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Photo: Sinha.
Avoiding Poor Habit Transfer

William N. Nickas, Editor-in-Chief

Familiarity is comfortable. When you have a favorite pair of jeans, coat, or boots, there’s an ease to how they fit and feel. You know what you’re getting into. I’d argue that familiarity in design and construction is similar. It’s comfortable, and if we’re not careful and cognizant of this ease, we can settle into a stagnated design or construction approach. Like our favorite sweatshirt, our past solutions provide comfort; it’s easy to reach for the tried and true. These reliable solutions are consistent performers and solve the problem. It’s at that point of comfort when we might enter an area of “poor habit transfer.”

This concept has many applications and came to light for me recently when a friend borrowed a tractor with far greater operational capabilities than his own. For example, this new machine has the ability to traverse 30-degree slopes. My friend used this tractor for a few days and then returned to his own machine. Later that week, we met up to chat and he relayed how his day almost ended in disaster. After he had become comfortable on steeper slopes with the borrowed machine, he found himself on a precarious situation on his own equipment. Fortunately, he was able to slowly lessen the slope and avoid rolling over.

If we allow our approach to become overly repetitive and settle for typical, familiar offerings even when the situation changes, we risk becoming a victim of complacency brought about by poor habit transfer. That is why continuing to push new ideas, advance innovative solutions, and reenergize professional discussions will keep us from becoming complacent.

It’s in this frame of mind that we began our four-part series, “Anchors in Concrete,” on implementing the recently added Article 5.13 of the American Association of State Highway and Transportation Officials’ AASHTO LRFD Bridge Design Specifications. The article about anchors in concrete in this issue of ASPIRE® concludes this series. We owe a great deal of thanks to Dr. Donald “Don” Meinheit, Dr. Ron Cook, and Mr. Neal Anderson for their efforts in energizing our minds on this topic. Concrete anchors are not the spicy-hot topic most bridge engineers gravitate toward, but their importance cannot be overstated. Structural integrity is dependent on all bridge components working in concert, and concrete anchors play a significant role.

Dr. Oguzhan Bayrak continues to advance our discussion related to redundancy and ductility for concrete bridge design. The importance of this topic is critical as we continue to advance concrete bridge solutions that require increased girder lengths, novel geometric elucidations of beam cross sections, and new materials.

Also in this issue, Dr. Richard Miller has graciously shared his approach to teaching ethics to his engineering students as we continue to learn from the Florida International University bridge collapse. With that event also in mind, we requested and were granted permission by the National Society of Professional Engineers to reprint their PE magazine article “A Load That’s Hard to Bear.”

Our profession demands our best. While others may become complacent, we must fight the doldrums that familiarity can bring. Let’s collectively work to avoid poor habit transfer. ▲
TOUGH JOBS REQUIRE THE BEST

Reggie Holt is a senior structural engineer at the FHWA. He manages the Concrete Bridge Program at the FHWA headquarters in Washington, D.C.

Dr. Donald F. Meinheit is a retired structural engineer who worked for Wiss, Janney, Elstner Associates Inc. He has been an active PCI member since 1975.

Dr. Richard Miller is a professor of civil engineering at the University of Cincinnati. He is chair of the PCI Research and Development Council and serves on several PCI committees, including the Bridges and Student Education Committees.

The events, dates, and locations listed were accurate at the time of publication but may change as local guidelines for gatherings continue to evolve. Please check the website of the sponsoring organization.

Reader Response:
The ASPIRE™ team received a reader comment on the Perspective article “Redundancy and Ductility for Bridge Design” in the Winter 2021 issue. The comments and editor’s response are summarized below.

The reader did not understand the last sentence in the first paragraph under the subheading Structural Redundancy, which reads “However, using such methods will typically reduce the safety margin, with due credit given to sophisticated analyses.” The reader suggested that “more sophisticated analyses resulting in better estimates of strength should produce higher strengths than the minimums prescribed in the American Association of State Highway and Transportation Officials’ AASHTO LRFD Bridge Design Specifications.” Otherwise, those prescribed minimums are not conservative. Why would the use of more sophisticated analyses, which should produce higher strengths, “typically reduce the safety margin”? It would be very helpful if the sentence in question could be fully explained, starting with answering whether more sophisticated analyses resulting in better estimates of strength produce higher or lower strengths than the prescribed minimums from the AASHTO LRFD specifications.

Editor’s Response:
As the reader states, it would seem logical to assume that the use of more-refined methods of design or analysis would lead to better designs. However, the situation is more complex than one might first assume.

More-sophisticated methods of evaluating the section capacity will typically result in a higher estimate for the resistance for a given section. However, using the more refined approach can result in a design that is less conservative because the designer may provide less reinforcement to support the design loads. On the other hand, the use of more-refined methods of design or analysis may provide less reinforcement to support the design loads for a given section. However, using the more-refined approach can result in a design that is less conservative because the designer may provide less reinforcement to support the design loads.
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Ove Arup (1895–1988) was the structural innovator behind the Sydney Opera House, founded a global engineering firm, and was one of the most admired engineers of the 20th century. From those auspicious beginnings, Arup, the engineering firm that he founded, is now a well-established, multidisciplinary consulting firm with more than 90 offices in 30 countries and a large portfolio of construction and infrastructure projects throughout the world. It is well known for its creative approach to structural design and its willingness to innovate.

The Founder’s Vision
In 1946, Ove Arup founded his consulting business in London. From the start, he had a clear vision for the company. He was passionate about training the next generation of engineers and believed that hiring the right people was essential to success. For that reason, he personally interviewed many prospective employees.

Late in his career, Arup summed up his principles and vision for the firm’s future in what is known as his “key speech.” In that 1970 presentation to his partners around the world, he advocated for choosing projects that prioritize quality. “Our work should be interesting and rewarding,” he said. “Only a job done well, as well as we can do it—and as well as it can be done—is that. We must therefore strive for quality in what we do, and never be satisfied with the second-rate.” In the same speech, he promoted humanitarian values in the workplace, saying, “If we can reach a stage where each man or woman is respected for the job they do, and is doing his or her best because the atmosphere is right, because they are proud of what we are and do and share in the general enthusiasm, then we are home.”

According to Matt Carter, a principal and the Americas region bridge and civil structures skills leader in Arup’s New York, N.Y., office, the firm remains committed to its founder’s philosophy. “Arup chooses to work where we can add value, and since we are not driven by growth, we can target those projects where we feel we can make a difference.”

Ove Arup’s design of the Kingsgate Bridge, Durham, England, in 1963 was prescient of our current mode of accelerated bridge construction. The two cantilever sections of this pedestrian bridge were swung out and connected in less than an hour. The Kingsgate Bridge was the last project that Ove Arup designed and epitomizes his philosophy of total design integrating engineering and architecture.

Tawatinâ Bridge is a new 853-ft-long extradosed light rail and pedestrian bridge over the North Saskatchewan River in Edmonton, Alberta, Canada. The structure is being constructed as a cast-in-place concrete, balanced-cantilever bridge using two moveable form travelers and is part of the 27-km Edmonton Valley Line light rail transit design-build project. Photo: TransEd Partners.

Global Strategies from Organic Growth
From one man’s vision of “total design,” Arup has grown into an international, multidisciplinary consulting firm shaping the world

by Monica Schultes
a difference—whether it is innovative engineering, technological challenges, or unusual community issues that we need to address. These are frequently large complex bridge projects or pedestrian bridges which have their own unique complexities where we feel we can make more of a difference."

"Arup chooses to work where we can add value, and since we are not driven by growth, we can target those projects where we feel we can make a difference."

"We grow organically," he adds. "You won’t see Arup making major acquisitions. We pay attention to the hiring of each and every individual. It enables us to keep a strong link to our culture. Each person, as they join the firm, becomes a part of that."

Ove and the original partners left the firm in trust to its employees, and employee ownership remains part of the firm’s culture. Each employee is given shares in Arup, which means they profit when the firm profits. If employees leave Arup, they relinquish their shares; as a result, there are no external shareholders. "By having a global profit share, we can work together without internal barriers, especially in the bridge department,” says Carter.

Signature Projects
While Ove Arup believed bridges were a way to integrate architecture, structure, and construction through his signature holistic approach of total design, the Sydney Opera House was the most transformational of his designs. When Danish architect Jørn Utzon won an international competition for the project in 1957, he asked Ove Arup to help execute his exceptional design ideas, including the elaborate concrete shells on the exterior. Arup and his colleagues took advantage of emerging computer technology to calculate stresses and then devised a cost-effective technique to build molds, precast the ribs, and prefabricate the shells off site. Construction of the Opera House lasted more than a decade, with its official opening by Queen Elizabeth occurring in 1973.

Carter recalls, “Like the Sydney Opera house, there are outstanding bridge projects that demonstrate our work across the globe and demonstrate Arup’s version of definition design.”

Definition Design
"We use the ‘definition design’ concept on design-build or public-private partnership (P3) projects to ensure the final design achieves the original intent. We seek the innovation that comes with tailoring the final design to the contractor’s means and methods while protecting the architectural quality,” explains Carter.

"Recently, we used definition design for a project in Montreal as a way of bringing an architectural vision to life for the Canadian government. The definition design mandates the external form of the structure and allows the project to achieve an aesthetic design within the P3 framework without blowing up the budget that sometimes happens with design competitions." Carter adds, “During the development of preliminary design, we went to all the stakeholders in Quebec for feedback. With this local review and our own cost estimating, we were confident that the design which was revealed to the public would fit within the budget and would receive support and public endorsement.”

The Arup bridge team faced an array of engineering, technical, and community challenges as it served as the engineer of record for the Gerald Desmond Bridge replacement in Long Beach, Calif. The bridge provides a vertical clearance of 205 ft over the water to accommodate the large ships that use the port.

Cast-in-place concrete box girders and tall piers form the approaches to the 1000-ft-long main span cable-stayed bridge. Photo: Marie Tagudena.
In addition, he says, “The technical challenge was heightened as time did not permit an architectural design competition, and simply describing aesthetic requirements within traditional procurement documentation can lead to unexpected results.” The key steps of Arup’s approach included clarifying the owner’s intentions in terms of aesthetics to the bidders, ensuring that technical and visual dimensions of the design were considered simultaneously, and providing a plan for safeguarding architectural integrity through the final design stages of the project.

“Definition design has helped us with design-build and P3 procurement methods to avoid utilitarian structures and allows us to control other aspects of the design,” Carter explains. “Having independent architectural input challenges us during this process and provides feedback during preliminary stages. When design intent is revealed for the RFP [request for proposal] stage, there is already endorsement and buy-in from the stakeholders. The owner is confident that they will get the resulting bridge that comes out the other end of the bidding process.”

“The owner is confident that they will get the resulting bridge that comes out the other end of the bidding process.”

P3 Consortium
Arup is currently part of a P3 consortium working on the Tawatinâ Bridge under construction in Edmonton, AB, Canada. As the lead designer and engineer of record, Arup is collaborating with American Bridge and TransEd Partners (composed of Bechtel, EllisDon, Bombardier, and Fengate Capital Management Ltd.) to deliver the 853-ft-long asymmetrical bridge by 2022.

The three-span, extradosed bridge uses seven continuous cable stays to support two tracks of light rail and a shared-use path over the North Saskatchewan River. The Tawatinâ Bridge forms part of the Edmonton Valley Line light rail transit, which connects the community of Mill Woods in southeast Edmonton to the city’s downtown core. In this unique design, a concrete box girder supports the rail, and a pedestrian bridge is suspended from below. The cast-in-place superstructure is being constructed with the balanced-cantilever method using two moveable form travelers from the central pier outward.

In August 2020, crews installed the final set of cable stays on the bridge, and in September, TransEd made the connection of the gap between the north and south riverbanks.

Value-Added Strategies
“As we replace aging infrastructure, clients want long-life bridges, which demand a high quality of design for concrete structures,” suggests Carter. Performance-based or prescriptive-based design are both useful in certain circumstances. “We believe that there is a balance to get the right result for durability and aesthetics and that...
definition design can assist with getting that balance.”

“As we replace aging infrastructure, clients want long-life bridges, which demand a high quality of design for concrete structures.”

The Salesforce Transit Center in San Francisco, Calif., is part of $6 billion project that has transformed regional transportation in the Bay Area. The bus ramp bridge system is 1849 ft long and is the first vehicular cable-stayed bridge built in California. Arup collaborated with other members of the project team to find solutions that could meet the stringent requirements of right-of-way and myriad design challenges. (For more on this project, see the Summer 2019 issue of ASPIRE®.)

Arup’s involvement dates back to 1998, when the team was selected to prepare the Transbay Terminal Improvement Plan. Work on concept validation began in June 2008 and included complete engineering design, bid phase, and construction administration services through project completion. In total, Arup provided civil, geotechnical, and bridge/highway engineering design services, as well as transport and pedestrian planning, extreme events evaluation, pedestrian modeling, fire and life-safety review, tunnel ventilation design, and rail coordination for this landmark addition to San Francisco’s infrastructure.

Seismic Solution
The Arup bridge team faced an array of engineering, technical, and community challenges as it served as the engineer of record for the Gerald Desmond Bridge Replacement in Long Beach, Calif. The bridge, which connects the Port of Long Beach to downtown Long Beach and surrounding communities, is a major route for trucking imported cargo inland from the port. The new bridge, which opened in October 2020, is the first long-span, cable-stayed bridge in California, which is one of the most tectonically active regions in the world.

Arup provided an innovative design for the main span and towers where the deck is seismically isolated from the towers through viscous hydraulic dampers. According to Carter, the 515-ft-tall cast-in-place concrete towers are flexible enough to stand by themselves during a major earthquake. “The top of the tower could move as much as 6 ft, with the concrete at the bottom not even cracking,” he explains. The approach spans are cast-in-place concrete box girders that were constructed using a movable scaffold system, the first U.S. application of this innovative construction procedure. (For information on the construction of the approach spans for this project, see the Concrete Bridge Technology article on page 33 of this issue of ASPIRE.)

Carter also says an innovative traffic feature on this project reduced the need for an additional bridge structure, allowing the Port of Long Beach to regain valuable land. A “Texas U-turn,” which is a common feature at intersections in Texas, enables vehicles traveling on one side of a frontage road to make a U-turn without stopping at a traffic signal. The Texas U-turn was introduced to reduce the expense of building (and maintaining) flyover ramps for vehicles entering and leaving the port. This nonstop U-turn is among many key features of the new bridge that facilitate efficient flow of cargo traffic in and out of the port facility.

Looking Forward
Arup faces new challenges going forward. One is digital transformation: making sense of how automation and data-driven design can lead to efficiencies and new products and services.

Another challenge is how to operate responsibly in the shadow of climate change, which threatens to further strain an already ailing infrastructure and increase the risk of natural disasters for communities worldwide.

According to Carter, Arup positions the United Nations Sustainable Development Goals front and center in its business philosophy and strategies. The firm believes climate change should be considered on every project and encourages engineers to participate in discussions about how the world can mitigate and adapt to changes in the environment. By taking this stance, Arup demonstrates that the firm’s ethos and culture still reflect Ove Arup’s core values.

Reference
A Load That’s Hard to Bear

by David Siegel, National Society of Professional Engineers

Preface

For many professional engineers, the bridge collapse at Florida International University in 2018 is a failure that’s still difficult to fully comprehend.

