth key (**Fig. 1** and **2**). Other variables considered were span lengths, beam depths, skew angles, and deck types. The analysis assessed the stresses within the shear keys due to temperature changes, live loads (HL-93 truck loading), and lateral post-tensioning forces (varying from 93 to 106 ksi). The effect of reinforcement in shear keys was also investigated. Neither post-tensioning nor reinforcement in the shear keys contributed significantly to crack mitigation; therefore, these aspects of the study are not discussed in detail in this article. Detailed findings of the analytical investigation can be found in the full NCHRP research report.

Full-Scale Testing

Two full-scale bridge sections, each consisting of three beams and two shear-key joints, were constructed and tested in the laboratory (**Fig. 3**). These tests evaluated the performance of a narrow, full-depth (Type IV) shear key filled with either a standard nonshrink grout or a high-bond grout, and a wide, full-depth (Type V) shear key filled with small-aggregate concrete. The beams were preheated to simulate field temperature gradients before the shear keys were cast. The specimens were then subjected to 30 cycles of temperature

Figure 1. Shear-key configurations: typical shear key (a), and proposed shear-key configurations (b–d). All Photos and Figures from Miller et al.[1]



variation and 100,000 cycles of live-load application. Dye-penetration tests were conducted to assess leakage at various stages. Additionally, ASTM C1583[2] pull-off tests were performed to evaluate the bond strength between various shear-key materials and different beam surface preparations (smooth, exposed aggregate, sandblasted) under both dry and prewetted conditions.

Key Findings and Their Implications

The research yielded several significant findings that have direct implications for the design and construction of adjacent precast concrete box-beam bridge systems:

- **Temperature stresses dominate.** The FEA results demonstrated that temperature-induced stresses are significantly larger than those caused by live loads, confirming that temperature fluctuations are the primary driver of shear-key cracking.
- **Deeper shear keys perform better.** In both the analytical modeling and the full-scale testing, deeper shear keys—especially those that were full depth—performed better than traditional partial-depth shear keys. Full-depth shear keys provide a larger bonded area between the shear key material and the beam, and the FEA suggests that compressive stresses can develop at the bottom of these keys, potentially preventing full-depth crack propagation and leakage.
- **An "ungrouted top" helps prevent leakage.** Measurements of the thermal gradients in the beams, taken while the beams were stored in the fabricator's yard, confirmed that the thermal gradient shown in Article 3.12.3 of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[3] is reasonably accurate for box beams. There is a considerable temperature gradient over the top 4 in. of the beam when the top surface is heated by the sun but the face is shaded. A crucial finding was that stress in the area most susceptible to thermal gradients was significantly reduced when the top 4 in. of the shear key was not filled with grout. This approach, combined with a deeper shear key, moves the grouted area away from the region of highest thermal stress, and that reduces the overall tensile stress in the key and mitigates cracking. Full-scale tests confirmed the effectiveness of this detail in preventing leakage.
- **Bond plays a critical role.** The research demonstrated the importance of achieving a strong bond between the shear-key material and the beam

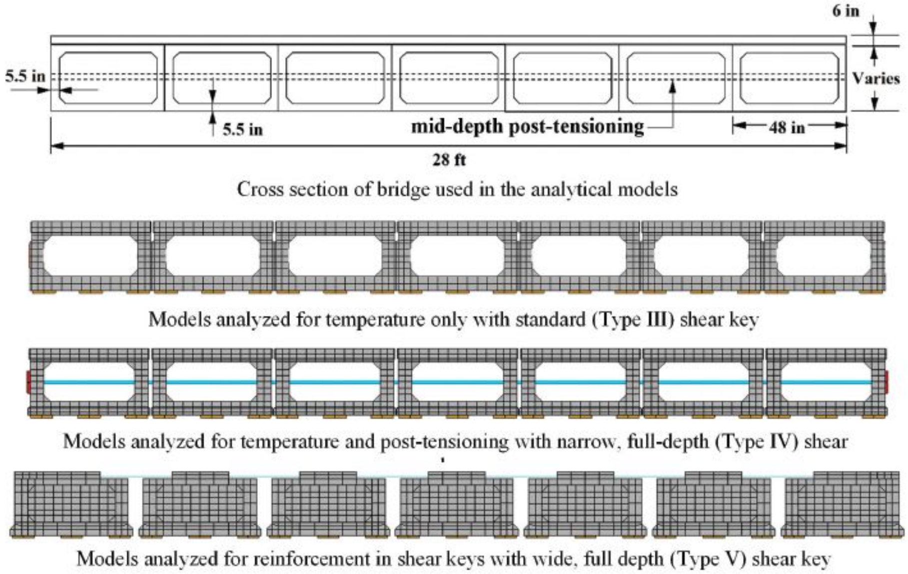


Figure 2. Cross sections of the adjacent precast concrete box-beam bridge models subjected to analytical testing.

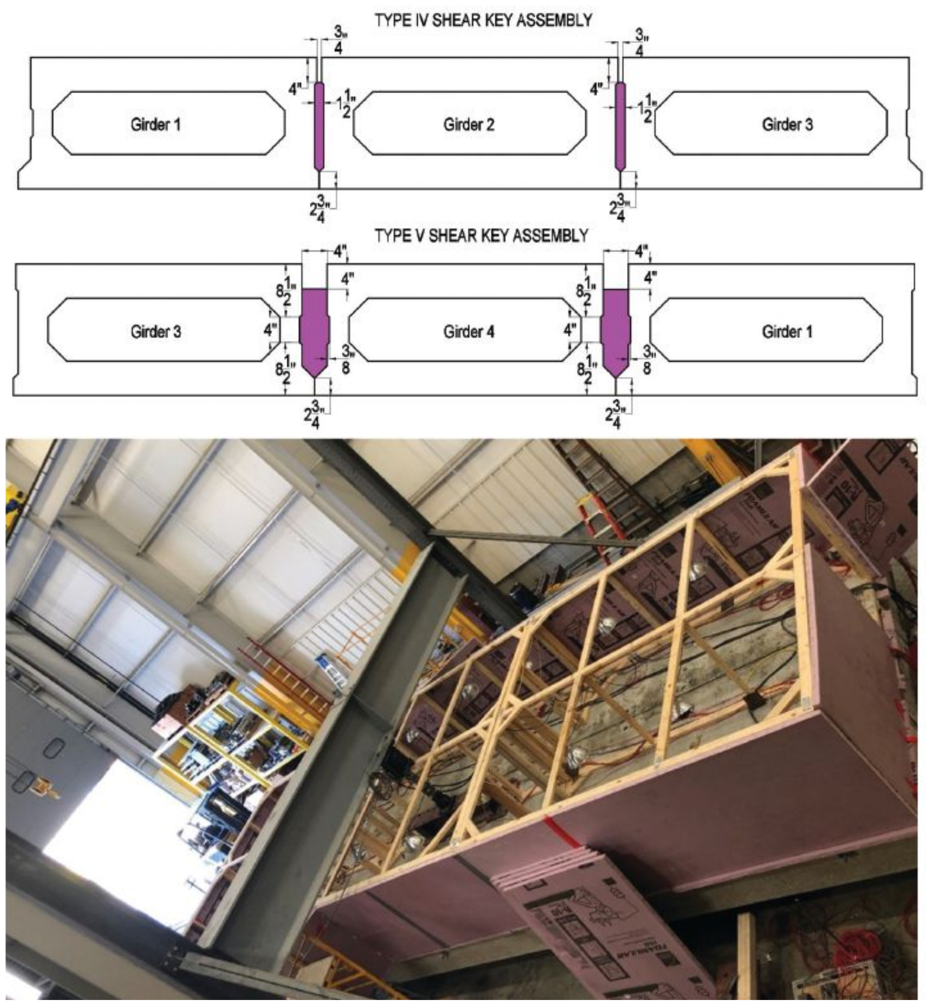


Figure 3. Test cross sections (top), and testing setup showing details of thermal loading (bottom). An insulated box with high-intensity heaters and heat lamps generated heat over a three-girder assembly. Thermocouples and vibrating wire strain gages were used to monitor thermal gradient and strain across shear keys.

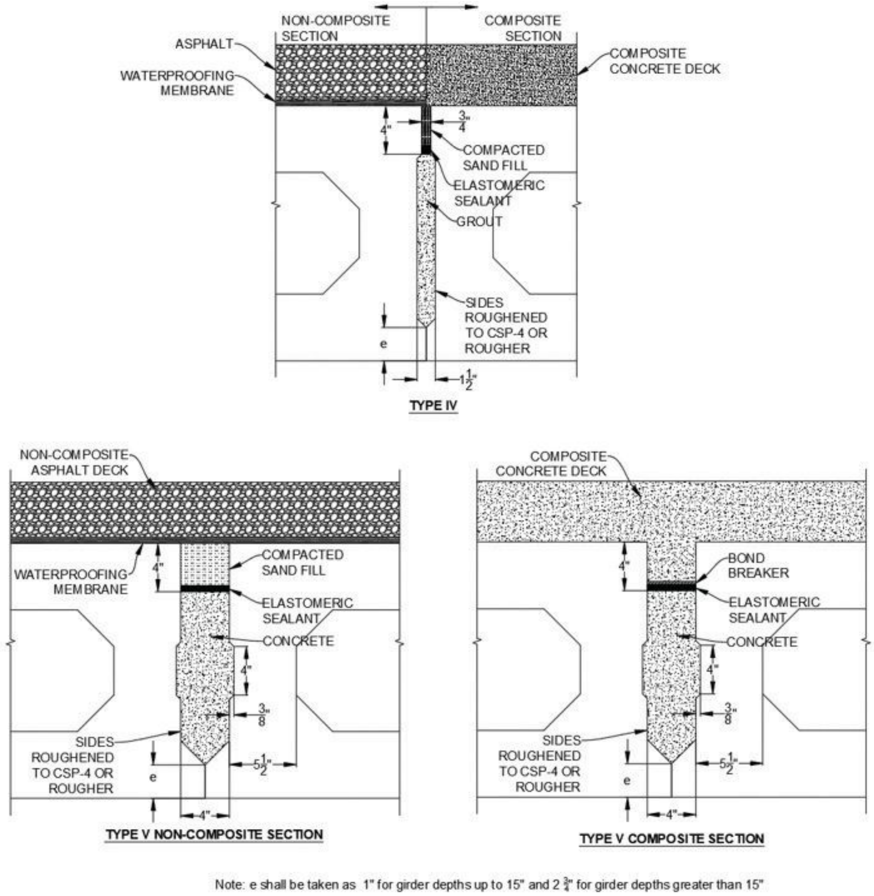


Figure 4. Shear-key configurations proposed as a result of analytical and experimental testing.