“On Thursday, March 15, 2018, about 1:46 p.m., a partially constructed pedestrian bridge crossing an eight-lane roadway in Miami, Florida, experienced a catastrophic structural failure in the nodal connection between truss members 11 and 12 and the bridge deck. The 174-foot-long bridge span fell about 18.5 feet onto SW 8th Street, which consists of four travel lanes and one left-turn lane in the eastbound direction, and three travel lanes in the westbound direction. Two of the westbound lanes below the north end of the bridge were closed to traffic at the time of the collapse; however, one westbound lane and all five eastbound lanes were open. Eight vehicles located below the bridge were fully or partially crushed. One bridge worker and five vehicle occupants died. Five bridge workers and five other people were injured.”

—National Transportation Safety Board (NTSB) Accident Report, Pedestrian Bridge Collapse Over SW 8th Street, Miami, Florida, March 15, 2018

“Engineers, in the fulfillment of their professional duties, shall: (1) Hold paramount the safety, health, and welfare of the public.”

—National Society of Professional Engineers (NSPE), Code of Ethics for Engineers

Since March 15, 2018, the basic facts of the Florida International University bridge collapse have been assembled and widely reported. The analysis in the National Transportation Safety Board’s accident report has spread far and wide, and there has been a profession-wide struggle to come to grips with the tragedy. That struggle can be seen in the many articles, webinars, seminars, online discussion boards, and chats among colleagues. The question that’s typically asked, like a reflex: What can we learn so something like this never happens again?

Now, two-and-a-half years after the collapse, NSPE members, regardless of discipline, are still trying to make sense of the failure. Reading the NTSB report and other technical analyses don’t necessarily make it easier. Professional engineers are obligated to hold paramount the safety, health, and welfare of the public. In this case, the obligation wasn’t met.

“Screaming Loudly”

At NSPE’s Virtual Professional Engineers Conference in August [2020], NTSB Chairman Robert Sumwalt summarized the investigation’s findings. Three critical errors were identified: (1) the bridge was underdesigned, (2) the peer review was insufficient, and (3) there was a failure to close the bridge to traffic and workers.

The findings note that on the main span truss member 11/12 nodal region, there was visible cracking more than 40 times larger in width than generally accepted cracks for a reinforced concrete structure. Investigators said, based on interviews, that the cracking was the bridge talking to them. “I say the bridge wasn’t talking, it was screaming,” Sumwalt said. “It was screaming loudly that there was something desperately wrong with the design of this bridge and something needs to be done before people die.”

He added: No matter where you sit in the project hierarchy—a new engineering school graduate, a construction manager without a four-year degree, or anywhere else—you can’t stay silent. “If something doesn’t look right, you have a professional and a moral obligation to wave the bullshit flag and say, ‘You know what, I’m just not comfortable with this.’”

The Reaction

Those who listened to Sumwalt’s presentation had different reactions and took away different lessons. Kathy French, PE, drew a comparison to the
Federal Energy Regulatory Commission’s report on the February 2017 failure of the Oroville Dam spillway in California. The incident, brought on by heavy rains, forced the evacuation of more than 180,000 people living downstream. The vice president, environmental, at LS Power in Chesterfield, Mo., says the presentation reinforced a simple but challenging ethos. “Hubris and complacency cannot be allowed in engineering, and it is beholden on all of us in the profession to speak up when something looks or feels off or to foster in those we are managing or mentoring that they also have an obligation to speak up anytime something looks or feels off,” she says. “No one is above having their plans or designs questioned.”

“It’s not just a civil engineering issue...it is a professional issue.”

She finds many parts of the NSPE Code of Ethics for Engineers in the rubble of the collapse, foremost the obligation to hold paramount the public safety, health, and welfare. And, among others, “Engineers shall be guided in all their relations by the highest standards of honesty and integrity” and acknowledge their errors.

One particular Board of Ethical Review case makes Sprague realize that the outcome in Miami could have been different. The case, 98-9, is a simplified scenario of the well-known circumstances surrounding a potential disaster due to the design of the Citicorp Center in New York City.

The facts of the case are similar to the FIU bridge project. The story of the Citicorp Center building features an innovative design and a potentially life-threatening design calculation error that was missed by many engineers before and during construction. The difference, however, was that the Citicorp Center design error was recognized, remedial action was taken, and disaster was averted.

Although the technical details of the two cases are different, the professional issues are not. At the end of the day, professional engineers “want to be respected as professionals,” Sprague says. “Regardless of the discipline, we all have the same code of ethics.”

Breaking Strain
Discussion of the bridge collapse among Society members also took place in NSPE’s online Open Forum. About a month after the NTSB released its final accident report in October 2019, Eric Tappert, PE, of Coopersburg, Pa., started the conversation. Like many members, Tappert wears the stainless steel ring of the Order of the Engineer on the little finger of his working hand. His posting to the Open Forum provided a link to the NTSB report and a link to Rudyard Kipling’s poem, “Hymn of Breaking Strain.”

Tappert volunteers his time to the Order of the Engineer in Pennsylvania, and he notes that the Kipling poem is always read during ceremonies. “The poem highlights the fundamental shortcomings of humans, as opposed to basic physics,” he says. “It is these shortcomings that lead to ethical problems, bad design, and ultimately failures. The poem makes that clear, and the last verse gives us direction for the future.”

Oh, veiled and secret Power
Whose paths we seek in vain,
Be with us in our hour
Of overthrow and pain;
That we—by which sure token
We know Thy ways are true—
In spite of being broken,
Because of being broken
May rise and build anew
Stand up and build anew.

The Order of the Engineer’s roots are connected to the Canadian Ritual of the Calling of an Engineer, which started in 1925 and was written by Kipling. Coincidentally, myth holds that the iron used to make the ritual’s original rings came from the wreckage of the Quebec Bridge, which collapsed during construction in 1907, killing 75 workers.

Other NSPE members shared their thoughts in the Open Forum, too. They saw the root cause in the project’s organizational structure and contractual arrangements, and they noted the possible role of complacency. They also pointed to the Engineers’ Creed pledge, “To place service before profit, the honor and standing of the profession before personal advantage, and the public welfare above all other considerations,” as well as the professional engineer’s standing as a leader who accepts placing the public interest over corporate or personal interest.

“To place service before profit, the honor and standing of the profession before personal advantage, and the public welfare above all other considerations.”

Joseph Englot, PE, of HNTB Corporation, asked whether a whistleblower law could better protect engineers and the public. As the company’s national director of infrastructure security, he
would like to see a law stating that an engineer who observes a serious safety issue and reports it to the appropriate building department official or public owner-agency, regardless of signing a nondisclosure agreement, will not be subject to any penalty from a nondisclosure agreement.

In a later email exchange, he summarized his objective as a professional engineer in light of the bridge collapse: “To ensure that anything that I design must not risk public safety for any reason. In fact, that is why I have been educated and trained in my profession, to ensure that any product I help to develop is safe, as the first requirement.” Once that product is determined safe, he adds, it can be optimized to reduce cost, increase service life, and improve performance.

Reflection

When the strain on the FIU bridge became more than it could bear, it collapsed in less than two seconds. Lost were the lives of an 18-year-old political science student at Florida International University, a 37-year-old worker on the bridge, a father and tower crane technician, a 60-year-old systems technician, and longtime partners who owned a party rental business.

For some professional engineers, the tragedy is still raw, and leaves unanswered questions. Could something like that happen on one of my projects? Kathy French, who has 16 years as a professional engineer, says Sumwalt’s presentation “left me doing some serious reflection if I have been or become dismissive of my associates’ questions or concerns and how to ensure I foster a challenging attitude in my team.”

Sprague, whose resume includes decades of bridge project experience, also reflects on her personal experience. She asks, what would I do? What’s my ultimate responsibility? What about the people I work with? What kinds of checks and balances do we have in our office? The answers are hard to come by. “Could something like this happen at my company?” she adds. “Or would we be humble enough to realize that something’s not making sense and let’s call for help?”

Professional engineers excel at solving technical problems, and in some ways, piecing together the forensic evidence from a failure and reaching a conclusion about what happened may be the easy part. But engineering is a human endeavor, and human behavior and decision-making are not as easy to understand.

“Engineers will never be perfect, but we do need to understand our own failings so we can take steps to minimize their impact.”

To reach better conclusions, perhaps asking the human questions in concert with the technical questions will shed a brighter light. “Engineers will never be perfect,” says Tappert, “but we do need to understand our own failings so we can take steps to minimize their impact. That’s why things like the FIU bridge failure need to be ethics lessons, and the technical stuff is only background for the people story.”

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Guides to Preservation of Highway Bridges

by George Hearn, University of Colorado Boulder

Bridge preservation, as defined by the American Association of State Highway and Transportation Officials (AASHTO), is a program of actions or strategies that prevent, delay, or reduce deterioration of bridges or bridge elements; restore the function of existing bridges; keep bridges in good condition; and extend their life. This concept can also be described as maintenance and repair applied to bridges that are currently in good or fair condition with the intention to keep them in good or fair condition. Preservation keeps bridges in service without modification to bridge capacity, design type, materials, or function.

Bridge preservation can be an important initiative to help agencies reduce costs over the service lives of their bridges. The National Cooperative Highway Research Program (NCHRP) has been working with AASHTO to develop guides to (a) the general preservation of highway bridges and (b) the preservation of highway bridge decks. This article explores some of the concepts and definitions used in the development of the forthcoming guides. Further information on the concept of bridge preservation and detailed information on the development of the two guides to preservation are available in NCHRP Report 950.

Candidates for Bridge Preservation
Candidates for preservation are bridges that meet criteria in condition, capacity, robustness, and durability. Bridge condition is indicated by general condition ratings of the U.S. Department of Transportation National Bridge Inventory and by element-level condition states defined in the AASHTO Manual for Bridge Element Inspection. The capacity of a bridge is a combined assessment of deck width, roadway alignment, clearances, number of traffic lanes, live-load rating, and weight restrictions in relation to desired service for a route. Robustness in a bridge is the absence of vulnerabilities to sudden failure by earthquake, flood, overload, fatigue, fracture, or attack. Durability is an assessment of resistance to deterioration of a bridge's construction materials, design details, and devices such as bearings and joints.

Preservation Actions
Preservation actions maintain good or fair condition of a bridge and extend service life. These actions comprise maintenance and repair of major components of a bridge as well as maintenance, repair, or replacement of other components. Major components are decks, superstructures, substructures, and culverts; other components are joints, bearings, drains, railings, approach slabs, approach embankments, and channel protection.

Preservation is not intended to increase capacity of a bridge; for that reason, actions to replace or reconstruct major components of a bridge are not preservation. Preservation may, however, increase durability. Increased durability may result from new coatings, new overlays, or new joints, and, in general, from actions to repair or replace other nonmajor components.

Benefits of Bridge Preservation
Bridge preservation can reduce agency costs for bridge service. Cost is evaluated using preservation-cycle cost analysis (PCCA). PCCA assumes that every bridge will be replaced eventually, that preservation actions can extend bridge service life, and that costs include the ongoing costs of preservation actions plus the cost of future replacement of a bridge. Longer service life is a reduction in the cost of replacement, and this reduction can be great enough to offset the costs of ongoing actions in preservation. When preservation is applied to bridges in good or fair condition, there can be a net savings in agency costs.

PCCA can be applied to the overall preservation of a bridge, or to preservation of a bridge component. It can also be applied to a network of bridges. At a network level, PCCA yields annual quantities of preservation work and annual costs of preservation. Agencies can develop budgets for preservation and track the delivery of preservation programs using PCCA.

Bridge Preservation Program
Preservation is one phase in overall management of bridges. The service life of a highway bridge is a progression of phases that include new construction, preservation in service, and eventual replacement. Bridge preservation complements existing programs for new construction, and for rehabilitation and replacement of bridges.

Preservation actions respond to defects in bridges and provide repairs. Preservation
programs emphasize cyclic actions that can be taken in advance of defects. The renewal of coatings, overlays, joint seals, and other features that protect major components of bridges can be performed at intervals based on the service life of protective features without waiting for defects to appear.

Guides to Bridge Preservation and Bridge Deck Preservation

NCHRP Project 14-36 developed two AASHTO guides for highway bridge preservation: a general guide to bridge preservation and a guide to preservation of highway bridge decks. The general guide addresses all components of bridges, including bridge decks, whereas the deck guide provides greater detail on materials and methods for preservation of bridge decks. Both guides are based on comprehensive data collection, cost and benefit analyses, and development of catalogs of bridge element preservation actions. Both provide the definition of preservation, criteria to identify candidates for preservation, listed preservation actions, measures of preservation effectiveness, and procedures to monitor and evaluate preservation programs. Examples of application of PCCA to bridges, bridge decks, and bridge networks are also provided.

NCHRP Project 14-36 was completed in 2020, and AASHTO Technical Committee T-9, Bridge Preservation, reviewed and edited both guides. The general guide to bridge preservation was approved by the AASHTO Committee on Bridges and Structures and will be published in 2021. The guide to deck preservation will be considered for publication at the 2021 committee meeting. Detailed information on the development of both guides is available in NCHRP Report 950.

References


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Perspectives on Structural Behavior and Redundancy: Concrete Bridge Behavior and New Materials

by Dr. Oguzhan Bayrak, University of Texas at Austin

My first article on structural, load path, and internal redundancies, published in the Summer 2020 issue of ASPIRE®, started a series of discussions. To further these discussions, my article in the Winter 2021 issue focused on first principles and fundamentals that are of primary importance in the context of redundancy, ductility, and resilience. This article focuses on the behavior of concrete bridges and considerations for incorporation of new materials into design specifications.

Behavior of Concrete Bridge Beams

The American Association of State Highway and Transportation Officials’ AASHTO LRFD Bridge Design Specifications¹ has been calibrated on the basis of element-level behavior. To explore the fundamental aspects of this calibration, my Winter 2021 article focused on flexural response. Let us now focus on an example where we look at the behavior of reinforced concrete beams failing in flexure and shear. Envision classical laboratory tests where simply supported beams are subjected to two-point loading. Figure 1 illustrates a series of four comparable tests resulting in four different failure modes. As shown in the figure, the underreinforced beam experiencing flexural failure displays a substantial amount of ductility, which is demonstrated by increasing deflection at a relatively constant load. Such ductile behavior is desirable so it has some of the highest strength-reduction factors (ϕ-factors) in the AASHTO LRFD specifications.

In cases where the behavior can be classified as tension controlled, prestressed concrete bridge elements (both pre- and post-tensioned) have a ϕ-factor of 1.0, whereas comparable reinforced concrete beams have a ϕ-factor of 0.9. The difference between these two ϕ-factors is rooted in higher quality control and quality assurance associated with prestressed concrete elements, minimal levels of statistical variation in prestressing strand properties, better control of component dimensions or bridge geometry associated with prestressed concrete bridges, and a greater level of variability and construction tolerances associated with cast-in-place, reinforced concrete bridges, as well as a higher variability of material properties in ordinary reinforcing bars. In the context of calibrating ϕ-factors,

EDITOR’S NOTE ON STRUCTURAL BEHAVIORS SERIES

Robustness, redundancy, resiliency, and ductility of concrete bridges are being discussed by Dr. Bayrak in a series of Perspective articles that began in the Summer 2020 issue of ASPIRE®. This series is seen by ASPIRE as an important discussion for the concrete bridge community as it begins to consider new materials. These new materials have properties that differ significantly from conventional materials, which may lead to different element behavior. For example, some of the new materials exhibit ductile behavior, whereas others do not. These differences require new approaches to design, but the framework necessary for establishing these new design approaches is not clearly defined by current design specifications.

It should be noted that not all potential failure modes considered in the American Association of State Highway and Transportation Officials’ AASHTO LRFD Bridge Design Specifications are ductile failure modes, so ductility should not be considered as the only criterion for acceptable bridge designs. For example, concrete breakout failure for embedded anchors is quite brittle, which is recognized in calibration of the applicable equations in the AASHTO LRFD specifications. Furthermore, a more complete understanding of the actual capacity of a bridge is provided when the system-level robustness is considered. Different types of redundancies are inherent in concrete bridges, such as load transfer between girders; these contribute to the overall robustness of the bridge by providing multilayer protection against sudden failure. However, quantifying the contribution of these redundancies is not easy. Current AASHTO LRFD specifications have simplified bridge design by considering only element behavior. Therefore, they do not consider overall system behavior and the redundancies that contribute to it.

The article in this issue focuses on prestressed concrete bridge components reinforced with emerging materials. Some background information is provided on the toughness and ductility of those materials, and the impact these properties have on element behavior. Approaches to including consideration of the behavior of concrete bridge systems will be a topic discussed in a future article in this series.
In the context of calibrating $\phi$-factors, reduced variability in prestressed concrete bridge components (due to all of the aforementioned factors) results in an increase in the $\phi$-factor. Use of steel forms instead of wood forms allowing tight dimensional tolerances, verification of the prestressing force by measuring both force and elongation, and the high emphasis placed on controlling bridge geometry (for example, in segmental bridge construction) all contribute to a higher $\phi$-factor.

The behavior of the overreinforced beam shown in Fig. 1 is quite different from that of the underreinforced beam. Whereas the underreinforced beam displays substantial levels of ductility, the overreinforced beam’s behavior is governed by crushing of concrete before yielding of the flexural tension reinforcement. The AASHTO LRFD specifications make accommodations for this type of behavior (see AASHTO LRFD specifications Fig. C5.5.4.2-1) and explain the behavioral difference and its impact on the $\phi$-factor in Article C5.5.4.2, as follows:

A lower $\phi$-factor is used for compression-controlled sections than is used for tension-controlled sections because compression-controlled sections have less ductility, are more sensitive to variations in concrete strength, and generally occur in members that support larger loaded areas than members with tension-controlled sections.

With this behavioral difference, the specifications require that a $\phi$-factor of 0.75 be used for compression-controlled sections, rather than the 0.9 to 1.0 values for tension-controlled sections. The lower value is used to build in a greater safety margin for compression-controlled flexural failures. In short, if failure is controlled by yielding of reinforcement with minimal variability in material properties and the associated ductile behavior, a higher $\phi$-factor is used. If the behavior is governed by crushing of concrete, which has a higher variability in material response and is coupled with a brittle failure mode, a lower $\phi$-factor is used.

Focusing on shear, we can see that the behavior of a beam failing in shear can be quite brittle if the beam does not have any web reinforcement or contains insufficient shear reinforcement (case 3 in Fig. 1). By providing shear reinforcement, the nature of the brittle failure shown in Fig. 1 (compare case 3 with case 4) is altered. At least two factors contribute to the $\phi$-factor of 0.9 for shear in the AASHTO LRFD specifications: the more ductile shear behavior and a shear strength estimate methodology (the modified compression field theory) that was recognized to be an accurate predictor of shear strength when it was incorporated into the specifications in 1994.

Starting with the eighth edition of the AASHTO LRFD specifications, additional emphasis has been placed in differentiating B-regions (beam or Bernoulli regions) from D-regions (disturbed or discontinuity regions) and using strut-and-tie modeling (STM) provisions for D-regions. That is to say, a great majority of the substructure components and portions of segmental bridges and cable-stayed bridges are designed to comply with the STM provisions of the AASHTO LRFD specifications. Similarly, STM of end regions of pretensioned concrete girders is necessary, especially in cases where stresses greater than 0.18 $f_p$ are to be justified. For STM, the AASHTO LRFD specifications require $\phi$-factors for tension ties that are consistent with the $\phi$-factors of the previously discussed tensioned-controlled cases—that is, $\phi = 0.9$ for reinforcing bars serving as tension ties, and $\phi = 1.0$ for prestressing strands serving as tension ties. For compression members analyzed with STM, the AASHTO LRFD specifications require $\phi = 0.7$. Once again, these $\phi$-factors reflect behavioral differences as well as variability seen in test results.

Conventional and Emerging Materials in Concrete Bridges

Throughout the history of civilization, construction materials and methods have continually evolved. For example, breathtaking ancient stone masonry arches, which continue to serve communities around the world, would be prohibitively expensive to construct today. Bridge engineers try to balance aesthetics, bridge owners’ requirements, and budgets with the needs of the communities that will be served by the bridges they design. Concrete bridges continue to be the preferred solution in most cases due to their resilience, cost-effectiveness, and safety record.

In the early part of the 20th century, cast-in-place reinforced concrete bridges were more common. With the introduction of standardized precast, pretensioned concrete bridge beams, prestressed concrete bridges quickly became the dominant bridge type. Over the years, pretensioned concrete girder sections have been optimized. Segmentally constructed concrete bridges (both cast-in-place and precast concrete) became popular for long spans and have been used by some communities as signature bridges. The
reinforcement have improved for both pretensioned and post-tensioned concrete bridges. Additional sizes and grades of prestressing strands were introduced into the marketplace. More recently, to improve durability of concrete bridges, new materials have been introduced to the marketplace. Stainless steel strands and carbon fiber-reinforced polymer (CFRP) strands are now commercially available.

Figure 2 compares typical stress-strain curves for conventional and emerging types of reinforcement. It shows that Grade 60 reinforcing bars have the highest deformation capacity (rupture strain) of the four materials included in this graphic. Conversely, stainless steel strands and CFRP strands display lower deformation capacities. On the basis of this observation, let us focus on the behavior of Grade 60 reinforcing bars and the Grade 270 seven-wire strands, both of which have been routinely used in reinforced and prestressed concrete bridges. Examining the ultimate strain experienced by these materials, there is a substantial difference (a factor of three to four) between these two materials with respect to ultimate deformation capacity.

Let us now consider the areas under the stress-strain curves for these two materials. The areas are a measure of toughness of the materials. Engineering toughness is the ability of a material to absorb energy before failure. Toughness is typically quantified by integrating the stress-strain curve or looking at the area under the curve. The assumed yield and ultimate strengths of Grade 270 low-relaxation strands are about three to four times as large as corresponding values for Grade 60 reinforcing bars. Without getting into detailed calculations, the areas under the curves, or toughness, for these two materials, which have been successfully and routinely used in practice, are somewhat comparable.

Let us now introduce new materials into this discussion. CFRP and stainless steel strands have comparable levels of toughness with each other, with stainless steel strands enclosing less area under its stress-strain curve.

This observation leads us to a natural question: What is the acceptable level of material toughness to provide acceptable structural response? This question may be followed with another: Should other deformation-based measures be considered at the element, or even structure, level to provide an acceptable structural response? This relates to the ability of a material, element, or structure to deform enough to provide warning prior to any type of brittle failure, as discussed in my article in the Winter 2021 issue. As long as adequate levels of cracking and structural deformation can be observed before collapse, we can conclude that sufficient material toughness, or satisfactory behavior at the element or structure level, exists.

Figure 3 compares the moment-deflection behavior of simply supported prestressed concrete girders reinforced with conventional carbon steel strands and those reinforced with CFRP strands. The ultimate moment and deflection experienced by both types of prestressed concrete beams shown in this figure are nearly the same for this particular case. However, the area under the curve is less for the beam with the CFRP strands than for the beam with conventional strands. This difference can be accommodated in the context of structural redundancy and associated resiliency. If an equivalent area under the curve is desired, the design for the beam with CFRP can be modified to include more strands, compressive strength of concrete can be adjusted, or both can be modified. Therefore, by appropriate design and detailing, we can achieve a similar level of toughness in both cases. The same approach can be taken for designs using stainless steel strands.

The preceding discussion excludes the primary benefit of using CFRP or stainless steel strands. The corrosion resistance of these two materials, when needed, offers tremendous advantages over conventional carbon steel. Where environmental exposure conditions are
advantages over conventional materials to find new materials that offer performance is an important factor related to performance and is a topic that will be discussed in a future article.

In the context of equivalency, there are some parallels with seismic design principles. Figure 4 shows that equivalency between a yielding system and an elastic system can be established on the basis of principle of maximum displacement or principle of maximum potential energy. For more flexible systems, equivalent displacement leads to more realistic approximations of actual response. Although these are seismic behavior and design principles, the parallels between these principles and the preceding discussion are clear. That is to say, the need to look for equivalency between yielding systems and systems that remain linear-elastic has been a challenge both in seismic applications and at the strength limit state for other loading scenarios such as gravity loading.

New Technologies
As we advance concrete bridge construction, we continually seek to find new materials that offer advantages over conventional materials to improve structural efficiency or address a problem. Similarly, the concrete bridge community considers the implementation of new systems to address observed field problems or to advance the state of practice. In this context, and to address tendon corrosion issues related to grouting practices, the post-tensioning industry is considering the use of flexible fillers.

This technology has previously been used in Europe and the Far East but is new to the U.S. market. The goal of using flexible fillers is to improve the in-service performance and durability of tendons. Furthermore, using flexible fillers allows tendons to be replaced, more post-tensioning to be added, or the post-tensioning force to be modified to accommodate new design loads.

An unintended consequence of the use of flexible fillers in bridges is the loss of internal redundancy that we have in grouted systems. In a grouted tendon, if a wire breaks due to an unforeseen reason, the tendon has a chance to redevelop the broken wire over a certain distance from the wire break point. In effect, grout facilitates internal redistribution so that the impact of a wire break at a section can be contained within the immediate vicinity. With flexible fillers, this type of internal redundancy is eliminated. This does not mean we cannot or should not use tendons protected with flexible fillers; their use with external tendons can be appropriate. The use of grouted tendons within girder webs has advantages that relate to the shear strength of the webs and the system redundancy. In short, while new technologies have their place in our tried-and-true concrete bridge solutions, their advantages and potential negative impacts on structural behavior should be thoroughly considered before their adoption.

Final Remarks
Overall, concrete bridges have been performing exceedingly well. We have examples of prestressed concrete bridges built in the 1950s that continue to serve their intended design functions. Many of the early segmental bridges in the United States are still in service and in good shape. From the simplest to the most complex, concrete bridges have gone through a wide variety of laboratory and field testing. It is not a coincidence that our cost-effective concrete bridges continue to serve our communities as originally intended. Many of these systems have been optimized in many ways over the years.

As the bridge engineering community, we must exercise caution to adopt and bring in new materials and technologies. This is not intended to mean that we should not improve our state of practice. It is quite the opposite—we should continuously improve our practice. However, in doing so, we should be careful about intended and unintended consequences of our decisions. In bridge engineering, we do not get something for nothing. For example, implementation of corrosion-resistant materials may be coupled with the use of more brittle materials. The science and engineering that back the concrete bridge industry are quite mature, and we have the necessary tools to scrutinize new materials and technologies during their adoption stage. Once a bridge is in service, field inspection personnel need to be trained for the performance indicators that relate to new materials, systems, and technologies. Frankly, this logic is a part of the simple evolution (as opposed to revolution) we have witnessed in concrete bridges since their worldwide adoption for their cost-effectiveness, maintainability, and overall versatility.

References
When finished, the Wekiva Parkway will complete the State Route (SR) 429 beltway link around Orlando in central Florida, which is intended to relieve traffic and support economic development. The Wekiva Parkway #204 Systems Interchange provides fully directional access between SR 429 and SR 453 and is at the heart of the 10-mile-long corridor.

Eight concrete U-beam bridges were included in the design of the interchange. The design team delivered a unique solution by implementing a spliced, haunched, curved post-tensioned, precast concrete U-beam to improve geometrics and reduce the project footprint. The superstructure alternative came at a lower overall cost to the client than any other available option and required a revolutionary, innovative design. This innovative bridge is the first in Florida to use haunched, curved, post-tensioned, precast concrete U-beams. This approach accommodated the region's growth while protecting the environmental legacy and contributing to a picturesque parkway that emulates the aesthetic features of nearby historic rock walls and stone cladding. (See the Fall 2020 issue of ASPIRE® for an article on another portion of the Wekiva Parkway project.)

**Ramp Geometry**
The design team worked within the constraints of the landscape by matching the vertical geometry to the naturally elevated southwestern section of the project limits to reduce the environmental impact on the region. Noting how the natural grade generally falls off toward the east, the team designed the corridor geometry to work with the landscape to reduce the parkway’s footprint.

The roughly 46-ft-wide ramp K carrying SR 429 epitomizes the project’s innovative bridge design. This 2550-ft-long third-level flyover curves at a 1200 ft radius. The length of the bridge was dictated by the vertical clearance required over ramp M and a limiting mechanically stabilized earth wall height of 30 ft at the approaches. The ramp K structure has seventeen 150-ft-long spans in multispan continuous units.

Haunched precast concrete U-beams were also used without post-tensioning on some of the other bridges in the interchange, which provided aesthetic consistency and value throughout the project. Modified Florida U 48 beams that vary in depth from 4 ft to 6 ft 5 in. were used on the single-span, 73-ft-long ramp J bridge; modified Florida U 63 beams that vary in depth from 5 ft 3 in. to 8 ft 5 in. were used for a two-span, 274-ft 8-in.-long bridge; and modified Florida U 72 beams that vary in depth from 6 ft to 9 ft 7 in. were used on the single-span, 131-ft 6-in.-long ramp L bridge.

**Substructure**
Substructure heights vary from 50 ft for an expansion pier to 17 ft for the integral hammerhead design. End bent sections measure 5 ft wide and 4 ft high. Piles supporting the end bent were staggered to improve the group effect of the deep foundation and the resistance to lateral load. The typical 12 x 6-ft pier column sections are supported by pile footings ranging from 24 to 30 piles at the expansion piers.
The design of integral pier 11 required 35 piles and a 10-ft-tall, 7-ft-wide diaphragm. The cap heights on the expansion piers vary from 5 to nearly 14 ft. The design team chose to use 18-in.-square prestressed concrete piles throughout the foundations to achieve greater economy.

The pier height variability stems from a nearly 5% grade in the vertical curve. This relatively steep vertical curvature allowed the 8% super-elevated two-lane structure to span over Wekiva Parkway. With just over a 17-ft minimum vertical clearance at this location, the structure depth constraints played a key role in the design efficiency. During design, a team often finds itself exploring technically challenging solutions out of sheer necessity. This project was no stranger to obstacles, and pier 11 attested to the timeless proverb “form follows function.” The use of a post-tensioned integral pier cap reduced the project footprint. The design decision lowered the overall level of the interchange (fourth to third level), reducing the project’s impact on the natural environment.