surface. Pull-off testing revealed that roughening the beam surface through sandblasting to a concrete surface profile (CSP) of at least 4 (as defined by the International Concrete Repair Institute[4]) or using an exposed-aggregate surface significantly enhances bond strength. Prewetting the beam surfaces before the shear-key material is placed also demonstrably improves bond. A minimum bond strength of 200 psi is recommended.

- **Material selection is key.** The full-scale tests highlighted the superior performance of a high-bond-strength, nonshrink grout in preventing leakage and achieving excellent bond. While a standard nonshrink grout and small-aggregate concrete also performed acceptably, the importance of using materials with high-bond properties and nonshrink characteristics is evident.

Recommendations for Implementation

Based on the comprehensive research findings, NCHRP Report 1026 proposes several key recommendations for the design and construction of adjacent precast concrete box-beam bridge systems:

- Adopt the deepest possible shear key and intentionally leave the top 4 in. ungrouted. This strategy maximizes the bonded area and minimizes stress concentrations in the critical thermal gradient zone. **Figure 4** presents proposed details for Type IV and Type V shear keys that incorporate this feature.
- Ensure that the beam surfaces that interface with the shear-key material are roughened to achieve a moderately roughened surface greater than or equal to CSP-4. Exposed-aggregate surfaces are also highly recommended, especially when the fill material is concrete. Prewetting the beam surface before the shear key material is placed also improves bond.
- Prioritize the use of high-bond-strength, nonshrink grout for filling shear keys. If concrete is used, incorporating a nonshrink additive is strongly advised. A minimum average bond strength of 200 psi—as demonstrated by ASTM C1583 pull-off testing—is suggested as a benchmark for acceptable performance.

Conclusion

NCHRP Research Report 1026 is a valuable contribution to the body of knowledge on adjacent precast concrete box-beam bridge systems. By investigating the causes of shear-key cracking and leakage through a combination of analytical modeling and full-scale testing, the research team developed evidence-based guidelines for improved design and construction practices. With an emphasis on deeper shear keys with ungrouted top regions, proper surface preparation, high-quality bonding materials, and the rational assessment of bond strength, the guidelines provide a clear path toward enhancing the durability and service life of these bridge systems. The findings strongly suggest that implementing these recommendations will substantially reduce shear-key cracking and leakage, ultimately contributing to a more durable bridge infrastructure.

References

1. Miller, R., B. Shahrooz, A. Haroon, E. Steinberg, W. Hamid, A. Chlosta, and C. Slyh. 2023. *Guidelines for Adjacent Precast Concrete Box Beam Bridge Systems*. National Cooperative Highway Research Program (NCHRP) Research Report 1026. Washington, DC: National Academies Press. https://doi.org/10.17226/27029.
2. ASTM International. 2020 *Standard Test Method for Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direct Tension (Pull-Off Method).* ASTM C1583/C1583M-20. West Conshohocken, PA: ASTM International.
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Acknowledgments

The author would like to acknowledge the other authors of the NCHRP report: Richard Miller, Bahram Shahrooz, Eric Steinberg, Waleed Hamid, Alexander Chlosta, and Caleb Slyh.

A Crack Is Not a Crack: Shear Cracking

by Dr. Oguzhan Bayrak, University of Texas at Austin

As part of the ongoing *ASPIRE*® series on cracking, this article focuses on shear cracking that may occur in conventionally reinforced concrete members in service. To explore the topic of shear cracking in this article, we will set the context first, establish the guiding principle in engineering mechanics next, and finally provide useful tools from research projects on the subject of shear cracking.

Context

Shear strength of concrete members has been a topic of intense research, discussion, technical debate, and design code and specification development for quite some time. The large number of variables that influence the shear strength of concrete members formed the initial basis of research in the late 19th and early 20th centuries. Some explanations of load transfer from the points of application of loads into the supports involved early versions of strut-and-tie models for structural components.[1,2] As research and development efforts continued around the globe, reinforced and prestressed concrete member designs in the United States for most of the 20th century were based on developing shear force and bending moment diagrams, and designing all sections along the length of structural members by using the demands imposed. Even today, this approach—the sectional design of concrete components—forms the basis for a great majority of our designs in "B regions" (that is, beam regions or Bernoulli regions).

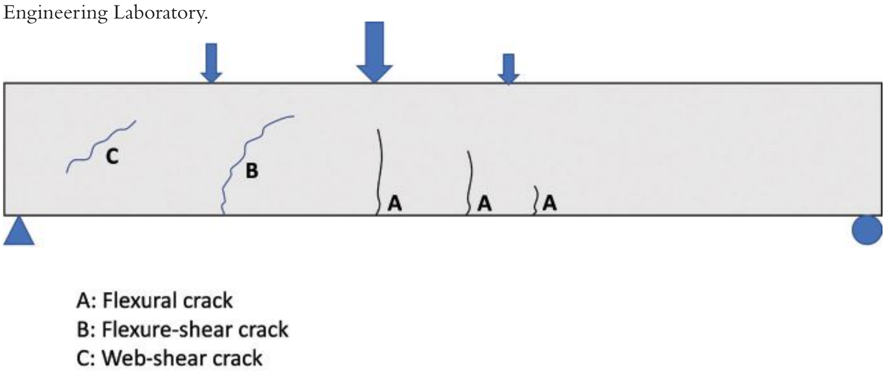
Sectional design of concrete components involves evaluating the demands imposed by a variety of load combinations on each section along the length of a structural component, determining the capacity, and confirming that the capacity of each section is greater than the demands. In this design approach, we use load factors and strength-reduction factors to achieve a target reliability factor that is consistent with the calibration of the applicable design provisions. For cases in which we do not have prestressing force to improve the shear resistance, the nominal shear capacity of a concrete component is the sum of the concrete contribution to shear strength and the stirrup (shear reinforcement) contribution to shear strength. When a prestressing force is present, we should account for the vertical component of the prestressing force, in accordance with design specifications. However, to simplify the discussion, only conventionally reinforced members are considered in this article.

At this point, it is useful to discuss the historical development of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[3] in general terms to further develop the context. Before the first edition of the AASHTO LRFD specifications was introduced in 1994, AASHTO's *Standard Specifications for Highway Bridges*[4] closely followed provisions for shear design that are still part of the American Concrete Institute's *Building Code Requirements for Structural Concrete (ACI 318-25) and Commentary (ACI 318R-25)*.[5] With the introduction of the LRFD version of the bridge design specifications, AASHTO adopted a sectional shear design approach that is based on the modified compression field theory (MCFT) originally developed at the University of Toronto. Thus, the current inventory of bridges in the United States contains components that were designed using different approaches. This context serves as a backdrop as we discuss the mechanics of shear resistance in greater detail.

Structural Behavior

To discuss the shear cracking and shear resistance of a typical concrete component, let us consider a beam that is being loaded to levels that exceed typical service loads. More specifically, let us direct our attention to the beam depicted in **Fig. 1** as it is gradually loaded to failure. As the externally applied loads increase, we expect to see the formation of flexural cracks (cracks marked as "A" in Fig. 1). These cracks form when the longitudinal stresses reach the tensile strength of concrete. The typical flexure-shear crack (shown in Fig. 1 as crack "B") forms when a flexural crack that

Figure 1. Cracks in a typical reinforced concrete beam. All Photos and Figures: Ferguson Structural Engineering Laboratory.



originally formed when longitudinal tensile stresses reached the tensile strength of concrete turns into a shear crack by changing its inclination due to the presence of shear stresses that change the direction of maximum principal tensile stress. It is important to note that for a flexure-shear crack to form, a flexural crack must form first. A web-shear crack (crack "C" in Fig. 1) forms when the diagonal tensile stress reaches the tensile strength of concrete without penetrating the flexural compression zone or the tensile side of the beam. Such cracks can form in members with thin webs or where large, concentrated forces are applied near the supports. In all cases, the formation of cracks relates to the tensile strength of concrete.

In cases where an adequate quantity of shear reinforcement is present, the initial formation of shear cracks (types B and C) does not signal imminent shear failure of the member. Additional shear cracks are typically needed to push the beam in Fig. 1 toward failure.