Superstructure

The superstructure of ramp K is composed of variable-depth haunched U-beams that are 6 ft deep at midspan and 9 ft 7 in. deep at the piers. The U-beams feature a constant 8 ft 6 in. center-to-center web spacing at the top of the 10-in.-thick webs. Typically, the bottom flange width varies between just over 4 ft at supports to exactly 6 ft at midspan, with 1-ft 9-in. constant-width top flanges. The haunched geometry in the spliced-girder system serves dual purposes: The more obvious is to blend aesthetically with local signature bridges.
and historical arch structures. The other lies in the system's efficient use of global continuity to reduce structure depth and dead-load moment at midspan and to increase capacity at piers where negative moment demand peaks. The variable-depth post-tensioned design made it possible to span 150 ft with a relatively wide superstructure using only two 72-in.-deep concrete U-beams for the 46-ft-wide ramp.

**Unique Design Solutions**

To fully appreciate the structure's design, one must keep in mind that a simply supported standard 72-in. prestressed concrete Florida U-beam (without post-tensioning) typically spans 155 ft at a 12-ft center-to-center spacing. By incorporating post-tensioning and using continuity at supports, the design team was able to increase the usual 12-ft spacing to 25 ft 6 in. and eliminate a girder line. This presented a new challenge: generating a deck design that satisfied the demand set by the 17-ft 8-in. deck span between webs of adjacent girders. Because a refined deck design with finite element analysis was performed, the resulting slab thickness could be optimized to 10 in. using conventionally reinforced concrete. The exception is the thickness of the deck overhang, which was increased to 10.5 in. when necessary to accommodate 42-in.-tall, F-shape traffic barriers in accordance with Florida Department of Transportation standard plans.

Over-the-road shipping constraints had to be carefully considered due to the 150 ft span and 1200 ft radius of the precast concrete segments. The size limits for hauling the curved precast concrete girder segments were 14-ft maximum width (including curvature for the 1200 ft radius) and 135 ft maximum length. Ramp L girder segments were the longest on the project, at 131.3 ft. Segments were shipped approximately 20 miles from Leesburg, Fla., to the project site.

**Materials**

The moderately aggressive environment required a 2-in.-thick steel reinforcement concrete cover for the superstructure. The top surface of the precast concrete beam flanges used a 1-in.-thick cover because it would be protected by the cast-in-place deck slab. All beams used 270-ksi, low-relaxation post-tensioning strands, and Grade 60 mild reinforcement was used for the entire project. Normalweight concrete was used for all components. The bridge decks required concrete with a typical 4.5-ksi compressive strength; an 8.5-ksi design compressive

Erection of a curved precast concrete U-beam in unit 4 of ramp K. Temporary towers are used to support and stabilize girder segments prior to splicing and post-tensioning. Photo: Ken Zagers.
strength (with 6.0-ksi strength required at transfer) was used for the precast concrete U-beams.

**Post-Tensioning**

The tendon arrangement of the solid rectangular integral pier 11 cap consisted of six tendons centered on the hammerhead cap. A traditional downward concave tendon profile was used to balance the negative moment demand of the pier cantilevers. Each tendon was tensioned to 891 kip and was made up of nineteen 0.60-in.-diameter strands. All tendons were grouted with high-strength, non-shrink grout.

The unit 3 tendon profile is described here as an example of the project’s post-tensioning design. The U 72 girders used a total of two bottom-flange tendons and four continuity tendons in each 10-in. web. Flange tendons were swept and transversely spaced at 2 ft on center over supports. At the balanced cantilevered sections over piers 10 and 11, the slab tendons were anchored at the ends of the precast concrete element. Typical to the unit, slab tendons were anchored at each splice location. The 12-strand bottom tendon profiles characteristically ran along the center of the flange and were tensioned to 522 kip each. The addition, the deepening of the U-beams over the piers increases the sculptural interest of the bridge for everyone passing through the interchange. Finally, the ability of just two lines of beams to carry the ramp simplifies the appearance of ramp K from below.

It is exciting to see the use of an integral pier cap for U-beams. By reducing the height requirement at the interchange’s key clearance point, the overall height of the interchange was reduced, further supporting the interchange-in-a-park theme. This pier cap may have been more expensive than the other caps, but the savings in grading and walls, as well as the interchange’s smaller footprint, offset the higher cost of this one pier. The final result is an attractive driving experience for the residents and tourists in Central Florida that was also the lowest cost design available.

Over-the-road shipping constraints had to be carefully considered due to the 150 ft span and 1200 ft radius of the precast concrete segments. Size limits for hauling the curved precast concrete segments were 14 ft maximum width and 135 ft maximum length. Segments were shipped approximately 20 miles from Leesburg, Fla., to the project site. Cross section of haunched U-beam at the pier is shown at right. Photo: A2 Group Inc. Figure: Atkins.

Interchanges are usually designed solely for the routing of vehicles. Their forms result from the sum of the geometries of their roadways and ramps, and their topography from the automatic application of whatever typical sections are assigned to the roadways. The outcome is generally a mechanistic landscape that looks like nothing in nature. The Wekiva Parkway’s designers took a different approach: they designed the grading between the ramps as an extension of the rolling Florida topography around the interchange. The roadways and ramps look like careful additions to the preexisting natural topography. The curved and lengthened wing walls register as attempts to preserve the existing ground surface. The user’s experience is more like driving through a park than negotiating a high-speed interchange.

Ramp K is the most prominent feature of the interchange, and the innovation involved in its design has paid functional, economic, and aesthetic dividends. The curved and haunched U-beams support the interchange-in-a-park theme by minimizing girder depths at clearance points, thereby minimizing the amount of grading and the length and height of the retaining walls. In
exception was the midspan element between piers 10 and 11, where four-strand spot tendons were added that were tensioned to 174 kip.

The eight 12-strand (0.60-in.-diameter) continuity tendons connected the entire three-span unit. These tendons were typically centered on the web, tensioned to 522 kip each, and anchored at the 3-ft-thick end diaphragms. At the end diaphragms, the top tendon in each web is 18 in. from the top fiber, and the remaining three tendons are vertically spaced at 15 in. on center. At midspan, the tendons are equally spaced 6 in. on center, with the lowest tendon positioned 6 in. from the top of the bottom slab. At piers 10 and 11, the highest tendon is just 4.5 in. from the extreme top fiber of the beam. The three lower tendons are typically spaced at 6 in. on center.

**Conclusion**
This innovative bridge is the first in Florida to use haunched, curved, post-tensioned precast concrete U-beams. The spliced, precast concrete U-beams provided a cost-effective solution that lowered profiles with haunched girders, leading to shorter columns and minimal fill material at the approaches. The project’s aesthetic goals were achieved with an efficient structure and minimal impact to the environment. Traditionally, curved bridge structures are fabricated with steel rather than concrete. Post-tensioned precast concrete increases typical U-beam girder spacing and provides improved durability, while still accommodating over-the-road shipping of the precast concrete girder segments.

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Standing in the heart of New Braunfels, Tex., the San Antonio Street Bridge at the Comal River is a historic arch bridge originally constructed in 1923. The arch spans and girder spans supported a roadway with two lanes, each only 10 ft wide; lacked shoulders; and had narrow 4-ft-wide sidewalks that posed safety risks for pedestrians and drivers. In addition, with the posted weight limit on the bridge, it could not support heavy vehicles, including emergency response vehicles. Like many bridges of its age, the structure needed significant safety improvements to remain operational. With these existing conditions, the bridge was labeled substandard, which qualified it for the Texas Department of Transportation (TDOT) Off-System Bridge Program. This enabled work to begin to rehabilitate the existing bridge.

In addition to the need to preserve the bridge’s historical architecture, another project challenge was that the structure stands in a popular recreational spot. Because the Comal River draws thousands of visitors who tube down the spring-fed body of water, it was important to keep river closure to a minimum.

### Aesthetic Considerations

With community input showing a strong desire to maintain the historical architecture of the existing bridge, designers set out to develop a plan that met necessary safety requirements as well as aesthetic preferences. The existing historic structure was a series of five concrete spandrel arches, each with a span length of 70 ft, and a 30-ft-long approach span on each end. The proposed superstructure replacement design removed the existing deck and floor beams but kept the arches, piers, footings, and most of the spandrel columns to preserve the aesthetics of the original structure. This design called for a shallow superstructure to span between columns at the piers. A modified precast concrete box-beam superstructure with overhangs and finials was developed to resemble the architectural features of the original bridge. In addition, crashworthy railings with a historic appearance were provided, in combination with period light poles at pier locations. A three-dimensional rendering of the proposed scheme helped achieve community buy-in. This design maintained the original appearance by preserving the arches and shorter spandrel columns but removed them from the structural system.

### Condition Assessment

Team members performed a site visit and condition assessment early in the design process. The key objectives of the assessment were to determine construction access, evaluate the structural integrity of the existing concrete, and investigate the bridge for signs of scour, along with the other typical assessment items.

Construction access was critical to the project because crane access was needed for both the superstructure removal operations and placement of the proposed box beams and other elements. Also, because the river is a popular recreation area, potential access restrictions and easements were evaluated to determine the best way to gain equipment access while maintaining public safety and reducing the project’s economic impact on local businesses.

Structural integrity, particularly of the arches, was critical to the design. The
The proposed solution would use the existing arch foundations to support new transverse frame bents that would be connected to the bases of the arches using epoxy adhesive with high-strength dowels. The concrete strength of the arch was assessed using a rebound hammer and found to be between 6000 and 8000 psi, much higher than what had been specified in the 1923 plans. The assessment did not identify any spalls, cracks, or other signs of distress in the arches that would have required repair.

Finally, a scour evaluation of the existing foundations was performed. The existing foundations were spread footings, and, upon investigation, no signs of scour were evident.

**Structural Design**

The selected design used 20-in.-deep precast, prestressed concrete adjacent box beams with a 5-in.-thick cast-in-place (CIP) concrete deck. This superstructure is a standard system in Texas; however, at a 70-ft-span length, it approached the upper span limit for the beam type. The proposed design also included widening the bridge from 30 to 48 ft, making way for substantially wider and safer vehicle lanes and sidewalks on each side of the bridge. The overall structure length was increased from 410 to 450 ft by increasing the approach span length on each end of the bridge from 30 to 50 ft and retaining the five original 70 ft main arch spans. The approach spans were lengthened to avoid the footprint of the existing abutment foundations and to land the bridge where the grade of the ground was more level.

The new transverse bent frames located at the base of adjacent arches were designed as CIP elements. It was important to ensure the proposed dowel bars would not conflict with the existing reinforcement in the arches. Therefore, the design called for removing the surface concrete to expose the reinforcement before drilling the holes for the new dowels. After installation, the dowels would provide a full moment connection for the new frames to the arch bases.

**Construction**

Work for the project began in August 2019, and official closure followed at the beginning of September. The first major task was to demolish the existing deck and remove some of the spandrel columns. The crew completed demolition using a section-by-section method. Each original 70-ft-long arch span comprised seven individual 10-ft-long deck spans. The deck was removed by cutting each 10 ft span loose at the tops of the columns and then lifting the span with a crane. This method minimized the risk of damaging the remaining spandrel columns and arches below.

The contractor proposed several design modifications to accelerate the construction schedule and enhance worker safety. The goal was to allow the river to open for recreational activities earlier than the original December 2020 date. Two alternatives for accomplishing the task called for converting CIP elements into precast concrete.

The CIP transverse bent frame was replaced with a precast concrete frame with semi-pinned connections for the frame-to-arch connection. This alternative took advantage of the repetitive and consistent dimensions of the six frames. In addition, the precast concrete bent frame used a higher class of concrete than its CIP counterpart. The modification had several other advantages.
advantages, the most notable of which was moving the forming and casting of the frames off the critical path. The six frames were constructed and ready for installation as soon as demolition was complete. Their erection took two days. Lastly, the precast concrete bent frames provided an opportunity to alter the moment-resisting system in the frame-to-arch connection. The precast concrete alternative used CIP concrete keys to develop flexural capacity, which was less invasive to the existing arch than drilling and installing multiple high-strength dowels. The post-installed keys also eliminated the risk of fit-up conflicts between the frames and the originally proposed embedded dowels.

To the project team’s knowledge, this is the first time an entire spandrel column/floor beam (spandrel frame) has been replaced with a single precast concrete unit on any historic arch rehabilitation. A second precast concrete alternative developed by the contractor was selecting to precast the deck and overhang on the exterior beams before erection. This option eliminated the need for forming the overhang, which greatly reduced the construction schedule for the deck and improved worker safety.

The precast concrete alternatives introduced their own challenges. The concrete mixture proportions and formwork design had to be carefully considered to mitigate and avoid shrinkage cracking. Considerable attention was also paid to addressing stresses from lifting the bent frames from their casting positions and rotating them vertically. Finally, shipping large and heavy components to the site took extensive coordination among all parties.

Conclusion
This unique superstructure replacement and bridge rehabilitation project used creative precast concrete solutions and maintained the historical character of the existing bridge by preserving the pier foundations, arch ribs, and some spandrel columns. The rest of the spandrel columns and the entire superstructure above the arches—the railing, deck, and floor beams—were replaced, and a new load path was created with precast concrete box beams carrying the deck loading to new precast concrete bents at the piers.

The proposed schedule had the bridge opening to traffic by the end of 2020. With the alternatives proposed by the contractor, the schedule was significantly accelerated, allowing the bridge to be officially opened to traffic on July 15, 2020. Public access to the river for recreational activities was opened on Memorial Day 2020, just 10 months after the project began.

The collaboration of the engineer of record, the contractor, City of New Braunfels, and TxDOT ensured the project was completed successfully, on budget, and ahead of schedule. The widened roadway and sidewalks provide a safer experience for travelers and pedestrians, and with the load posting removed, emergency vehicles can now cross the bridge. The project is a testament to collaboration, innovation, and dedication to the safety and infrastructure of the New Braunfels community.

Nick Nemec is a senior engineer at Modjeski and Masters in Round Rock, Tex., Michael Hyzak is bridge design director for the Texas Department of Transportation Bridge Division in Austin, Jorge Hinojosa is a project manager at Bexar Concrete Works in San Antonio, Tex., and Bill Mayfield is a project manager at Capital Excavation in Buda, Tex.
Sumiden Wire Products Corporation has been supplying prestressing steel to the U.S. prestressed concrete industry for over 40 years, providing high-quality ASTM A416\(^\text{®}\) prestressing strand, ASTM A882\(^\text{®}\) epoxy-coated prestressing strand (see Concrete Bridge Technology article in the Spring 2020 issue of ASPIRE\(^\text{®}\)), and, more recently, ASTM A1114\(^\text{®}\) stainless steel prestressing strand (see two Concrete Bridge Technology articles in the Spring 2018 issue of ASPIRE\(^\text{®}\)).