As mentioned previously, early versions of shear design provisions in the AASHTO standard specifications mirrored the provisions in ACI 318. Those provisions were predicated on the fact that the shear strength of concrete V_c in a typical beam could be determined at the formation of the initial shear crack (either flexure-shear or web-shear). While the shear transfer mechanism changes after the formation of a shear crack, the design provisions were developed based on the assumption that V_c does not change in magnitude as loading increases. From a behavior standpoint, well after the formation of the initial shear crack, near the ultimate load-carrying capacity, the concrete contribution to shear strength involves the cumulative contributions of shear carried in the flexural compression zone, shear transferred across the diagonal cracks due to aggregate interlock, and flexural tension reinforcement serving as dowels to bridge the cracks. Collectively, these three mechanisms were considered the "concrete contribution to shear strength" and added to the stirrup contribution. However, we gained insight over the years from additional structural tests and came to understand that the tensile strength of concrete is influenced by the component size. Therefore, we realized that the simplified approach that has been used in shear design for decades could be unconservative in some cases because it did not account for the so-called size effect. In 1994, the AASHTO LRFD specifications adopted shear design provisions based on the MCFT, where V_c is attributed to the ability of cracked concrete to transfer shear stresses. With the adoption of MCFT, we no longer view the formation of the initial shear cracks as the "shear strength of concrete"—a perspective shift that represents an important and significant change in bridge design philosophy.

Field Performance of Substructure Components

The formation of shear cracks in the existing inventory of bridges is not uncommon. **Figure 2** shows a straddle bent cap with a rectangular cross section with shear cracks. **Figure 3** shows a variable-depth bent cap that is supported on multiple columns, and **Fig. 4** shows an inverted-tee straddle bent cap. The size of the cracks in Figs. 2–4 have been enhanced for visibility. Shear cracks that formed in service conditions such as those seen in Fig. 2, 3, and 4 served as the reasons for a series of research projects

Figure 2. Straddle bent with a rectangular cross-section and shear cracks. Cracks have been enhanced. Source: Adapted from Birrcher et al.[6]

Figure 3. Multicolumn bent with both shear and shear-flexure cracks. Cracks have been enhanced. Source: Adapted from Birrcher et al.[6]

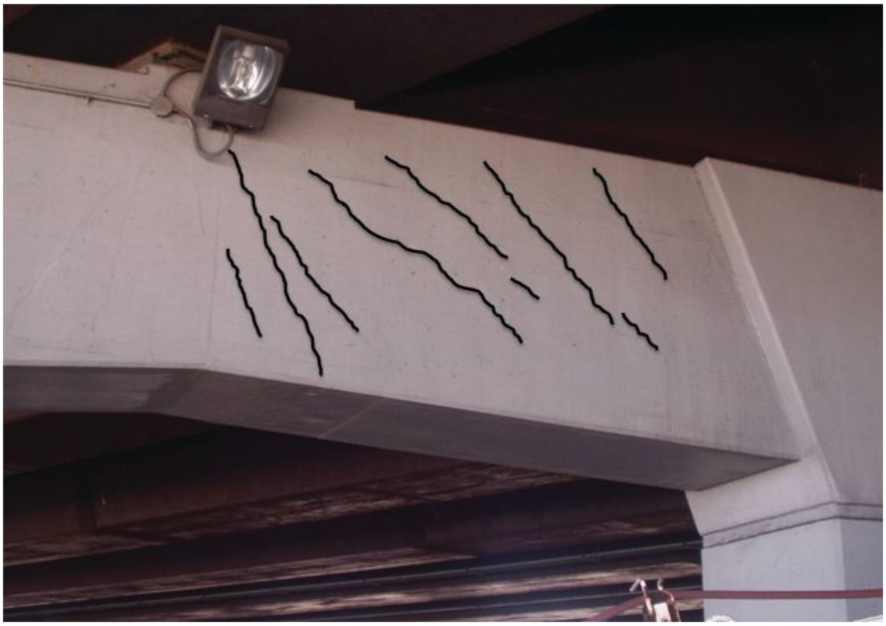


Figure 4. Straddle bent with an inverted-tee cross section has shear cracks. Cracks have been enhanced. Source: Adapted from Larson et al.[7]

conducted at the Phil M. Ferguson Structural Engineering Laboratory.[6,7] While there are many aspects of these comprehensive research efforts that inform our understanding of the behavior of members subjected to shear-critical loads, I will focus here on the field evaluation component.

Rectangular Bent Caps

Because crack widths can be influenced by the specimen size, the investigators at Ferguson Laboratory conducted large-scale component tests and collected extensive data about the widths of diagonal cracks. Rigorous analysis of the data led to the development of a simple chart that can be used to evaluate the residual capacity in diagonally cracked, conventionally reinforced bent caps with rectangular cross sections; that chart is summarized in **Table 1**. It is important to note that the research team has also identified parameters of secondary nature that contribute to the inherent variability of the estimates summarized in the table.

An examination of Table 1 leads to the following observations:

1. Moving down vertically in the table, for a given diagonal crack width, we can see that a rectangular cap is closer to its design shear capacity if it contains a greater quantity of

Table 1. Diagonal crack width–to–capacity relationship for rectangular bent caps.[6]

Load on the Member, Quantified as a Percent of Ultimate Capacity on Average (± scatter)						
Reinforcement / w_{max} (in.)	0.01	0.02	0.03	0.04	0.05	0.06
$\rho_v = 0.002$ $\rho_h = 0.002$	20 (+10)	30 (±10)	40 (±10)	50 (±10)	60 (±15)	70 (±15)
$\rho_v = 0.003$ $\rho_h = 0.003$	25 (±10)	40 (±10)	55 (±10)	70 (±10)	80 (±10)	90 (±10)
$\rho_v > 0.003$ $\rho_h > 0.003$	30 (±10)	50 (±10)	70 (±10)	85 (±10)	~ Ultimate	~ Ultimate

Notation:
w_{max} = maximum measured diagonal crack width (in.)
ρ_v = reinforcement ratio in vertical direction ($\rho_v = A_v / bs_v$)
ρ_h = reinforcement ratio in horizontal direction ($\rho_h = A_h / bs_h$)
A_v & A_h = total area of stirrups or horizontal bars in one spacing (in.^{2})
s_v & s_h = spacing of stirrups or horizontal bars (in.)
b = width of web (in.)

Directions:
1). Determine ρ_v and ρ_h for bent cap
2). Measure maximum diagonal crack width, w_{max}, in inches
3). Use chart with w_{max}, ρ_v, and ρ_h to estimate % of capacity

Important Notes:
In this chart, the maximum width of the primary diagonal crack in a shear-critical member is linked to the load on the member, quantified as a percent of its ultimate capacity. The intent of this chart is to aide field engineers in evaluating residual capacity in diagonally-cracked, reinforced-concrete bent caps subjected to concentrated loads at a/d ratios between 1.0 and 2.0. This chart was developed from crack width data from 21 tests of simply-supported reinforced concrete beams with overall heights between 42" and 75". The testing was conducted at an a/d ratio of 1.85. Data has shown that diagonal crack widths may slightly decrease with decreasing a/d ratio. The same crack width at a smaller a/d ratio indicates that a higher percentage of capacity from the above chart has already been reached.

This chart should be used in conjunction with sound engineering judgement with consideration of the following limitations:
-variability in crack widths in general (± scatter)
-members loaded at a/d < 1.85 may be at slightly higher % of capacity
-differences between field and laboratory conditions
-implications of an unconservative estimate of capacity

This chart is not intended to be used for inverted-tee bent caps.

crack-control reinforcement. For example, for a diagonal crack width of 0.03 in., 70% (±10%) of the cap's capacity would be exploited by the loads acting on the cap if both the crack-control reinforcement ratio in the horizontal direction ρ_h (skin reinforcement) and the crack-control reinforcement ratio in the vertical direction ρ_v (stirrups) were greater than 0.3%. The same crack width would signal a 40% (±10%) utilization of the ultimate capacity if the crack-control reinforcement were 0.2% in both the horizontal and vertical directions. This makes sense: the greater the amount of crack control reinforcement, the larger the load needed to open the crack.

2. For a given crack-control reinforcement ratio, as the diagonal crack widths increase, the rectangular bent cap comes closer to using its ultimate capacity. Let's take the 0.3% crack-control reinforcement ratio as an example. A diagonal crack of 0.010 in. points to a 25% (±10%) usage of the total capacity, whereas a diagonal crack width of 0.040 in. indicates that the component is loaded to 70% (±10%) of its ultimate shear capacity.

As we can see from the examples discussed, the chart presented in Table 1 is a useful decision-making tool for the inspection of a rectangular bent cap with diagonal cracks. Importantly, interpolation among various entries in the table is possible.

Inverted-Tee Bent Caps

The chart presented in Table 1 is intended for rectangular bent caps, and its application to inverted-tee bent caps is not advisable, as will be illustrated. To develop an understanding of the behavior of inverted-tee bent caps, the research team at Ferguson Laboratory tested a series of inverted-tee bent caps.[7]

At the conclusion of their comprehensive testing on inverted-tee caps, the team had a wealth of data about the formation and opening of shear cracks throughout the loading regimen to which the caps were subjected. The insights gained in the testing served a variety of objectives, and interested readers are advised to read the full research report.

In their investigation of the field performance of inverted-tee bent caps, the researchers followed a similar format to that used in the study of rectangular bent caps. **Table 2** summarizes the recommendations of the researchers. Importantly, the trends discussed previously regarding the observations on Table 1 for rectangular bent caps remain valid for inverted tees, although the percentage of the available capacity that is exploited differs. For example, for a diagonal crack width of 0.04 in. in an inverted-tee cap reinforced with a crack reinforcement ratio of 0.3% in each direction, we can see that the cap is loaded to 75% (±15%) of its capacity, as opposed to 70% (±10%) for a similar rectangular cap, as discussed earlier. This difference in the percentages can be attributed to tension-chord loading (inverted-tee section) being more "punishing" on a cap than compression-chord loading (rectangular section) would be. Conversely, we can see that the difference between 70% and 75% is small in relation to data scatter observed in the tests (±10% for rectangular caps and ±15% for inverted-tee caps). Interestingly, while the application of loads in the ledges of an inverted-tee cap adds a tension field as hanger reinforcement works to hang the load up to the compression chord, the ability of a diagonal crack to transfer stresses across the crack seems to be at a similar level of maturity for a given diagonal crack width.

Table 2. Relationship between diagonal crack width and capacity for inverted-tee bent caps.[7]

Load on the Member, Quantified as a Percent of Ultimate Capacity on Average (± scatter)

Reinforcement \ w_{max} (in.)	0.01	0.02	0.03	0.04	0.05	0.06
$\rho_v = 0.003$ $\rho_h = 0.003$	30 (± 10)	50 (± 15)	65 (± 15)	75 (± 15)	80 (± 15)	90 (± 10)
$\rho_v = 0.006$ $\rho_h = 0.006$	40 (± 10)	65 (± 10)	85 (± 10)	~ Ultimate	~ Ultimate	~ Ultimate

Notation:
w_{max} = maximum measured diagonal crack width (in.)
ρ_v = reinforcement ratio in vertical direction ($\rho_v = A_v / bs_v$)
ρ_h = reinforcement ratio in horizontal direction ($\rho_h = A_v / bs_h$)
A_v & A_h = total area of stirrups or horizontal bars in one spacing (in.^{2})
s_v & s_h = spacing of stirrups or horizontal (skin reinf.) bars (in.)
b = width of web (in.)

Directions:
1). Determine ρ_v and ρ_h for bent cap
2). Measure maximum diagonal crack width, w_{max}, in inches
3). Use chart with w_{max}, ρ_v, and ρ_h to estimate % of capacity. Interpolate for intermediate values ρ_v and ρ_h. For unequal ρ_v and ρ_h use the average of the two when reading off the chart.

Important Notes:
In this chart, the maximum width of the primary diagonal crack in a shear-critical member is linked to the load on the member, quantified as a percent of its ultimate capacity. The intent of this chart is to aid field engineers in evaluating residual capacity in diagonally-cracked, reinforced-concrete bent caps subject to concentrated loads at a/d ratios between 1.0 and 2.5. This chart was developed from crack width data from 33 tests of simply supported reinforced concrete inverted-T beams with overall heights between 42" and 75". The testing was conducted at a/d ratios of 1.85 and 2.5.

This chart should be used in conjunction with sound engineering judgment with consideration of the following limitations:
-variability in crack widths in general (± scatter)
-members loaded at a/d < 1.85 may be at slightly higher % of capacity
-differences between field and laboratory conditions
-implications of an unconservative estimate of capacity

This chart is intended to be used for inverted-T bent caps. Not applicable with reinforcement ratios above 0.6%

Informed Decision-Making

In all cases, engineering judgment and analysis are necessary to determine the appropriateness of structural retrofit and/or load posting for a bridge that is supported by a cap with diagonal cracks. Tables 1 and 2 can be viewed as additional tools in the toolbox to inform decision-making about reinforced concrete bridge substructure members with diagonal shear cracks.

To the best of my knowledge, such information does not readily exist for prestressed concrete (both pretensioned and post-tensioned concrete) superstructures. With that said, the MCFT-based shear design provisions in the AASHTO LRFD specifications may also come in handy in evaluating the capacity of bridge components in service when tools such as those presented in Tables 1 and 2 do not apply. Thoughts and research to consider for shear capacity evaluation include the following:

- The AASHTO LRFD specifications adopted MCFT, at least in part, because of the theory's accuracy in predicting the shear behavior of reinforced and prestressed concrete members. MCFT-based predictions offered the consistency needed to calibrate the shear design provisions in the AASHTO LRFD specifications.

- After the AASHTO LRFD specifications adopted MCFT-based shear design provisions in 1994, many research teams investigated the provisions' accuracy and conservativeness. For example, Nakamura et al.[8] examined an extensive database of shear design provisions from around the globe and concluded that the MCFT-based provisions were the most accurate.

- Zaborac et al.[9] and Holt et al.[10,11] offer detailed approaches that can be used in evaluating the inventory of prestressed concrete beams. The approaches described in these publications have a strong theoretical basis and therefore serve as great tools to use when evaluating prestressed concrete superstructures. Importantly, the approaches outlined in these documents to aid in the load-rating process employ MCFT in an inverse manner compared to that used in design. (Design starts with loads and designs or details a member, whereas inverse analysis starts with a crack pattern and/or structural details and then estimates the load acting on the member.)

Concluding Remarks

Structural behavior of shear-critical components has been a topic of significant interest to many researchers and funding agencies since the end of 19th century and the beginning of the 20th century. Significant insights gained around the globe have helped in the code and specification development processes. Research and development efforts for the AASHTO LRFD specifications and AASHTO's *Manual for Bridge Evaluation*[12] have helped advance state-of-the-art procedures used to evaluate structural components that show signs of distress. The evaluation of the existing inventory of structures, part of the stewardship efforts that are underway, remains a topic of interest in the research community. In the future, I fully expect that the toolbox for bridge evaluation will be further populated by useful tools and techniques developed in the United States and around the globe. As the bridges in the current U.S. inventory age and the traffic loads continue to increase, we will need to extend the service lives of our bridges and participate in the responsible renewal or retrofit of those bridges that require such actions.

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Concrete Segmental Bridges—

Preliminary Design Approximations for Creep Redistribution, Post-Tensioning Secondary Moments, and Thermal-Gradient Stresses

by R. Kent Montgomery, GM2 Associates

This article, which is the third in a series discussing preliminary design approximations for concrete segmental bridges, covers methods to estimate creep redistribution, post-tensioning secondary moments, and thermal-gradient stresses. For most loads, such as superimposed dead loads and live loads, moment demand can be determined through the use of a simple finite element model with a limited number of nodes and elements, and no consideration of time-dependent effects. Before the introduction of desktop computers, beam charts were used for this purpose. However, determining the moments and stresses covered in this article is a more time-consuming process that requires more-precise modeling; therefore, simplified methods and approximations are helpful for preliminary designs.

For instance, the determination of the moment demands from creep and shrinkage requires a complex time-dependent—and time-consuming—model, which is typically developed in final design. Therefore, a simplified method for determining redistribution moments is desirable for preliminary design. Post-tensioning secondary moments are typically treated as a demand and are dependent on the number, size, and profile of the selected tendons in the bridge. Secondary moments can be calculated with a finite element model that includes the modeling of post-tensioning tendons in final design. However, a simple and noniterative method for determining secondary moments is desirable for preliminary design. Similarly, the computation of stresses due to nonlinear thermal gradients is time-consuming, and a simple approximate calculation method is desirable for preliminary design.

Creep Redistribution

When a superstructure is erected in a static scheme different from the final static scheme of the bridge, the forces in the superstructure will tend to redistribute due to creep. To help understand the redistribution effect, consider the structure constructed using the balanced-cantilever method shown in **Fig. 1**. After the cantilevers are erected, there are large negative moments over the piers and no positive moments at midspan due to the self-weight of the cross section. As shown in Fig. 1, cantilever stresses have higher levels of compression in the bottom fiber than in the top fiber. Therefore, creep of the concrete will cause the cantilever to deflect downward over time. If the cantilever is free, there is a corresponding rotation at its tip. However, if continuity has been established between cantilevers, the rotation of the ends of the cantilevers is restrained. As seen in Fig. 1, the moments that develop because of this restraint are positive. Therefore, for this balanced-cantilever example, positive moments develop at midspan and the negative moments over the piers are reduced. The net effect is a redistribution of the self-weight moments, with the moment diagram shifting downward (**Fig. 2**).

In general, for any structure, there is a self-weight moment diagram (and corresponding set of coincident forces for every degree of freedom) that occurs at the end of construction. Historically, this moment diagram has been termed the S_1 state. This S_1 moment diagram is dependent on the geometry and properties of the structure (span length, cross-sectional properties, self-weight, and so on), as well as the sequence and methods used to construct the structure.

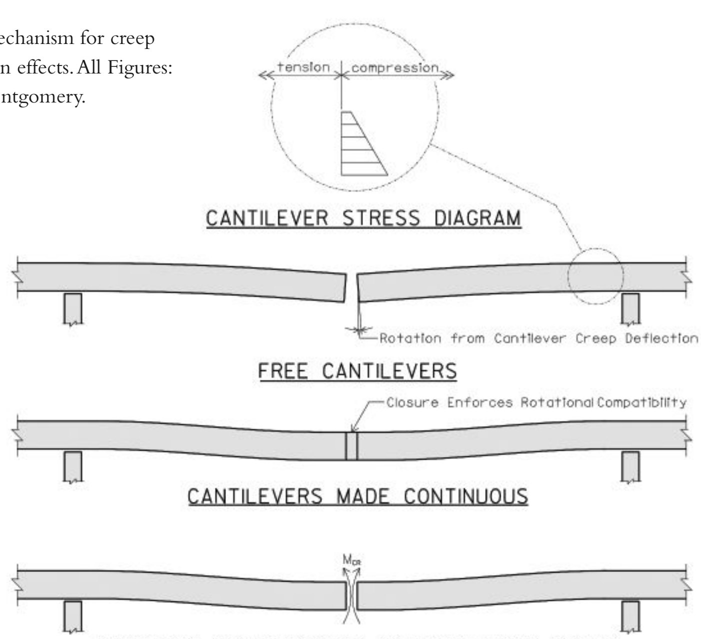

Figure 1. Mechanism for creep redistribution effects. All Figures: R. Kent Montgomery.