In 2019, Sumiden Wire became aware of a concerning development in the U.S. steel-fiber supply chain for Buy America–compliant ASTM A820\(^\text{®}\) Type I chopped steel fibers for ultra-high-performance concrete (UHPC). The primary supplier of these Buy America–compliant chopped steel fibers announced its intention to exit the market, creating a supply problem for those working to advance UHPC technology in the bridge industry. (For details on Buy America requirements see the Perspective article in the Fall 2020 issue of ASPIRE\(^\text{®}\).) Because Sumiden Wire is a member of a global network of steel wire manufacturing companies, it was able to reach out to an overseas sister plant and gather the necessary technical know-how to quickly enter this market. As of January 2021, Sumiden Wire has officially started production of 0.2 mm diameter \(\times\) 13 mm long (0.008 \(\times\) 0.5 in.), high-tensile strength (greater than 2800 MPa \([400\text{ ksi}]\)), brass-coated ASTM A820 Type I steel fibers in its centrally located Dickson, Tenn., manufacturing plant. These fibers are currently offered in 20-kg (44-lb) bags.

At present, Sumiden Wire’s capacity to supply steel fibers is about 360 metric tons (800,000 lb) per year. The manufacturing systems are modular and can be expanded easily if marketplace demand supports the additional investment. Currently, the focus is on supplying the 0.2 \(\times\) 13 mm (0.008 \(\times\) 0.5 in.) fiber product. Variations on the standard product can be easily accommodated if other fiber lengths are needed.

As a long-term U.S.-based supplier to the U.S. prestressed concrete industry, Sumiden Wire has been fortunate to witness firsthand the evolution of prestressing steel reinforcement technologies. Whether the changes were increasing tensile strengths, the evolution from stress-relieved to low-relaxation strand, or the more recent developments in corrosion-resistant prestressing technologies, Sumiden Wire has participated in each of these phases of the industry’s development. Now, with the continued research, development, and commercialization of UHPC mixtures and technologies, the company is excited to participate as a supplier of the steel fibers critical to this innovative technology.

As a further expansion of its product line and to meet marketplace demand for higher-strength prestressing steels, Sumiden Wire is now also able to supply the 0.6-in.-diameter Grade 300 strand shown in Table 1. To achieve this strand’s higher-strength, Sumiden Wire currently supplies this product using a combination of higher-strength wire rods as a raw material, in addition to increasing the actual area to greater than the area of the normally supplied 0.6-in.-diameter strand, while still staying well within the overall diameter tolerances specified in ASTM A416. Due to limited demand, this product is not currently kept in inventory; it is produced when an order is received. The addition of this higher-grade strand is well timed to meet expected future demand as state departments of transportation, such as the Minnesota Department of Transportation, start specifying this grade.\(^3\)

Table 1. Material properties for 0.6-in.-diameter Grade 300 prestressing strand

<table>
<thead>
<tr>
<th>Nominal Diameter, in.</th>
<th>Grade, ksi</th>
<th>Diameter Tolerance, in.</th>
<th>Minimum Breaking Strength, lbf</th>
<th>Minimum Yield Strength, lbf</th>
<th>Nominal Area, in.(^2)</th>
<th>Relaxation at 1000 hr</th>
<th>Nominal Weight, lb per 1000 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.6</td>
<td>300</td>
<td>±0.026, -0.006</td>
<td>65,100</td>
<td>58,590</td>
<td>0.217</td>
<td>&lt;2.5% at 70% GUTS (guaranteed ultimate tensile strength)</td>
<td>740</td>
</tr>
</tbody>
</table>

Table: Sumiden Wire.

In January 2021, Sumiden Wire began producing 0.2 mm diameter \(\times\) 13 mm long (0.008 \(\times\) 0.5 in.) high-strength steel fibers for ultra-high-performance-concrete mixtures. Photo: Sumiden Wire.

addition of this higher-grade strand is well timed to meet expected future demand as state departments of transportation, such as the Minnesota Department of Transportation, start specifying this grade.\(^3\)

References


Jon Cornelius is the executive vice president and general manager of the PC strand division for Sumiden Wire Products Corporation in Dickson, Tenn.
**PCI’s New Architectural Specification Program and Its Effect on the Bridge Construction Community**

by Randy Wilson, Precast/Prestressed Concrete Institute

PCI’s architectural precast concrete certification categories are about to change. How will that affect the bridge producers whose plants are PCI certified under the existing BA (bridge products with an architectural finish) category? The simple answer is that producers who deliver bridge components conforming to PCI-MNL-116, *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products*, and produce precast concrete components with an architectural finish should be certified in the new AD category, while also maintaining their current bridge product category (B1 to B4). This article focuses on the impact of the new AD category, which applies to heavy structural components that are produced from standard steel molds, rather than highly complex custom-molded components covered in other categories.

First, we should answer two questions: “Why the change?” and “What are the options?”

**Why the Change?**

Making an engineer’s or architect’s vision come to life can be both exhilarating and challenging. What is easily developed in the design stage is often difficult to achieve in reality. Designer creativity and advanced production techniques are expanding the expressive potential of architectural precast concrete. In response, and through collaboration between the design community and the precast concrete industry, PCI’s Architectural Certification Program is expanding with new categories that are more specific to the various construction markets that use the benefits of precast concrete solutions. These new categories align the designer’s expectations with the demonstrated capabilities of the precast concrete producer and the components required for the project.

Most bridge components perform a utilitarian or structural purpose first and an aesthetic purpose second. As such, there are certain limitations to color, shape, texture, and tolerances compared with a purely architectural or facade panel with custom forming, complex shapes, and tight tolerances. However, more communities are realizing that bridges contribute to the community’s culture by reflecting the local architecture. The visual interest of a bridge (texture, color, function, and form) is a key aspect discussed in Frederick Gottemoeller’s “Aesthetics Commentary” in every issue of *ASPIRE®*. Using PCI manuals, owners, architects, engineers, contractors, and precast concrete producers can define acceptable product standards for structural performance, connection types, tolerances, and other quantifiable characteristics. However, aesthetic traits are often determined by one person’s or one group’s assessment of appearance-related criteria, including uniformity.

**What Are the Options?**

There is currently a wide variety of PCI-certified precast concrete plants across North America. These plants have different production techniques, different levels of technology, and different product focuses, and are grouped into A (architectural), B (bridge), and C (commercial) categories. Expanding the architectural certification program with four new categories, AA, AB, AC, and AD, better aligns precast concrete producers’ capabilities with designers’ expectations. The PCI architectural certification category requirements (Table 1) provide an overview of the producer’s capabilities that are required for each category.

The requirements do not necessarily represent the full extent of an individual producer’s capabilities. Categories AA, AB, and AC align with PCI-MNL-117, *Manual for Quality Control for Plants and Production of Architectural Precast Concrete Products*, which is rarely used for bridge products. Category AD aligns more closely with the bridge and commercial markets that produce components in accordance with MNL-116 (structural concrete).

Figure 1 shows example mock-up panels for the PCI architectural certification categories. Category AD is for the most simplistic panels—flat...
panels with features that may include a one-colored, single-textured face, reveals, or thin brick, and with tolerances per PCI-MNL-116. Panels in category AC encompass the features of category AD with added single-cast returns and more stringent tolerances per PCI-MNL-117. Category AB is for panels that are more complex than those in category AC. An AB panel may incorporate sequential returns, multiple colors or textures, a three-dimensional formed surface, or embedded material such as stone or tile, or be a curved panel. Finally, panels in category AA have the same features as AB panels but with more stringent tolerances for production and erection.

PCI-MNL-135, Tolerance Manual for Precast and Prestressed Concrete Construction, is being updated to reflect the changes in the new architectural certification categories and is currently moving through the review process. Until it is released, the current edition will be used along with the tolerances specifically listed in the supplemental conditions for the new certification program and the new architectural guide specifications that are scheduled to be published in September 2021.

The certification categories require the precast concrete producer to demonstrate specific manufacturing capabilities, achieve specific aesthetic finishes, and produce finished mock-up panels that meet specific tolerances defined for each category. The auditing process will verify the complexity level achievable by the specific precast concrete plant.

Bridge project specifiers who elect to have precast concrete components with complex shapes, multiple colors, custom formliners, and other specialty finishes, should consider specifying the AA, AB, or AC categories and should consult with local PCI-certified plants that are qualified under the B1, B2, B3, or B4 categories, as appropriate. Specifiers are strongly urged to reach out to PCI or PCI-certified producers to understand the precast capabilities.
concrete producer’s specific manufacturing capabilities and establish category-specific limitations, before including these PCI architectural precast category requirements in concrete bridge project specifications.

**When Is This Change Happening?**

Beginning October 1, 2021, the A1, BA, and CA categories will be discontinued. All references to these categories should be removed from the specifications and replaced with the new architectural categories. To be clear, the existing bridge certifications categories (B1, B2, B3, and B4) will remain unchanged.

All PCI-certified producers who are using category A1, BA, or CA must declare a new category via the PCI certification application process. That process includes producing three mock-up panels to demonstrate capabilities, with one of those mock-ups being produced in the presence of an auditor. There will be no default or grandfathering of certification categories. Every architectural precast concrete producer that is currently PCI certified must be recertified under the new program to obtain an architectural group certification.

All precast concrete projects involving architectural treatments bidding on or after October 1, 2021, should include one of the new categories when specifying any precast concrete component with an architectural appearance. It will be imperative for precast concrete producers to work diligently with specifiers during design and during the bidding process to ensure the proper certification categories are specified.

**Conclusion**

The new specification categories provide value to various construction industry stakeholders by aligning the PCI-certified precast concrete producer’s proven aesthetic capabilities with project requirements. They also provide a format for the precast concrete producer to discuss specific capabilities, establish realistic expectations, and avoid costly discrepancies, thereby reducing project risk for all parties.

**References**


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**Randy Wilson** is the director of architectural precast systems for the Precast/Prestressed Concrete Institute in Chicago, Ill.
The opening of the new Gerald Desmond Bridge in October 2020 signified a major upgrade for the Port of Long Beach, Calif. As a key link in the port’s infrastructure, this bridge plays an important part in the nation’s economy as a whole—it is estimated that 15% of U.S. imports pass over, and likely under, the bridge. With a vertical clearance of 205 ft over the water, the new bridge provides a significantly larger navigation envelope for modern shipping needs. Although easily overlooked, cast-in-place concrete box girders have been constructed on each side of the channel to make the long climb to the 1000-ft-long main span cable-stayed bridge, which is the centerpiece of the project.

The use of post-tensioned concrete box girders as the workhorse structures on the approaches was a natural fit for this project. The California Department of Transportation has used this type of construction extensively, and the structures have a reliable history standing up to two things California is famous for: traffic and earthquakes. It is for these reasons that post-tensioned concrete was used in the design-build proposal from Arup and SFI (a joint venture among Shimmick Construction, FCC Construction, and Impregilo S.p.A.) in 2012.

Hand-in-glove with box-girder construction has been the industry’s use of what many call “California” (conventional) falsework. To the casual observer, these temporary works can appear baffling, but they are a well-understood and economical means of bridge construction on the West Coast. This type of falsework, however, is not practical for heights over 120 ft, which led the team to consider other methods for the tallest portions of the approach structures. They selected a movable scaffold system (MSS) as a means to continue conventional box-girder construction to the height of the main-span bridge.

MSSs are self-launching equipment used to support formwork and concrete for the span-by-span construction of bridges; such systems are commonly used in the construction of long, straight viaducts in Europe and Asia. Whereas launching gantries are not uncommon for the construction of precast concrete segmental bridges, there are no recent examples of MSS projects for cast-in-place work in the United States.

On this project, the details of this construction method were heavily influenced by the design requirements, and seismic considerations were an obvious driver in the bridge design. To that end, integral column connections were essential for seismic performance. As a result, the approach units were configured as integral frames having two to four spans of approximately 230 ft per span. Expansion and contraction between units was allowed at quarter-span hinges. This arrangement limits the amount of movement and bending in the integral columns due to creep, shrinkage, and temperature effects.

Due to schedule and logistics, the project required two MSS units; these were designed and supplied by Strukturas AS. One unit was put to work on the west approach, the other on the east.

The MSS operations required close coordination with the superstructure design. For example, the direction of construction dictated the location of construction joints and hinges, and ultimately had a strong influence on

Rigging assembly for erection of the brackets under the movable scaffold system. Each bracket supported a load of 3000 kip, which was transmitted to the pier wall by a shear lug on the bracket. Photo: SFI.
The movable scaffold system (MSS) was arranged in an underslung configuration, where the launching girders straddle the pier columns below the superstructure. With an empty weight of 3000 kip, each MSS could support approximately 7000 kip of concrete. At the lead pier, the two main girders were supported on brackets anchored on the sides of the pier columns, avoiding the need for temporary columns to the footings. Each bracket supported a load of 3000 kip, which was transmitted to the pier wall by a shear lug on the bracket. This detail required a temporary opening in the pier wall, which was patched after the span was complete and the bracket removed.

For concrete placement, the rear of the MSS was supported near the quarter span by a “gallows” beam. This beam transferred the weight of the MSS and concrete to the webs of the box girder near the construction joint to hold the formwork tight to the completed section. For concrete placement, the rear of the MSS was supported near the quarter span by a “gallows” beam. This beam transferred the weight of the MSS and concrete to the webs of the box girder near the construction joint to hold the formwork tight to the completed section. Similar to traditional box-girder construction, the spans were constructed and cast in two stages. The bottom slab and webs were cast first and partially post-tensioned before the deck slab was constructed. As with conventional post-tensioned box girders, the majority of the tendons were located in the webs and were draped between piers. With the construction joints near the quarter point of the span, where bending demands are modest, the PT anchors were spaced over the depth of the web in full-height blisters to allow access for tensioning from inside the box girder. At these blisters, tendons from adjacent spans overlap, making the tendon profile continuous from span to span.

For the deck concrete placement, the slab over the interior cell of the box girder was supported using shoring similar to that used in multistory buildings, which was the post-tensioning (PT) profile. For continuous superstructures using MSS, superstructure concrete is placed from the quarter point of one span to the quarter point of the next. This is a normal practice for many structure types because it takes advantage of an inflection point, and it had the benefit on this project of minimizing bending demands in the MSS. With the construction method having a direct influence on the design, the engineer and contractor had to commit to the construction sequence early in the planning.
The launching girders varied in configuration and capacity along their lengths based on demands and function. The center section of each launching girder consisted of a heavy box girder to support the concrete span. At each end, the girders transitioned to a lighter truss configuration for launching. With the nose and tail required to launch 230 ft from pier to pier, the MSS measured over 500 ft long.

To support and access the MSS, assembly and disassembly were done on the ground. Each MSS unit was delivered in approximately one hundred 40-ft shipping containers and then the pieces were bolted together in the footprint of the bridge. McNary Bergeron & Associates provided construction engineering for the temporary works associated with lifting the assembled MSS into position. With the equipment assembled on the ground, strand jacks supported on temporary frames at the top of the pier columns were used to lift the MSS to height at a rate of 20 ft per hour over the duration of a work shift. Because the MSS obstructed all crane access from overhead, the support brackets were then positioned underneath for a similar lift using strand jacks.

Due to the overhead constraints, the brackets were lifted in pairs using a rigging system resembling a marionette puppet. Each bracket was equipped with rubber wheels to guide it up the face of the column. At the top of the lift, the wheels were retracted to place the shear lugs into the pockets on the sides of the column. Bars in the top chord of the bracket were then tensioned to hold the entire assembly tight to the column, and the MSS was lowered onto the launching wagons and brackets. After spans were completed, the brackets were taken down using similar methods and cycled ahead to the next pier.

Because the approach spans consisted of two girder lines, each MSS unit made two runs. After the first run, the equipment was lowered and moved to the next heading using self-propelled modular transporters.