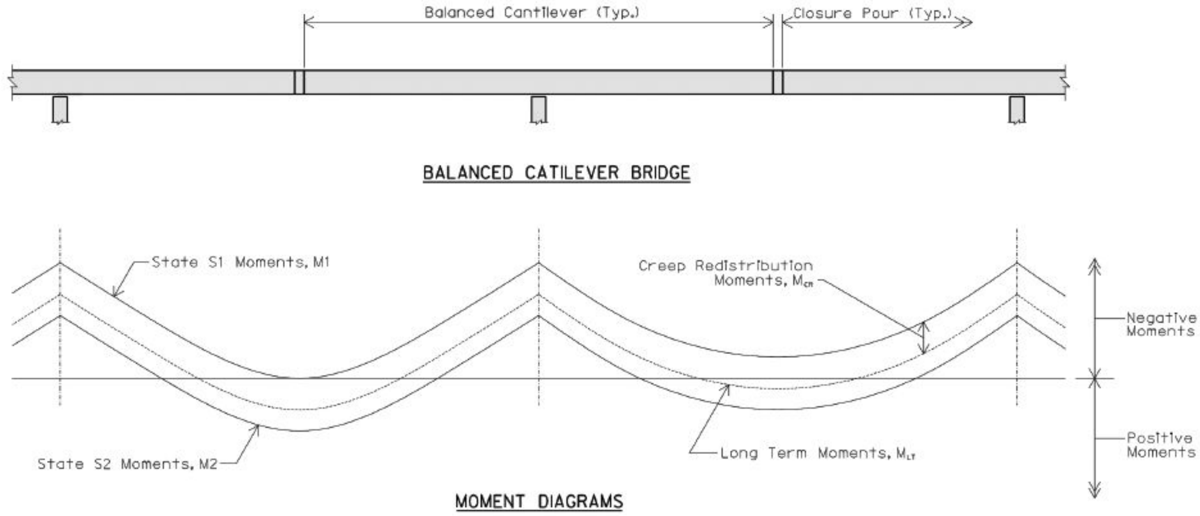

Figure 2. Creep redistribution in a concrete segmental bridge constructed using the balanced-cantilever method.

A different self-weight moment diagram would apply if the structure were analyzed as continuous for self-weight loading (as if the entire structure were built on falsework). This moment diagram is termed the continuous state or, historically, the S_2 state. As a general rule, concrete creep will cause the moment diagram to shift from the S_1 state toward the S_2 state as follows:

$$M_{LT} = M_1 + M_{CR}$$

where
M_{LT} = long-term moment
$M_{CR} = (1 - e^{-\psi})(M_2 - M_1)$ = creep redistribution moment
M_1 = S_1 state moment
M_2 = S_2 state moment
ψ = creep coefficient

Typically, the moment diagram never fully reaches the S_2 state but is instead somewhere between the S_1 and S_2 states. For preliminary design, it is reasonable to make the following assumptions:

- $(1 - e^{-\psi}) = 0.0$ for creep redistribution moments at the end of construction
- $(1 - e^{-\psi}) = 0.5$ to 0.7 for long-term creep redistribution moments

Figure 2 illustrates the use of this concept.

Note that although reference has been made to self-weight moments in the discussion about redistribution, the amount of post-tensioning and the point in time when a member is tensioned influence the amount of redistribution that occurs. For example, if additional cantilever post-tensioning is used, the compressive stress diagram in the cantilever becomes more uniform between the top and bottom fibers. Therefore, creep will produce more axial contraction but less downward deflection. As discussed earlier, it is the downward deflection that leads to the redistribution effect; therefore, adding cantilever post-tensioning reduces the amount of creep redistribution for the example shown in Fig. 2. However, the assumptions made earlier are usually adequate for preliminary design.

Other load effects, including barriers, wearing surface, live load, and temperature effects (uniform temperature and temperature gradient) are typically analyzed for the continuous structure and, as such, are not subject to the same time-dependent considerations.

Secondary Moments

Post-tensioning secondary moments arise in statically indeterminate structures due to restrained deformations from the post-tensioning. The secondary moments most relevant to this discussion are caused by restrained rotations. For example, a simple-span bridge is free to rotate at the ends of the span due to the primary post-tensioning forces. However, if the bridge is continuous, the ends of the span are not free to rotate unrestrainedly, and secondary post-tensioning moments develop. Note that primary post-tensioning forces are those applied by the prestressing without any restraints:

$$P_p = P_{pt}$$
$$M_p = P_{pt} \times e$$

where
P_p = primary post-tensioning axial force
P_{pt} = applied post-tensioning axial force
M_p = primary post-tensioning moment
e = tendon eccentricity

Forces developed due to restraint reactions are termed secondary forces. For calculation of stresses, the sum of these forces should be used. For the strength limit state, the secondary moments are treated as demands and the prestressing itself is used in capacity calculations.

The secondary moments for any tendon are dependent on the tendon length, profile, and position in the bridge. Therefore, analysis programs are the best option to exactly calculate secondary moments. However, the process of defining tendons in such a program to arrive at a post-tensioning layout would be iterative and time-consuming. The assumptions presented herein allow for simple, quick calculations with enough accuracy for preliminary design.

An important concept, without getting into detailed calculations, is that secondary moments are proportional to free-end rotations (rotations without restraints). Free-end rotations are then proportional to the area above or below the neutral axis for the primary moment diagram along the span (classic moment-area theorem). Experience has shown that some simple assumptions about the magnitude of secondary moments as a percentage of primary moments are sufficiently accurate for determining a preliminary post-tensioning layout to advance to final design.

For typical concrete segmental bridges constructed by the span-by-span method, it is reasonable to assume that the secondary moments in interior spans are 50% of the positive primary moment from all tendons. In other words, for each interior span, the secondary moment in the span is positive and equal to 50% of the maximum primary moment from all tendons. This assumption holds true when the span lengths in a unit are roughly equal (±20%) and the post-tensioning layout consists primarily of draped tendons anchoring in the pier or expansion diaphragms at the ends of each span and deviating in the span as shown in **Fig. 3**. The secondary moment diagram can be assumed to be constant across interior spans and decrease to zero at the free end of the unit (across the end spans). Therefore, the secondary moments at critical locations in an end span are less than the secondary moments in interior spans and can be determined by linear interpolation.

Note that the locations of the deviation diaphragms influence the magnitude of the secondary moments. Figure 3 shows that the closer the deviation diaphragms are to the pier diaphragms (that is, the further apart the deviation diaphragms are), the greater the area of the primary moment diagram below the neutral axis and, hence, the greater the secondary moments will be. However, the deviation diaphragms must be far enough apart such that the straight run of the tendons in the lowest position between deviation diaphragms captures the moment diagrams for load demands, including the effects of creep. (Note that creep redistribution is small for span-by-span bridges.) Spacing the deviation diaphragms approximately one-quarter of the span length apart is typically optimal (Fig. 3). Spacing them further apart increases the magnitude of the secondary moments and, therefore, decreases the efficiency of the tendons. Spacing them closer together does not capture the load-demand diagram as described previously. For interior spans, locating the deviation diaphragms so that they are centered in the span is usually optimal. For end spans, locating the deviation diaphragms so that they are centered on a location 40% of the span length from the end of the unit is usually optimal.

For constant-depth balanced-cantilever bridges, it is reasonable to assume that the secondary moments are 50% of the positive primary moment from all continuity tendons. (For each span, the secondary moment in the span is positive and 50% of the maximum primary moment from all continuity tendons.) This assumption applies to both bottom slab and draped span-by-span-style continuity tendons. Note that the bottom-slab tendons are not draped, and the primary moment diagram is a straight line at the maximum eccentricity. The area under the primary moment diagram is smaller for shorter bottom slab tendons, resulting in smaller secondary moments, and the area under the primary moment diagram is larger for longer bottom slab tendons, resulting in larger secondary moments (**Fig. 4**). The assumption that the secondary moments are 50% of the primary moments for bottom slab tendons is based on the average for all tendons. The secondary moment diagram can be assumed to be constant across interior spans and decrease to zero at the end of the unit across end spans. The same considerations for span-by-span bridges apply to locating the deviation diaphragms for draped tendons.

For variable-depth balanced-cantilever bridges, the location of the neutral axis is not a straight line across the span but instead follows a profile similar to the intrados profile (**Fig. 5**). The bottom slab tendon profile follows a profile just below the upper surface of the bottom slab, and it reasonable to assume that the secondary moment for these bottom slab tendons is positive and is 50% of the primary moment. Due to the profile of the neutral axis, the area under the neutral axis is smaller for draped tendons and is partially offset by the area above the neutral axis. Therefore, draped tendons

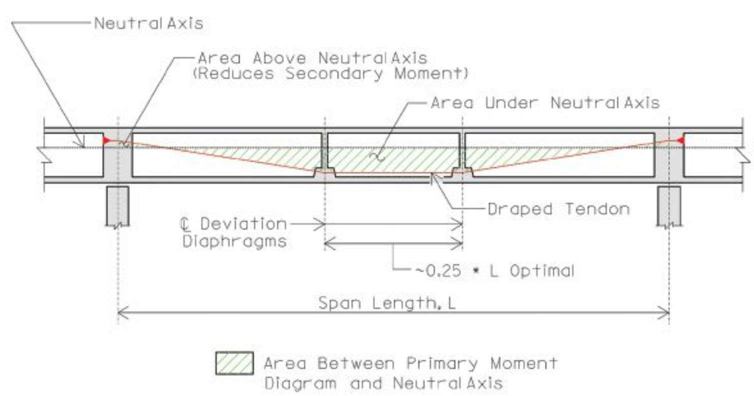

Figure 3. Draped post-tensioning tendon layout for a concrete segmental bridge constructed using the span-by-span method.