**Conclusion**

With the upfront cost of the equipment, MSS is most viable on projects with a significant number of consistent span lengths and geometry. It is also a very linear method of construction that requires close coordination with the design and is best suited for design-build or alternate delivery methods.

With those constraints met, this project demonstrates that, in the right conditions, MSS is an efficient method for the construction of box-girder bridges. During the project, each MSS unit produced one span every two to three weeks, translating to approximately 90 ft of new bridge per week per heading. More importantly, the system provided an option where conventional falsework was not practical.

Compared with other construction methods, MSS has unique advantages. Geometry control and related engineering oversight are greatly simplified because spans are constructed in a single go. Similarly, the system minimizes construction joints and construction-related openings in the deck. Lastly, in the specific example of this project, MSS construction proved to be well adapted to seismic design details, including integral piers, hinges, and heavy longitudinal reinforcement.

The completion of the Gerald Desmond Bridge is a great achievement for everyone involved in the project, and the use of MSS represents the design-build team’s resourcefulness in pairing industry expertise with new methods to achieve great goals.

Jeremy Johannesen is a principal with McNary Bergeron & Associates in Broomfield, Colo.
Load Rating Segmental Bridges
by John Corven, Corven Engineering Inc., a Hardesty & Hanover Co.

Precast and cast-in-place concrete segmental bridges have become an important part of our country’s bridge inventory. The complexities of sophisticated designs, geometries, and construction techniques are more than offset by the adaptability and positive enduring impact of these bridges. Increased sophistication also means greater challenges when load rating segmental bridges. A few of these challenges are discussed in this article.

Level of Effort
Rating a segmental bridge takes more effort than rating a more conventional bridge type. The reason is that segmental bridge load rating needs to be based on an accurate estimate of stresses along the entire length of the bridge at the time for which the rating is performed. Specialized construction analysis software is required to estimate the state of stress according to the contractor’s construction methodology and sequence. The time-dependent nature of the concrete and prestressing steel is modeled to predict changes in stress with age, as well as the stress redistribution that results from changing statical schemes during the erection process.

The effort can be reduced if the owner has retained original analysis models that accurately reflect the as-built conditions of the bridge. Projects with less documentation may require substantial work to build models that accurately reflect the as-built bridge and the contractor’s construction sequence and erection methodology. In some cases, an in-depth search through construction documents is required to find needed information. Data that are typically collected include:

- Concrete material characteristics (strength, density, modulus of elasticity, creep, shrinkage)
- Prestressing steel and system characteristics (modulus of elasticity, relaxation, friction, wobble)
- Key dates (segment casting, segment erection, span closures)
- Tensioning data (dates, forces, elongations)
- Erection manuals (means and methods)

It is good practice for the owner to collect and catalog this information when building a new bridge to facilitate future load ratings.

Segmental bridge load ratings also take more effort because six different behaviors need to be rated. In the longitudinal direction, a bridge is rated with regard to flexural stresses, flexural strength, web principal tensile service stress, and web shear strength. The transverse cross section of the bridge is rated with regard to flexural stress and flexural strength. Each of these behaviors is rated for design loads, legal loads, and permit loads at both inventory and operating levels at all segment joints. The number of calculations is large, but effective use of spreadsheets or other software helps manage the effort. Recently, we performed load ratings of the new Interstate 20/Interstate 59 Central Business District Bridges in Birmingham, Ala. The load rating of this 172-span, 1-million-ft² bridge required over 160,000 individual rating evaluations. (See the Spring 2020 issue of ASPIRE® for details of this project.)

Service Limit State Calibration
Segmental bridge design criteria have matured over time from project-specific design criteria to guide specifications paired with load factor design (LFD) criteria to full inclusion in the current American Association of State Highway and Transportation Officials’ AASHTO LRFD Bridge Design Specifications.¹
Load-rating methodologies for segmental bridges likewise have developed over time. However, these methodologies have lagged behind design codification because of lack of calibration at service limit states.

The reliability-based AASHTO LRFD specifications were calibrated at ultimate strength with the load factor design requirements of the AASHTO Standard Specifications for Highway Bridges.\(^2\) No calibration was performed for service limit states or for fatigue and fracture limit states. These limit states were accommodated by factoring capacities such that member proportions produced under the new LRFD code would be comparable to those designed using the AASHTO standard specifications. Segmental bridge superstructures, though verified at all appropriate strength limit states, are fundamentally structures that are designed for service conditions. The lack of calibration at service limit states becomes problematic for load rating segmental bridges when the change between inventory (service or design load level) ratings and operating ratings (maximum permissible live load level) is expressed by a probabilistic measure (reliability index \(\beta\) changes from 3.5 to 2.5).

Calibration for the service limit state was investigated by the Florida Department of Transportation (FDOT) in 2002, with work performed by Dr. Dennis Mertz and Corven Engineering.\(^3\) This work led to the use of design lanes for inventory ratings and striped lanes for operating ratings. This philosophy is now incorporated into the AASHTO Manual for Bridge Evaluation.\(^4\) Unfortunately, the calibrating work performed by FDOT focused on what was a common segmental cross section in its inventory: three design lanes with two striped lanes. Care needs to be taken for other superstructure cross sections. For example, narrower cross sections with two design lanes and one striped lane tend to overstate the operating rating factors, while wider cross sections, designed for four lanes and striped for three, tend to underestimate the operating rating factors.

The good news on this front is that National Cooperative Highway Research Program Project 12-123, “Proposed AASHTO Guidelines for Load Rating of Segmental Bridges,” will soon be underway. Unfortunately, the results of the project will not be available for several years. In the meantime, some have used a sliding scale normalized around the FDOT calibrations to try to maintain target reliabilities over a range of bridge widths at operating conditions.

Rating Older Bridges

Segmental bridges that were designed under older specifications often will not rate at inventory level using load and resistance factor rating (LRFR) methodology. The reason for this typically lies in three areas: (1) the change from HS20 loadings of the AASHTO standard specifications to the notional loads of the AASHTO LRFD specifications without service limit state calibration (as discussed in the previous section), (2) changes in shear strength design provisions, and (3) previously unspecified limits on principal tensile stresses in webs. It is important to communicate possible methods of mitigating low ratings to owners who do not want to revert to the load factor rating or allowable stress rating methods permitted in the Manual for Bridge Evaluation.

Transit and Heavy Rail Bridges

Segmental construction has proven to be excellent for transit and heavy rail bridges. Particular care should be taken when asked to perform an LRFR of a transit bridge designed under an owner’s design criteria that may be LFD based. The reason is again one of calibration. Using owner-defined loads that have remained unchanged with load and resistance factors calibrated to highway vehicles, as in the LRFR, may lead to inappropriately high load ratings at some limit states.

Final Thoughts

Precast and cast-in-place concrete segmental bridges have robust load-carrying capacities. With a proper understanding of loads, design methods, and load and resistance factors, the accuracy of load ratings can be improved, which will allow owners to maximize the use of their segmental bridges as traffic needs change.

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John Corven is director of complex bridges at Corven Engineering Inc., a Hardesty & Hanover Company, in Tallahassee, Fla.
The Precast/Prestressed Concrete Institute (PCI) published a new "Recommended Practice to Assess and Control Strand/Concrete Bonding Properties of ASTM A416 Prestressing Strand" in the November–December 2020 issue of the PCI Journal. This recommended practice culminates many years of research evaluating test methods as well as test repeatability and reliability to provide uniform criteria for the precast concrete industry to use when specifying strand for bonded, pretensioned applications. An article in the same issue of the PCI Journal and an article in the following issue summarize the background research on strand bond, current design provisions, and the development of the recommended practice.

Current Design Provisions
The ninth edition of the American Association of State Highway and Transportation Officials’ AASHTO LRFD Bridge Design Specifications provides provisions for the development length of bonded strand in Article 5.9.4.3.2, as follows:

\[ l_d \geq \kappa \left( \frac{f_p}{3} - \frac{2}{3}f_{pe} \right) d_b \]  


where

- \( l_d \): development length in tension of pretensioned strand (in.)
- \( \kappa \): 1.0 for pretensioned panels, piling, and other pretensioned members with a depth of less than or equal to 24.0 in.; 1.6 for pretensioned members with a depth greater than 24.0 in.
- \( f_p \): average stress in prestressing steel at the time for which the nominal resistance of the member is required ksi
- \( f_{pe} \): effective prestress in the prestressing steel after losses (ksi)
- \( d_b \): nominal strand diameter (in.)

The development length consists of the sum of the transfer length, which is the length of embedded pretensioned reinforcement required to transfer the effective prestress to the concrete, and the bond length, which is the length of bonded pretensioned reinforcement required to develop the effective prestress used for the strength limit state design. The AASHTO LRFD specifications, Article 5.9.4.3.1, states the transfer length may be taken as 60\( d_b \).

PCI Recommended Practice
The "Recommended Practice to Assess and Control Strand/Concrete Bonding Properties of ASTM A416 Prestressing Strand" establishes two values for ASTM A1081 pullout strength: a minimum value and a high-bond value. For the minimum value, the required average pullout strength is 14,000 lb for 0.5-in.-diameter strand. For the high-bond value, the required average pullout strength is 18,000 lb for 0.5-in.-diameter strand. For other strand diameters and strengths, the pullout value is based on a ratio of diameter and specified tensile strength. Thus, for 0.6-in.-diameter Grade 270 strand, the values are 16,800 and 21,600 lb for minimum-bond-strength and high-bond-strength strands, respectively.

The recommended practice provides the following development length equation intended to replace the current AASHTO LRFD specifications equation:

\[ l_d = \left( \frac{120}{f'_{ci}} + \frac{225}{f'_{ic}} \right) d_b \geq 100d_b \]

where

- \( f'_{ci} \): concrete strength at transfer (ksi)
- \( f'_{ic} \): design concrete strength (ksi)

The coefficients in the numerators of the preceding equation differ from those published in the recommended practice because they have been converted for use with the AASHTO LRFD specifications, which use kip per square inch (ksi) for concrete strengths rather than pounds per square inch (psi). The change from calculating development length based on strand stress to using concrete strengths is based on research described in National Cooperative Highway Research Program (NCHRP) Report 603, where the preceding equation is given as Eq. (4.5).

Assuming a concrete strength at transfer of 4.5 ksi, design concrete strength of 6.5 ksi, and 0.5-in.-diameter strand results in a development length of 72 in., approximately equal to the current development length assuming \( f_p = 258 \) kpsi and \( f_{pe} = 170 \) kpsi. Furthermore, the recommended practice removes the 1.6 multiplier, which results in shorter development lengths than the current AASHTO LRFD specifications equation for typical bridge girders and members greater than 24 in. deep. Figure 1 illustrates the effect of the recommended practice on development length for 0.5-in.-diameter strand as the concrete strength varies.

The recommended practice also provides the following transfer length equation intended to replace the current AASHTO LRFD specifications provisions:

\[ l_t = \kappa \left( \frac{120}{\sqrt{f'_{ci}}} \right) d_b \geq 40d_b \]
stresses ($K = 0.8$), transfer lengths are reduced from the current 60$\text{db}$ with the $K$ value less than 1.0, which results in higher calculated fiber stresses at the end of the transfer length near the member’s ends.

**Impact on Design**

AASHTO Technical Committee T-10 is considering whether to implement the PCI recommended practice into the AASHTO LRFD specifications. The modified development length equation, while not a trivial change, will have a marginal effect on the design of most current prestressed concrete components for service limit state and strength limit state flexural checks. Components that may have previously required additional reinforcement to develop flexural strength within the development length may now be considered adequate. The shorter development length for members greater than 24 in. deep could have some effect on the strand length that a designer chooses to debond.

For minimum-bond-strength strand ($K = 1.6$), all common concrete strengths (anything less than 10 ksi) would result in an increased transfer length compared with the current 60$\text{db}$. For high-bond-strength strand ($K = 1.0$), the transfer length of strand in concrete with a compressive strength at transfer of 4 ksi is equal to the current 60$\text{db}$, but the transfer length is reduced for higher concrete strengths. For calculation of concrete stresses ($K = 0.8$), transfer lengths are reduced from the current 60$\text{db}$ with the $K$ value less than 1.0, which results in higher calculated fiber stresses at the end of the transfer length near the member’s ends.

**Effect on PCI Plant Certification**

Effective July 1, 2020, the PCI Plant Certification Program approved an
addendum to the *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products* (MNL-116) requiring at least annual ASTM A1081 test results from the strand supplier for all strand sizes in use at the plant. PCI committees are currently working on additional guidance following the new recommended practice requirement of quarterly ASTM A1081 test results for 0.5-in.- and 0.6-in.-diameter strands.

Additionally, Section 5, Quality Assurance, of the recommended practice states, “Through appropriate testing, it is recommended that the precast concrete producer confirm that the strand and concrete combination in use produces product performance that meets or exceeds predicted behavior.” However, a particular test is not specified. Possible test methods include the flexural beam test, the Peterman beam test, the Moustafa pullout test, the direct pull test, and strand draw-in measurements for sawed ends. Most of these methods require additional specimens for the strand and concrete combination, while strand draw-in measurements provide direct measurements on a fabricated component without additional specimens. Additional guidance and commentary to further clarify the intent of the recommended practice is currently being developed.

**References**


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**PCI Offers New eLearning Modules**

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

The PCI eLearning Center is offering a new set of courses that will help an experienced bridge designer become more proficient with advanced design methods for precast, prestressed concrete flexural members. There is no cost to enroll in and complete any of these new bridge courses. The courses are based on the content of the 1600-page PCI Bridge Design Manual, now available for free after registering with a valid email. While the courses are designed for an engineer with 5 or more years’ experience, a less experienced engineer will find the content very helpful for understanding concepts and methodologies.

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

All courses on the PCI eLearning Center are completely FREE.
During construction of a single-span precast, prestressed concrete girder bridge for the State Highway 130 project, both exterior beams rotated outward approximately 2.5 degrees during slab placement and lost full contact with their neoprene bearings (Fig. 1). The girders were 34-in.-deep Texas Department of Transportation (TxDOT) Type B beams, which are no longer used for new construction by TxDOT. The overall deck width was approximately 61 ft; there were nine beams in the cross section spaced at 6 ft 10½ in.; and the exterior fascia beams supported a 3 ft deck overhang, as measured from the beam centerline.

This troubling performance occurred despite the beam erection and slab placement bracing being installed in accordance with plan requirements and specifications. The plans called for temporary timber X-bracing spaced at 30 ft maximum for the approximate span length of 65 ft. Each temporary timber X-brace was coupled with a no. 5 reinforcing bar welded to the shear reinforcement that extended from the tops of the beams, to form a tension tie across the bridge width. These braces were key elements of TxDOT’s previous standard details for erection bracing and slab placement and were intended to address beam stability for wind loading and the torque caused by overhang brackets loaded with a slab screed and finishing machine.

With TxDOT’s widespread use of precast concrete partial-depth deck panels for deck construction, there is a complication in the installation of the tension ties between girders, which are no. 5 reinforcing bars welded to the projecting shear reinforcement. These top tension ties are installed during beam erection, then later removed as needed to allow installation of the 4-in.-thick precast concrete deck panels. The ties are then reinstalled after panel installation is

Figure 1. During construction of a single-span precast, prestressed concrete girder bridge for the State Highway 130 project, outward rotation of fascia beams resulted from deck construction. This photo was taken after removal of erection and slab placement bracing. All Photos and Figures: Texas Department of Transportation.
welded after precast concrete partial-depth deck panel installation

• Neoprene bearings with limited axial and rotational stiffness

Finite element analysis of the State Highway 130 bridge indicated that the beams leaned outward as a rigid body and that the timber X-bracing was mostly ineffective in preventing the observed fascia beam rotation. The Project 5706 report recommended horizontal bottom-flange bracing as a more effective restraint against fascia girder rotation, still coupled with the no. 5 tension tie at the top. The horizontal bracing is also significantly easier to install than X-bracing.

Research

The poor performance of the previous standard details led to a TxDOT-funded research project to better understand the problem and develop solutions. Project 5706, “Impact of Overhang Construction on Girder Design,” led by Dr. Todd Helwig at the University of Texas at Austin, investigated fascia girder behavior during deck and overhang placement for both precast concrete and steel girders. The research team identified a number of construction details that were potentially problematic and focused the research program on the following areas:

• Potential loss of contact between beam and timber X-bracing during deck installation (Fig. 3)

A bridge identical to the bridge that put focus on this problem was constructed with the recommended horizontal bracing. This second bridge, with geometry and overhang loading essentially identical to the first bridge, performed as desired. TxDOT engineers measured girder rotation at each end of both fascia beams complete. During beam erection, the no. 5 tie is welded close to the top of the beam, providing a stiffer and more effective connection with the projecting shear reinforcement. After panel installation, the no. 5 reinforcing bars need to be bent down on both sides of the 4-in.-thick precast concrete deck panels to allow welding to the shear reinforcement (Fig. 2).

Figure 3. Timber X-bracing (left) was a required element in previous bracing standards for the Texas Department of Transportation (TxDOT). However, this bracing would sometimes lose contact with the girder due to fascia beam rotation during deck construction (right). The new TxDOT standard bracing detail uses horizontal timber bracing between the bottom flanges.

Figure 4. As an alternative to the no. 5 reinforcing bar welded to the girder shear reinforcement, a 12-gauge galvanized steel strap anchored to the top flanges allows installation of precast concrete partial-depth deck panels without removing and reinstalling the top tension tie.
before and after deck placement and found no measurable outward lean of the fascia girders.

The research led to a recommended design methodology for erection and slab placement bracing. TxDOT implemented this design methodology on its new series of prestressed concrete I-girders, called Tx girders, that were specifically designed to provide greater stability during handling, transportation, and erection. The Tx girders require significantly less bracing than TxDOT’s historical beam sections that predated and were similar to AASHTO shapes. A key element in the bracing design is the stiffness of the neoprene bearings. The bearings for the new Tx girders are much wider and are designed to have more compressive and lateral stiffness, which helps limit the amount of external bracing required. TxDOT replaced the timber X-bracing with more effective and more easily installed timber horizontal bracing. The top no. 5 tension tie weld detail was retained.

A TxDOT construction engineer has suggested an alternative to the top no. 5 tension tie, which is to use a galvanized steel strap anchored to the girders’ top flanges to provide the necessary restraint. The advantage of this option is that the straps can be installed during beam erection and the precast concrete deck panels can be installed without having to remove the tie (strap). In addition, the galvanizing provides long-term corrosion resistance to the exposed steel strap. Figure 4 shows bridge girders braced with these steel straps.

**Implementation**

TxDOT made the first significant revision to its prescriptive construction bracing requirements for precast concrete Tx girders as a result of the recommendations of Project 5706. Implementation of the galvanized steel strap option has been limited and lessons have been learned, specifically that the 12-gauge steel straps are thick enough that the straps must be prebent to lie flat on top of the girder at the lower side of the deck cross slope.

TxDOT’s standard drawing (Fig. 5) for minimum erection and bracing requirements for its I-girders provides the bracing details and the brace spacing limits, both of which depend on girder type and overhang width.

In addition to the bracing discussed in this article, the standard drawing also requires a temporary diagonal timber and cable brace at each end of the first girder erected. The size of the cable and the cross section of the timber brace have been increased for the longer Tx girders compared to previous bracing details.

**References**


Doug Beer is a branch manager for the bridge division construction and maintenance and Taya Retterer is a bridge standards engineer with the Texas Department of Transportation in Austin. John Holt is a senior project manager with Modjeski and Masters in Round Rock, Tex.
Concrete bridge design and construction have seen the effects of optimized girder shapes, accelerated bridge construction, the introduction of new strand and concrete materials, and an increase in the number and size of prefabricated elements. All of these topics either affect, or can be affected by, concrete form manufacturing companies. To gain insights on how formwork and its use can impact the efficiency and safety of a bridge project, and how new influences in the bridge market can have an effect on bridge forms, ASPIRE® submitted a list of questions to a panel of experts from Helser Industries, PERI USA, and Hamilton Form Company for their perspectives. The following is an edited version of their responses about recent and future changes and challenges related to formwork in bridge construction.

**Q** What have been the biggest changes in bridge formwork over the last 10 years and what do you anticipate in the next 10 years?

**HELSER**: The biggest changes have been larger span requirements and therefore taller girders with heavier strand loading. We expect more of the same in the years to come, especially with the onset of more advanced materials.

**PERI**: Looking back on cast-in-place (CIP) concrete bridges, there hasn’t been much change in formwork solutions. Unlike other areas of the concrete construction industry, the bridge segment has not been on the cutting edge of technological advancements. From our perspective, construction timelines have been reduced and environmental concerns have increased. As a result, we are seeing more concrete elements such as columns and caps convert to precast concrete. With accelerated schedules, that trend to prefabrication appears to be here to stay.

**HAMILTON**: Bridge beams are getting taller; as an example, we built a form for a 15-ft-deep haunched girder. This trend drives higher hydrostatic and uplift forces for the forms. Additionally, adding taller sections to existing product families has precasters looking to split forms and other adjustable features to create more shapes from the same base form. Precasters are also focusing more on production efficiency, asking for rolling forms and other solutions that help them to set up quickly or reduce the time to strip and get ready for the next concrete placement.

What challenges or opportunities do you see in the future? How will issues such as transportation funding, labor shortages, and regulations affect our industry?

**HELSER**: The nation has a huge opportunity to improve aging bridges while simultaneously boosting the economy for years to come. We just need to come together on a large-scale infrastructure bill and start getting ahead of things.

**PERI**: With labor shortages across the board, we manufacture solutions that are easy to use in the field to make concrete construction work simpler, faster, and safer. As the industry turns to less-skilled labor, it is critical to improve on those concepts.

The Veleno Bridge replacement in Zapata County, Tex., near the Mexican border, is a 16-span bridge made up of precast concrete girders, deck panels, and pier caps. A reverse radius soffit and curved blockouts were used for rounded pier cap features that create architectural interest while preventing birds from nesting under the bridge. Photo: Hamilton Form.
We also continue to expand our digital capabilities, investing in 3-D [three-dimensional] design technology. 3-D printed structures are a disruptive technology to our business. In Germany, PERI has used this technology in the residential market, and we plan to continue to develop it in the United States for multiple uses. We definitely see it being widely adopted in the future. Separately, 3-D printed formwork has potential for some architectural finishes, and we will continue to monitor how that technology evolves.

HAMILTON: As labor markets get tighter, we are looking for more ways to help our customers by treating forming as part of an integrated production system that combines the setup, heating, tarping, and related activities in a more efficient manner. We are experiencing our own challenges finding skilled workers. As we look to automate and mechanize our processes, we want to bring those ideas to our forming solutions. Another long-term challenge to our business is new materials and manufacturing processes. 3-D printing of forms is currently replacing complex wood forms for architectural specialty shapes. We recently built a hybrid 3-D printed shape combined with a steel form and will adapt as it becomes more commonplace. It will be a while, though, before a 3-D printed bridge girder mold competes with a long-line steel-form production system for standard DOT [department of transportation] shapes.

What would you like industry professionals to know about formwork? What should engineers and designers know about forms to improve their designs and foster innovation?

HELSER: The best designs are those that find the right marriage between financial reality and innovative design. Too often, the expert opinion of the precaster or their form builder is asked too late in the design phase. What might have been a good design idea structurally can become cost prohibitive from a production standpoint and either halted completely or get caught in cycles of project-delaying redesign.

PERI: Industry and DOT professionals should become more familiar with the capabilities and limitations of formwork options and how varying form geometries may lead to either increased or decreased inefficiencies in productivity on the jobsite.

HAMILTON: Whether bridge engineer or industry consultant, there is sometimes confusion regarding what is and what isn’t achievable with new forms. There is usually a way to achieve new and complex shapes. Some common mistakes in specs are the absence of adequate draft to lift pieces out of forms. We try to collaborate with customers on large projects and we want to get involved early to add value to the project. Formwork is often overlooked as to how critical it is to the success of bridge projects.

What common mistakes do you see in plans and specifications regarding formwork?

HELSER: The most common mistake made by designers is forgetting to think about forming requirements and the added production labor that certain design features cause for a precaster.

PERI: Keep an open mind to new technologies. It is easy to reuse drawings and specs from an old project, but that is one reason why there hasn’t been much forward momentum on the CIP side. For instance, we have seen specs that require a steel face finish, but there might be other, more modern options that can achieve similar results. Regional specifications are tied to means and methods. For example, California falsework requirements make it hard for contractors to use modular systems or more competitive solutions. We would suggest opening up the specs to allow for innovation.

HAMILTON: Engineers and designers need to know any limitations on shaping steel. We also like to be involved with how to optimize precast concrete production. We have worked with a wide variety of precasters and can bring that experience to bear on new challenges.

Can you comment on form modifications or ways that existing forms could be modified to change the section?

HELSER: While any form can be modified to adjust its profile, the quality of the final product may dictate a new form. For instance, narrowing a web on
Touted as a material that will revolutionize the concrete industry, ultra-high-performance concrete (UHPC) is a relatively unknown quantity when it comes to new shapes, forms, and sizes.

**Do you think that UHPC will be a game changer for the bridge construction industry? If so, will the higher fluid pressures add a premium cost to the forming systems?**

**HELSER:** While UHPC may very well be a game changer for the industry, the fluid pressures are in reality very similar to a highly vibrated traditional concrete mixture. All Helser forms are designed for high vibration or a fully liquefied mixture, and are therefore fully capable of supporting the onset of UHPC with no additional cost.

**PERI:** We haven’t run into UHPC in the bridge market yet. The biggest thing that UHPC will mean for us is potentially higher pressures. We would have to help the contractors really button up the formwork for a single vertical placement that can handle the hydrostatic pressure on the bottom of the form. PERI continues to explore technologies that can measure these pressures in real time, helping to support UHPC projects by eliminating the need to design to full liquid head pressure.

**HAMILTON:** We are participating in the PCI research and development project on UHPC. We helped with the formwork for a few of the volunteer precasters, but since we already design our forms to the full hydrostatic pressure, we don’t anticipate it to be any different from SCC [self-consolidating concrete] to UHPC mixtures.

**Do UHPC forms require special sealing systems because the fresh material “runs like water”?**

**HELSER:** History has already driven all the required forming technology to support UHPC girders. Any time you combine a high-architectural-quality requirement with a structural member, the form is required to prevent slurry leakage while being heavily vibrated to remove entrapped air, which causes a full liquid head similar to UHPC pressures. Some existing forms that weren’t originally designed to that level may require some additional gasketing at the soffit and headers, but structural integrity is not an issue. It may require more care by the end user of the formwork in the way of daily cleaning and maintenance of the forms to ensure proper closure.

**HAMILTON:** Penetrations will not be a problem from a stress standpoint. As far as keeping leakages under control, there will be some details to resolve, but nothing too difficult to accommodate.

To better utilize the advantages of UHPC, designers are looking at 0.70-in.-diameter prestressing strand and other girder shapes, like the Florida I-beam, which can have 17 strands in the bottom row. Can beam bed pallets be redesigned to be self-stressing for the bottom row of seventeen 0.70-in.-diameter strands? That would help producers avoid major in-ground bulkhead expenditures.

**HELSER:** The onset of higher compressive strengths and larger spans has already pushed the boundaries of many existing abutment structures. Helser has already helped customers combat this by adding a self-stressing soffit with a low-profile stress head to pick up the lower rows of strand while allowing the upper rows to pass over to the existing abutment. While there are certainly limitations to every structure, we should be able to do the same for much of what UHPC brings.

**HAMILTON:** Larger strand does put more stress on the bulkheads. However, it is just a question of designing to accommodate it. When we went up from 0.5-in.-diameter strand, it was a similar evolution. Self-stressing soffit is one strategy, and others will develop to fill the need.

**Conclusion**

Form lengths have grown, tensioning capacities have increased, and the use of automation and equipment to improve productivity continues to evolve. Form companies have grown and changed to help advance the concrete bridge construction industry.

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**ASPIRE** acknowledges the insights and expertise from Joel Whalen, engineering manager, Peter Ollmann, president, and Marianne Methven, director of sales, Hamilton Form Company; Dan Straub, sales manager, PERI Formwork Systems; and Rick Thomsen, chief engineer, Helser Industries.
The Federal Highway Administration (FHWA) Global Benchmarking Program (GBP) serves as a tool for accessing, evaluating, and implementing proven global innovations that have the potential to significantly improve highway transportation in the United States. Instead of recreating advances developed by other countries, the program focuses on acquiring and adopting technologies and best practices that are already available and being used. Using the GBP approach, topics that support departmental and agency priorities and strategic goals are selected for studies. To this end, the 2019 GBP study on electrically isolated tendons (EITs) examined how European countries have successfully used EITs in post-tensioned (PT) bridge structures. This article summarizes the GBP study report Electrically Isolated Tendons in European Transportation Structures. (See the Spring 2019 issue of ASPIRE® for an article on the first EIT demonstration project in the United States.)

The study team comprised FHWA, state departments of transportation, and industry representatives. The purpose of the study was to assess how well EIT systems are serving their international owners and to determine the processes for EIT qualification, installation, and monitoring for implementation in the United States. Interviews and site visits were conducted to collect information relevant to adopting EIT practices in the United States and to improving the construction quality, durability, and long-term performance of PT bridge structures, which represent a significant component of the domestic bridge inventory.

The study was organized around the following topics:

- Experience in EIT implementation and planned future improvements
- Criteria for use of EITs
- EIT system approval procedures
- Approved systems and certification requirements
- Workforce training
- EIT construction and installation methods
- Inspection procedures and reading frequency
- Tendon voids and damage detection
- Penalties for not meeting acceptance thresholds
- Long-term monitoring
- Costs and benefits of EITs from the owner’s perspective

The study itinerary (Table 1) consisted of one week of site visits and interviews with bridge owners, academics, PT system producers and installers, and technical consultants in Italy and Switzerland. These two countries have extensive experience in deploying EIT technology and were selected based on the results of a desk review that investigated worldwide examples of EIT design and construction.