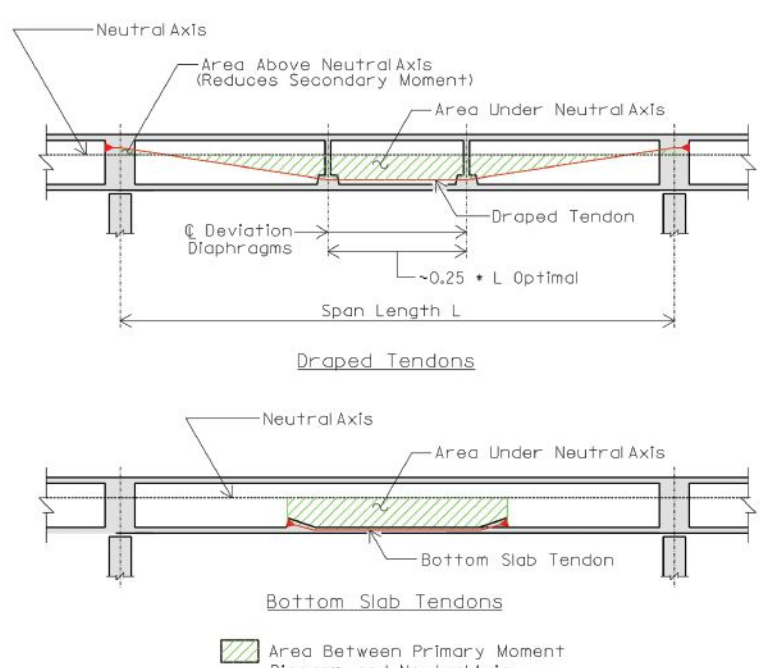

Figure 4. Continuity post-tensioning tendon layout for a constant-depth concrete segmental bridge constructed using the balanced-cantilever method.

are more efficient for variable-depth bridges than for constant-depth bridges, and it is reasonable to assume that the secondary moment for these draped tendons is positive and is 25% of the primary moment. The total secondary moment in the span is the sum of the secondary moments from the bottom slab and draped tendons. The secondary moment diagram can be assumed to be constant across all interior spans and to decrease to zero at the free end of the unit (across the end spans). The same considerations for span-by-span bridges apply to locating the deviation diaphragms for draped tendons.

For preliminary design, the number of required tendons can be estimated based on keeping stresses within the limiting stresses for the service limit state. Typically, end spans with external tendons represent the only situation in which the service limit state does not govern the amount of post-tensioning. A quick calculation of the moment capacity can determine whether the amount of post-tensioning in these spans needs to be increased.

For calculational purposes, the concept of tendon efficiency can be used to estimate the amount of post-tensioning. For example, for a tendon where the amount of secondary moment is 25% of the primary moment, the tendon is 75% efficient and the total stresses due to the post-tensioning can be calculated from the full axial force and 75% of the primary moment.

Temperature Gradient Stresses

Historically, a 10°C (18°F) positive linear temperature gradient was applied for design—with a positive gradient indicating that the top fiber is warmer than the bottom fiber. After 1989 and with the introduction of the American Association of State Highway and Transportation Officials' *Guide Specifications for Design and Construction of Segmental Concrete Bridges*,[1] nonlinear temperature gradients were specified, including a negative gradient. An accurate computation of stresses due to a nonlinear temperature gradient requires involved calculations; however, for preliminary design, simplifying assumptions can be made to ease the

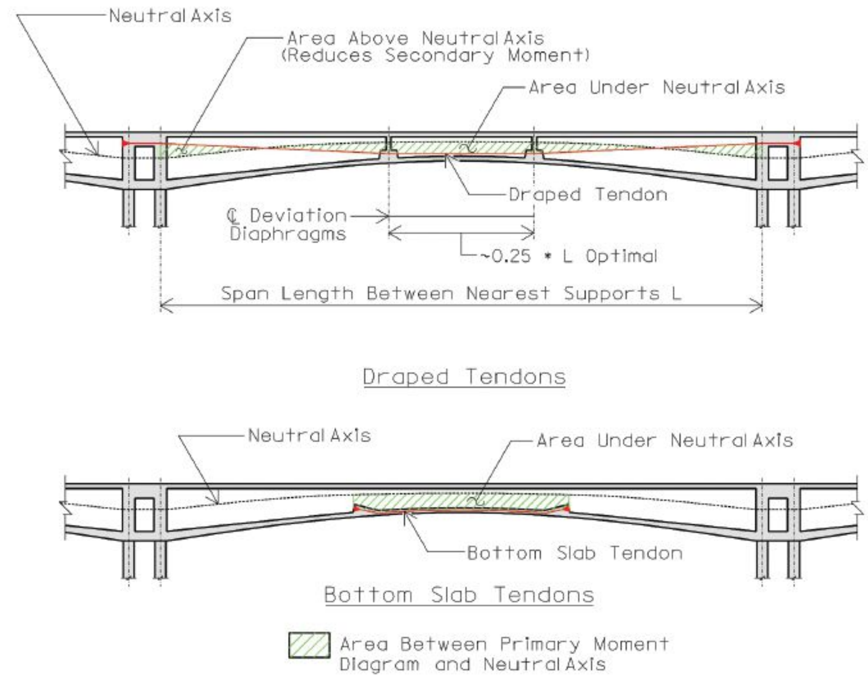

Figure 5. Continuity post-tensioning tendon layout for a variable-depth concrete segmental bridge constructed using the balanced-cantilever method.

computational burden. In concrete segmental bridges the stresses due to temperature gradients are much smaller than those from permanent loads and live loads; therefore, very simple assumptions yield adequate results.

First, the negative temperature gradient can be ignored for preliminary design. Negative temperature gradients typically only govern where the live-load stresses in the top slab are small (for example, near the free end of end spans), and refining the final design is relatively simple for these special regions.

For the positive temperature gradient, experience has shown that the following equivalent moment can be used for preliminary design:

$$M_{tg} = (\Delta T \times E \times I \times \alpha)/h$$

where

M_{tg} = equivalent moment for calculating stresses from a positive temperature gradient

ΔT = 15°F approximate temperature gradient for preliminary design

E = concrete modulus of elasticity

I = cross section moment of inertia

α = concrete coefficient of thermal expansion

h = overall depth of cross section

This thermal gradient moment is positive and roughly equivalent to the restraint moment that develops from a linear temperature gradient. The gradient of 15°F is a little less than the historically used 18°F linear gradient to account for the helpful bottom fiber compressive internal stresses from a positive nonlinear gradient. The full M_{tg} can be applied for interior spans, and the moment can be assumed to decrease to zero at the free end of the unit (across the end spans). For variable-depth spans, I and h can be taken at a section approximately 20% of the span length from the midspan.

Conclusion

The simplifications presented in this article can be used to calculate the amount of post-tensioning for concrete segmental bridges constructed by the span-by-span method and the amount of continuity post-tensioning for balanced-cantilever bridges. Methods to determine the amount of cantilever post-tensioning for balanced-cantilever bridges and the cross-sectional dimensions were discussed in previous articles in *ASPIRE*®. The remaining task for preliminary design is to lay out the individual post-tensioning tendons. This topic will be discussed in the next article in this series.

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Advancing Quality, Driving Innovation: The Epoxy Interest Group's Role in Reinforcing Concrete Bridges

by Brent Toller, Epoxy Interest Group

In 2008, the Epoxy Interest Group (EIG) was established within the Concrete Reinforcing Steel Institute (CRSI) to promote the use, quality, and performance of epoxy-coated reinforcing steel. What began as a focused effort to elevate the standards for corrosion-resistant reinforcement has since developed into a highly collaborative, technically driven, and growing trade association. EIG operates as an "institute within the institute," with its own steering committee, separate membership dues, budget, and dedicated website (epoxyinterestgroup.org).

The member companies include epoxy-powder suppliers, coating applicators, and reinforcing bar fabricators. All members are committed to EIG's mission statement:

> ***To increase the quality and performance of epoxy encapsulated rebar, dowel bars, and wire mesh in protecting the integrity of steel and concrete.***

EIG seeks to fulfill this mission by working closely with CRSI, owners, and other industry stakeholders to enhance quality across the entire epoxy-coated reinforcement supply chain. Two critical programs are the CRSI Epoxy Coating Plant Certification and the CRSI Certification for Fabrication of Epoxy-Coated Reinforcement. Both certifications are voluntary, but they are increasingly recognized and required by state departments of transportation (DOTs). In fact, 31 DOTs now require participation in the CRSI applicator certification program, which is a powerful testament to the program's value and reliability.

Launched in 1991, the CRSI Epoxy Coating Plant Certification for fusion-bonded epoxy-coating applicator plants focuses on ensuring that epoxy coating is applied with the highest standards. A third-party auditor accredited to ISO/ICE 17065:2012 evaluates plant operations with regard to coating thickness, adhesion, curing, holiday testing, surface preparation, and more. This auditing process is rigorous, and plants know they must perform at the highest level to meet the quality criteria during the coating process to achieve and maintain certification.

But what happens after the coating process—when the bar is fabricated, stored, and shipped? This is where EIG's second major program takes effect.

In response to DOT concerns and owner feedback, EIG worked with CRSI to introduce the CRSI Certification for Fabrication of Epoxy-Coated Reinforcement Program in 2013. This certification ensures that epoxy-coated reinforcing bars are handled, cut, bent, touched up, and transported properly, thereby safeguarding coating integrity and maintaining corrosion protection throughout the bars' journey to the jobsite. The program reviews quality-control processes, handling and storage procedures, repair protocols, and employee training. As the industry has embraced this program, DOTs have gained greater confidence that fabricated epoxy-coated reinforcing bars will perform to the highest expectations.

The CRSI Certification logo signifies that a plant meets quality standards for epoxy-coated reinforcement, verified through independent third-party audit. All Photos and Figures: Epoxy Interest Group.

From Experience: Why Certification Matters

As a former epoxy-coating plant manager, I can say from experience that these EIG and industry-driven programs are not just "check-the-box" exercises—

Epoxy-coated reinforcing bars, coated and then fabricated in accordance with ASTM A775 requirements.

Epoxy-coated reinforcing steel provides long-term corrosion protection in bridge applications, especially in areas exposed to deicing salts and moisture such as the driving surface of the Mount Hope Bridge (shown here), which spans the Arkansas River in Kansas.

they shape how a plant operates daily. As required by our CRSI certification, we trained every employee on our quality plan and CRSI's requirements. If we were not meeting CRSI's certification standards, we would risk losing business. At one point, our plant served 12 states, 10 of which required CRSI certification. If we failed one audit, our market would have been reduced from 12 states to 2.

The quality-driven environment that we maintained to pass the annual audits kept us sharp. We knew that the inspectors were reviewing our documentation, including our equipment inspection logs and our history of coating samples and tests. We also knew that we were accountable to our customers—DOTs, contractors, and engineers who expect durability, consistency, and excellence. The CRSI Epoxy Coating Plant Certification stamp meant our product was meeting the highest industry standards.

Raising the Bar for the Next Generation of Coating Technology

EIG is not only focused on current standards ASTM A775[1] and A934[2]—it is also actively shaping the future of corrosion-resistant reinforcement coatings. (See the Safety and Serviceability articles in the Spring 2022 and Fall 2022 issues of *ASPIRE*® for more information about the current standards.) Working in partnership with CRSI, EIG has supported the rollout of the new ASTM A1124 standard for textured epoxy-coated reinforcing bar.[3] This next-generation coating for reinforcing steel improves adhesion to concrete and long-term durability of the coating, and CRSI has already established a new certification program to support it. That means DOTs and owners can require the same level of quality assurance for this advanced coating that they expect for traditional epoxy coatings.

For more information about ASTM A1124 and textured coating developments, see the Safety and Serviceability article in the Winter 2024 issue of *ASPIRE*. This new technology represents a pivotal shift for the epoxy-coating industry.

EIG and CRSI: A Shared Mission

Our trade group's strength lies not only in the quality standards it maintains but also in its unity with our umbrella trade organization. Because EIG is a specialized trade group within the 100-year-old CRSI organization, our trade group benefits from the broader resources and stellar reputation of one of the longest-standing U.S. industry associations. Each EIG member is also a CRSI member, and we are unified in the greater mission to promote and support the steel reinforcement industry.

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Brent Toller is the technical marketing manager for the Epoxy Interest Group of the Concrete Reinforcing Steel Institute, and has more than 25 years of experience in reinforced concrete and corrosion-resistant materials. His background includes plant management and technical sales, with a focus on the performance and quality of highway products and protective coatings.

AASHTO LRFD

Shear Design Provisions

by Dr. Oguzhan Bayrak, University of Texas at Austin

With the introduction of new materials (such as portland limestone cements, ultra-high-performance concrete, and engineered cementitious composites), the discussion on shear strength of reinforced and prestressed concrete inevitably resurfaces. Recently, our *ASPIRE*® team has received questions about the applicability of the shear design provisions in the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[1] to new materials and new geometries and/or structural systems. The intent of this article is to summarize the shear design provisions of the AASHTO LRFD specifications and place that summary in context with respect to time and experimental data.

What Does the AASHTO LRFD Specifications Say?

Article 5.7.3 of the 10th edition of the AASHTO LRFD specifications provides guidance for shear design. Since the first edition of the AASHTO LRFD specifications (1994), these provisions have been simplified to eliminate iterations. However, the current edition preserves the original form of the design provisions based on modified compression field theory (MCFT); those provisions are provided as an alternative to the simplified form of the expressions in Appendix B5. Let us first look at the provisions of Article 5.7.3:

The nominal shear resistance, V_n, shall be determined as the lesser of both of the following:

$$V_n = V_c + V_s + V_p \qquad (5.7.3.3\text{-}1)$$

$$V_n = 0.25 f'_c b_v d_v + V_p \qquad (5.7.3.3\text{-}2)$$

in which:

$$V_n = 0.0316 \beta \lambda \sqrt{f'_c}\, b_v d_v \qquad (5.7.3.3\text{-}3)$$

$$V_s = \frac{A_v f_y d_v \left(\cot\theta + \cot\alpha\right)\sin\alpha}{s}\lambda_{duct} \qquad (5.7.3.3\text{-}4)$$

$$\lambda_{duct} = 1 - \delta\left(\frac{\phi_{duct}}{b_w}\right)^2 \qquad (5.7.3.3\text{-}5)$$

where:

V_p = *component of prestressing force in the direction of the shear force; positive if resisting the applied shear*

b_v = *effective web width taken as the minimum web width within the depth d_v, as determined in Article 5.7.2.8 (in.)*

d_v = *effective shear depth, as determined in Article 5.7.2.8 (in.)*

β = *factor indicating ability of diagonally cracked concrete to transmit tension and shear, as specified in Article 5.7.3.4*

λ = *concrete density modification factor, as specified in Article 5.4.2.8*

A_v = *area of transverse reinforcement within a distance, s (in.^{2})*

θ = *angle of inclination of diagonal compressive stresses, as determined in Article 5.7.3.4 (degrees)*

α = *angle of inclination of transverse reinforcement to longitudinal axis (degrees)*

s = *spacing of transverse reinforcement measured in a direction parallel to the longitudinal reinforcement (in.)*

λ_{duct} = *shear strength reduction factor accounting for the reduction in the shear resistance provided by transverse reinforcement due to the presence of a grouted post-tensioning duct. Taken as 1.0 for ungrouted post-tensioning ducts and with a reduced web or flange width to account for the presence of ungrouted duct.*

δ = *duct diameter correction factor, taken as 2.0 for grouted ducts*

λ_{duct} = *diameter of post-tensioning duct present in the girder web within depth d_v (in.)*

b_w = *gross width of web, not reduced for the presence of post-tensioning ducts (in.)*

The parameters β and θ may be determined either by the provisions herein, or alternatively by the provisions of Appendix B5.

For sections containing at least the minimum amount of transverse reinforcement specified in Article 5.7.2.5, the value of β may be determined by Eq. 5.7.3.4.2-1:

$$\beta = \frac{4.8}{\left(1 + 750\varepsilon_s\right)} \qquad (5.7.3.4.2\text{-}1)$$

When sections do not contain at least the minimum amount of shear reinforcement, the value of β may be as specified in Eq. 5.7.3.4-2:

$$\beta = \frac{4.8}{\left(1 + 750\varepsilon_s\right)}\frac{51}{\left(39 + s_{xe}\right)} \qquad (5.7.3.4.2\text{-}2)$$

The value of θ in both cases may be as specified in Eq. 5.7.3.4.2-3:

$$\theta = 29 + 3500\varepsilon_s \qquad (5.7.3.4.2\text{-}3)$$

Here, ε_s is an approximation of the longitudinal steel strain (Eq. 5.7.3.4.2-4) and s_{xe} is a crack spacing parameter (Eq. 5.7.3.4.2-7).

As it relates to segmentally constructed concrete bridges, the AASTHO LRFD specifications offer an alternate shear design method in Article 5.12.5:

In lieu of the provisions of Article 5.7.3, the provisions herein may be used to determine the nominal shear and torsion resistance of post-tensioned concrete box girders in regions where *it is reasonable to assume that plane sections remain plane after loading.*

Transverse reinforcement shall be provided when $V_u > 0.5\phi V_c$*, where* V_c *is computed by Eq. 5.12.5.3.8c-3.*

The factored nominal shear resistance, ϕV_n*, shall be greater than or equal to* V_u*.*

The applied factored shear, V_u*, in regions near supports may be computed at a distance* h*/2 from the support when the support reaction, in the direction of the applied shear, introduces compression into the support region of the member and no concentrated load occurs within a distance,* h*, from the face of the support.*

The nominal shear resistance, V_n*, shall be determined as the lesser of the following:*

$$V_n = V_c + V_s \qquad (5.12.5.3.8c\text{-}1)$$

$$V_n = 0.379\lambda\sqrt{f'_c}\,b_v d \qquad (5.12.5.3.8c\text{-}2)$$

in which:

$$V_c = 0.0632K\lambda\sqrt{f'_c}\,b_v d \qquad (5.12.5.3.8c\text{-}3)$$

$$V_s = \frac{A_v f_y d}{s} \qquad (5.12.5.3.8c\text{-}4)$$

$$K = \sqrt{1 + \frac{f_{pc}}{0.0632\lambda\sqrt{f'_c}}} \le 2.0 \qquad (5.12.5.3.8c\text{-}5)$$

The alternate shear design method for segmentally constructed concrete bridges is provided for the designer's benefit in recognition of the successful use of the alternate provisions for decades and the superior field performance of segmentally constructed concrete bridges in the United States.

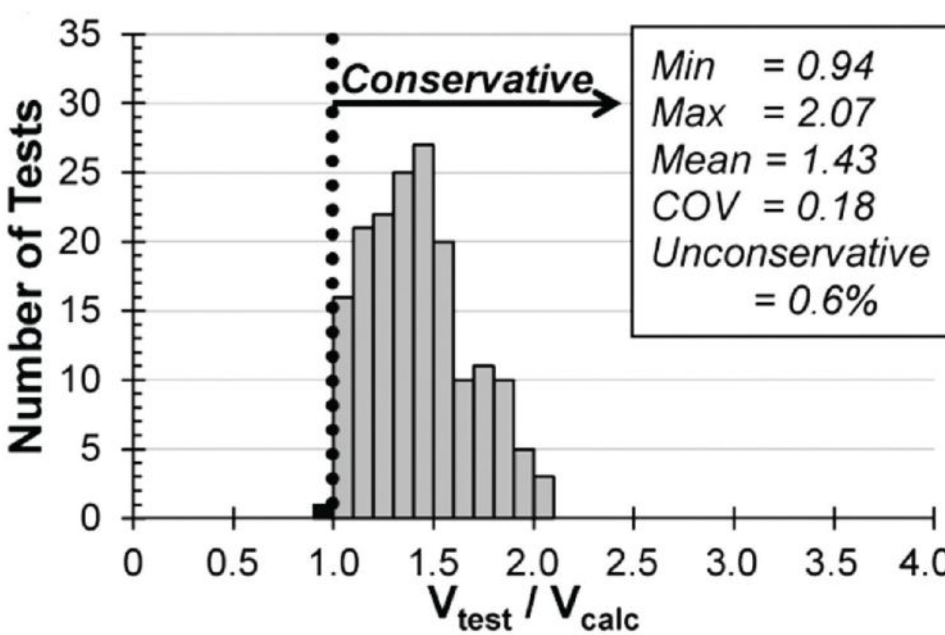
With that information serving as a backdrop, let us focus our attention on a research project we have recently completed at the University of Texas. Within the context of that research, we have assembled a comprehensive shear database to evaluate the statistical performance of a variety of code provisions from around the world.[2] Eisuke Nakamura, a Japanese researcher, provided significant contributions that allowed us to expand our shear database to include a large number of tests from Japan. I must also recognize the initial efforts of Alejandro R. Avendaño, a Panamanian researcher, who was instrumental in initiating this database. I view this collective international effort as significant because there are many Japanese tests on post-tensioned beams and without Nakamura's and Avendaño's work, we would not have such an exhaustive database. Our database at the completion of that research effort contained 1696 tests reported in 99 references dating from 1954 to 2010. We have since further expanded our database with more recently conducted tests, and at the writing of this article, we have documented approximately 2000 tests conducted on prestressed concrete girders. Interestingly, the more recent tests do not challenge the conclusions reached in 2013[2] that MCFT yields the best estimate of shear resistance for prestressed concrete members; rather those conclusions have been further validated by the newer tests.

The original derivation of the MCFT-based shear design provisions benefited from tests conducted on reinforced concrete panels. In effect, panel tests provided the necessary data for the calibration of the essential MCFT equations. Once the derivation was complete and the equations were calibrated and verified with panel tests, the application of the equations to reinforced concrete beam tests proved to be very successful. Furthermore, the application of MCFT-based expressions to the shear behavior of prestressed concrete components also proved to be successful. Nakamura et al.[2] is only one of many studies that prove the efficacy of MCFT-based shear design expressions.

Figure 1 illustrates the performance of the AASHTO LRFD specifications based on MCFT in predicting the shear strength of prestressed concrete beams included in the large database discussed earlier. Importantly, the set of 1696 tests assembled in the collection database was reduced to 171 representative tests with confirmed shear failures and sectional proportions that lend themselves to a fair evaluation of the design provisions for concrete bridges—that is, big beams with representative structural details.

The actual and unbiased reduction from the collection database to the evaluation database is described in detail by Nakamura et al. As they reported, the implications of the exceptional performance displayed by the MCFT-based shear design provisions of the AASHTO LRFD specifications are rather significant: First, the provisions work well for pretensioned (70% of the data) and post-tensioned (30% of the data) concrete beam tests. Second,

Figure 1. Performance of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications* shear design provisions based on the modified compression field theory for estimating the nominal shear strength of prestressed concrete components.[2]



the provisions work well uniformly across a variety of cross-sectional shapes (rectangular beams, I-beams, U beams, box beams, T beams and inverted-tee beams). Third, the statistical indicators listed in Fig. 1 represent the best performance among all design provisions evaluated in the paper. Nakamura et al. evaluated the accuracy and conservativeness of the eight shear design provisions included in five design codes. The MCFT-based design expressions yielded the most accurate shear strength estimations for prestressed concrete components. In this context, a coefficient of variation (COV) of 0.18 is impressive.

Nakamura et al. also evaluated the legacy shear design method employed in designing segmentally constructed concrete bridges. **Figure 2** shows that these provisions, as presented in the earlier sections of this article, proved to be conservative, albeit less accurate (COV = 0.29) in their ability of estimating shear strength.

The philosophical differences in the derivation of MCFT-based design expressions, on the one hand, and the principles used in developing the shear design expressions for use in segmentally constructed bridges, on the other, lead to the differences that we see in the statistical performance of these design expressions. Both sets of provisions work well as part of a process that results in a safe, sufficiently conservative shear design. In evaluating the likely shear strength of existing inventory of segmentally constructed bridges, we can see that MCFT-based expressions estimate the shear strength more accurately. Interestingly, within the context of the *Model Code 2020* published by *fib* (International Federation for Structural Concrete),[3] we use a framework called "Levels of Approximation." This framework recognizes that all calculations are approximations of actual strength values. Some approximations are more detailed/rigorous, and others are more useful in obtaining initial estimates for strength. All approaches are valuable within the rationale/reasoning of their derivation.

Since the publication of the paper by Nakamura et al., our testing and database assembly efforts at the University of Texas at Austin have progressed. At the writing of this article, our shear database contains additional tests and we can offer further validation. In this sense, it is important to know that the conclusions reported by Nakamura et al. remain unaltered, and additional testing did not reveal issues that require changes to the AASHTO LRFD specifications.

Moving from traditional materials and construction techniques to the new ones, I expect that design procedures with strong theoretical foundation will continue to perform well. Work is underway on ultra-high-performance concrete (UHPC) design guidance and the AASHTO concrete committee has adopted a procedure that is based on MCFT but calibrated with new UHPC data. As new materials come on board, the need for additional research will remain and we will continue to look at the applicability of rational shear design procedures, such as MCFT, in light of new data.

In addition, there will always be a place for simpler methods that can offer initial strength estimates for applications that are consistent with the intent of their derivation.

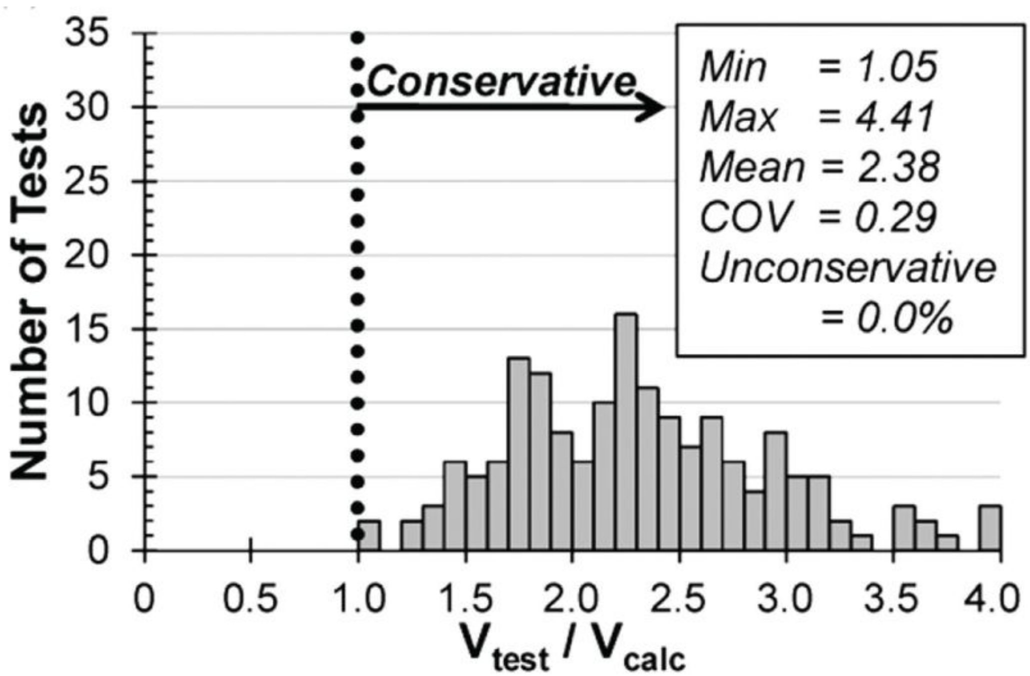
References

1. American Association of State Highway and Transportation Officials (AASHTO). 2024. *AASHTO LRFD Bridge Design Specifications*. 10th ed. Washington, DC: AASHTO.

2. Nakamura, E., A. R. Avendaño, and O. Bayrak. "Shear Database for Prestressed Concrete Members." *ACI Structural Journal* 110 (6): 908–918. https://doi.org/10.14359/51686147.

3. *fib* (International Federation for Structural Concrete). 2020. fib *Model Code 2020*. Lausanne, Switzerland: *fib*.

Figure 2. Performance of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications* segmental shear design provisions for estimating the nominal shear strength of segmental bridges.[2]



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

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

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