### Table 1. Electrically isolated tendons Global Benchmarking Program study itinerary, May 2019.

<table>
<thead>
<tr>
<th>Day (Date)</th>
<th>Location</th>
<th>Activity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monday, May 20</td>
<td>Rome, IT</td>
<td>Meeting - Italian Railways and Italferr (Designer/Owner/Operator)</td>
</tr>
<tr>
<td>Tuesday, May 21</td>
<td>Milan, IT</td>
<td>Site Visit - Piacenza Viaduct (in-service)</td>
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<tr>
<td></td>
<td></td>
<td>Meeting - Tensa (Producer/Installer)</td>
</tr>
<tr>
<td>Wednesday, May 22</td>
<td>Zurich, CH</td>
<td>Workshop - ETH Zurich (Researchers/Owners/Producers)</td>
</tr>
<tr>
<td>Thursday, May 23</td>
<td>Hirschthal, CH</td>
<td>Site Visit - Mittelmuhlen Bridge (in-service)</td>
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<tr>
<td></td>
<td>Wolhusen, CH</td>
<td>Site Visit - Wolhusen Bridge (under construction)</td>
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<tr>
<td></td>
<td>Olten, CH</td>
<td>Site Visit - Ground Anchor Wall near Olten Rail Station (in-service)</td>
</tr>
<tr>
<td>Friday, May 24</td>
<td>Bern, CH</td>
<td>Meeting - VSL and Stahilton (Producers/Installers)</td>
</tr>
</tbody>
</table>

Source: Federal Highway Administration.
Summary of Key Findings

EIT Design

EIT systems are beneficial for ensuring PT tendons are protected against corrosion induced by aggressive environmental and chemical exposures or stray electrical current. Both the Italian and Swiss rail authorities recommend EIT systems for PT structures. In Switzerland, EITs are required where protection from stray current is a concern, such as for structures carrying electrified rail or soil nail wall structures.

EITs provide the most robust protection of PT structures through tendon designs that encapsulate strands in corrosion-proof and verifiably leak-tight ducts. EITs also allow the integrity of the encapsulation system to be monitored through electrical tests while in service. This involves electrical isolation of internal tendon components from all exterior metallic components of a structure followed by periodic monitoring of electrical isolation. The components of an EIT system that ensure the electrical isolation of the PT strand from metallic reinforcement outside the tendon are critical. The basic envelope is formed by non-metallic polymer ducts with robust seals at the connections and watertight vent tubes. Anchorages are electrically isolated by a nonmetallic isolation plate between the anchor head and the bearing plate. An insulated electrical reference wire is connected to the PT strand via the anchor head and extends through the grout cap to a junction box. Electrical impedance between the strand and a ground wire connected to the mild reinforcement can be measured with an LCR (inductance, capacitance, and resistance) meter to verify isolation, or lack thereof.

System Qualification and Approval

It is European practice that PT systems must be qualified through a series of tests specified in \( \text{fib} \) (International Federation for Structural Concrete) Bulletin 75 Polymer-Duct Systems for Internal Bonded Post-Tensioning\(^*\) that verify the air and watertightness of the anchorage and duct assemblies under both positive and negative pressure tests. Other duct component tests evaluate dimensional tolerances, stiffness, external point load resistance, wear and fracture resistance, and bond behavior.

Certification and Training

Producers of PT systems for European projects undergo periodic certification to be authorized to produce PT systems for construction. PT system installers receive extensive training, and some producers have developed rigorous training and personnel certification programs for all aspects of post-tensioning system design, construction, and inspection. Such programs include both classroom lectures and hands-on practical training to verify proficiency.

Construction and Quality Control

Difficulties in properly installing EITs were encountered during early implementations in both Italy and Switzerland. Coordination among all parties involved in design, fabrication, and inspection, as well as the owners, is important to manage expectations and achieve the best results. Experienced practitioners note that although precise installation techniques and proper care are needed to successfully install EITs, installation should not require extraordinary effort. Rather, it should become part of the routine practice for all PT structures. A well-encapsulated or isolated tendon is the basis for the EIT concept but does not diminish the need for proper grouting procedures and checks. Implementations of EIT systems tend to readily receive the appropriate level of care due to the ability of the technology to verify tendon encapsulation and, therefore, installation quality.

A supplemental half-shell added to the exterior of the polymer ducts where supported on external reinforcement is effective in preventing local buckling, excessive curvature, or abrasion of strand through the duct. Electrical tests can be conducted after tensioning of strands and after grouting to screen for breaches. Post-installation acceptance tests are based on LCR measurements, where acceptable alternating-current resistance criteria are established to guard against stray current, long-term environmental exposure as well as fatigue and fretting corrosion. Recommended resistivity thresholds based on Swiss guidelines\(^*\) are presented in the GBP study report. These thresholds are not
absolute; rather, they are based upon expert judgment and may require interpretation.

**Operation and Long-Term Monitoring**

EIT systems can be periodically tested in service to assess whether the tendon remains electrically isolated, as indicated by resistance and capacitance from LCR measurements. Resistivity of the system, due to maturing of the grout, increases in a log-linear fashion over time. A low value of electrical resistance does not indicate corrosion, but rather damage of the isolation barrier (ducts) of the PT system. Thus, a sharp decrease in resistivity in service may indicate the system has been breached. The inner corrosion protection (such as grout) may still be intact and the tendons can still be monitored over time except for cases where short circuits have occurred due to a breach. Methods have been developed that may aid in locating a breach in an in-service tendon.

**Conclusion**

The study team determined that EIT technology provides the following benefits:

- Reliable construction quality control of PT tendon installation through validation of isolation by means of electrical resistance measurements
- Early warning of PT tendon envelope breaches, before grouting of tendons, which provides the opportunity to repair the breach
- Reliable and easily interpretable PT tendon encapsulation data during structure service life
- Minimal change to the current state of the practice for PT details and installation processes

The team developed a series of recommendations to aid in the implementation of EIT technology in the United States:

- Develop design guidance tailored to U.S. practice that familiarizes engineers, fabricators, and contractors with the features and requirements of the technology. This can be accomplished through FHWA, the Post-Tensioning Institute (PTI), and the American Segmental Bridge Institute (ASBI).
- Incorporate EIT requirements into PT specifications, including methods of system qualification and approval. This would involve adapting concepts from fib Bulletin 75, *Polymer-Duct Systems for Internal Bonded Post-Tensioning*, into PTI/ASBI M50.3, *Specification for Multistrand and Grouted Post-Tensioning*.5
- Develop personnel training and certification requirements. This can be accomplished by augmenting PT Installer Training and PT Inspector Training programs offered by PTI.
- Promote EIT design, installation, and acceptance through a program of education and outreach. Professional venues such as FHWA, American Association of State Highway and Transportation Officials, PTI, ASBI, and Transportation Research Board meetings and conventions can be used as forums to give presentations, and trade journals can be used to publish case studies and articles.
- Develop guidance for EIT system operation and long-term monitoring that includes LCR instrument selection and use as well as guidance for interpreting readings for varying climates and conditions across the United States.
- Conduct additional focused research to address questions or challenges unique to U.S. practice, such as the use of epoxy-coated mild reinforcement, post-grouting inspection of anchorages, the influence of environment on acceptance criteria, and long-term creep performance of polymer-composite isolation rings.

The full GBP study report *Electrically Isolated Tendons in European Transportation Structures* can be downloaded from the following webpage:

https://international.fhwa.dot.gov/programs/mrp/electrically_isolated_tendons.cfm

**References**

The past few issues of *ASPIRE* contain articles addressing the collapse of the Florida International University pedestrian bridge. The articles call for engineers to exercise sound engineering judgment and to be willing to stand up to pressure so that these catastrophic incidents do not occur. Clearly, we have a responsibility to ensure the public is protected.

However, does our responsibility as engineers to “protect the public health and safety” end with protection from physical harm? Is our only duty to make sure no one gets physically hurt or dies? Actually, our duty to protect the public extends far beyond physical harm to how engineering designs and projects may affect humans in other ways.

Student learning outcome #4 of the Accreditation Board for Engineering and Technology is “an ability to recognize ethical and professional responsibilities in engineering situations and make informed judgments, which must consider the impact of engineering solutions in global, economic, environmental, and societal contexts.”

Thus, we are required to teach our students not just how to complete a safe engineering design, but also how to consider the overall impact of that design or project on society as a whole.

It sounds simple, but it is much more complex and difficult than it appears, especially the part about considering “global and societal contexts.” Many engineers and engineering students are uncomfortable with this. Economic or environmental impacts can often be quantified, but societal impacts do not fall into the kind of neat, numerical solutions engineers prefer. Beyond that, engineers often believe that evaluation of global or societal impact is best left to others, such as people engaged in politics, social work, or similar professions. But experts in these other professions might not fully appreciate how an engineering design will impact society, nor be able to suggest solutions that would require input from engineers.

One example I use in my class involves two neighborhoods in Cincinnati, Ohio. The two neighborhoods were one until the early 1970s, when construction of an expressway divided the neighborhood into a north section and a south section. Back then, no one worried about how expressways affected neighborhoods—an expressway represented progress, which was considered a good thing. I don’t think there was a lot of room for public comment, either. Since that time, the northern area has become an eclectic neighborhood favored by artists and others desiring a diverse neighborhood. It even changed its name. The southern neighborhood, which was much more industrial, has declined since the industry that supported it closed down. The two neighborhoods have been in opposite directions. It is impossible to say what the fate of these neighborhoods would have been had the expressway not been built. Maybe both would have thrived, or both would have declined, or maybe the expressway made no difference at all.

The problem, I point out to my students, is that no one back then even bothered to consider what the expressway would do to the people in the neighborhood. Perhaps if the engineers had had a bit of foresight, features could have been included to keep the neighborhood as one. Perhaps some large, wide pedestrian bridges could have connected the two neighborhoods. Maybe the expressway could have been buried, like the “Big Dig” in Boston, Mass. These are expensive solutions, but the cost of neighborhood deterioration, loss of residents, decreased tax base, blight, and crime are often much higher costs in terms of both dollars and the impact on people.

There are also questions of fairness. All of us in Cincinnati benefit from that expressway, but only a few neighborhoods had to bear the burden of its impacts. This is not to say the expressway was not needed or is not beneficial; the point is that the engineers had an obligation to consider the impact on the people living near the expressway and to do what they could, within reason, to mitigate the negative effects.

Many people call this “social justice,” which is defined as “the view that everyone deserves equal economic, political and social rights, and opportunities.” This is a tricky issue, as the term social justice is fraught with political nuances. However, as I point out to my students, this is not incompatible with engineering ethics. Section III.1.f of the National Society of Professional Engineers Code of Ethics for Engineers states, “Engineers shall treat all persons with dignity, respect, fairness, and without discrimination”—almost the same definition provided for social justice, but without the political connotations.

What we are asking engineers to consider is the impact an engineering design or project will have on people. I point out to my students a parallel ethical situation: human subject research. Human subject research requires respect for persons, which involves recognition of the personal dignity and autonomy of individuals and special protection of those persons with diminished autonomy; beneficence, which entails an obligation to protect persons from harm by maximizing anticipated benefits and minimizing possible risks of harm; and justice, which requires that the benefits and burdens of research be distributed fairly.

Engineering projects affect humans, so when assessing societal impact, similar ethical principles should apply. When we design a project, are we considering the

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*by Dr. Richard Miller, University of Cincinnati*
impact it will have on the communities around it? Are we trying to maximize the benefits for all, or just some? Are we treating the opinions and needs of all people fairly as the Code of Ethics requires?

Students often ask how this applies to them. They see themselves as designing a bridge or a building or a pavement, and they don’t see where they would be involved with these issues. I have three answers to this.

First, some of them eventually will have this power. Graduates of the University of Cincinnati have risen to be state, county, and city engineers who have the power to decide which projects go forward, how those projects are accomplished, and how public funds are allocated. They have the final approval on the plans for these projects. These engineers have the obligation to listen to the public and make decisions about these projects fairly and in accordance with the Code of Ethics.

Second, those who do not rise to a decision-making level still have a responsibility to consider the human impacts of designs and inform those with decision-making power what those impacts might be. If an engineer finds a design flaw that causes a safety issue, he or she is required to report it. If an engineer sees a potentially negative impact of a design on a segment of the population, he or she has the same obligation to at least bring it forward for consideration. Again, the point is not whether a project should or should not go forward; it is that we need to consider all of the impacts it may have and try to mitigate the negative ones if possible.

Finally, as engineers, we are also citizens. Section III.2.a of the Code of Ethics states, “Engineers are encouraged to participate in civic affairs; career guidance for youths; and work for the advancement of the safety, health, and well-being [emphasis added] of their community.” Thus, even if we are not directly involved in projects, we have an obligation to inform our civic leaders of social impacts and call for engineering projects that equally distribute benefit and burden.

For a lot of my students, this is heavy stuff. Ethics usually is. Answers are not clear or easy. However, the true ethical issue is not whether you decide one way or the other. The issue is whether you consider the impacts on humans or not.

References

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IN THIS ISSUE

https://transedlrt.ca/updates/valley-line-southeast-milestone-the-tawatinà-bridge-is-connected
This web page features a video of the three-span, extradosed Tawatinà Bridge. Arup, featured in the Focus article on page 6, is the lead designer for this Edmonton, Alberta, project.

The Focus article on page 6 mentions that the United Nations (UN) Sustainable Development Goals are key to Arup’s business philosophy and strategies. This is a link to the 2015 UN resolution adopting these goals.

https://newgdbridge.com
The designer for the new Gerald Desmond Bridge at the Port of Long Beach, Calif., was Arup, the subject of the Focus article on page 6. The innovative erection equipment used to construct concrete segmental box-girder approach spans is discussed in the Concrete Bridge Technology article on page 33. This website has videos, photos, and news about the project.

http://www.sfigdb.com
The mobile scaffold system used to construct the concrete box-girder approach spans leading to the cable-stayed main span of the Gerald Desmond Bridge is featured in the Concrete Bridge Technology article on page 33. This website has a video of bridge construction activities.

National Cooperative Highway Research Program (NCHRP) Report 950, Proposed AASHTO Guides for Bridge Preservation Actions, which is discussed in the Perspective article on page 14, can be downloaded from this link.

This link is to an archived presentation that includes construction photos of Section 28 of the Wekiva Parkway project near Orlando, Fla., including the U-beam bridges in the Wekiva Parkway #204 Systems Interchange featured in the Creative Concrete Construction article on page 30. The PCI Architectural Certification brochure is available at this link.

As mentioned in the LRFD article on page 53, Article 5.13 of the AASHTO LRFD Bridge Design Specifications has adopted the provisions for anchors in concrete from ACI 318-14. With the sponsorship of the National Cooperative Highway Research Program, PCI developed a webinar series for bridge engineers on the requirements for designing, detailing, and installing concrete anchors. This link provides access to a Dropbox folder with the recorded webinar series, course handouts, and resources.

https://www.concreteanchors.org/publications
In the LRFD article addressing anchors in concrete on page 53, the author recommends that Special Inspection Guidelines for Post-Installed Anchors by Silva and Mattis be read by those writing specifications or planning field-testing programs for anchors. The publication was developed by the Concrete and Masonry Anchor Manufacturers Association and can be accessed from this website.
This article is part 4 in a four-part series addressing concrete anchorage in the reorganized Section 5 of the American Association of State Highway and Transportation Officials’ AASHTO LRFD Bridge Design Specifications,1 published in 2017. At that time, AASHTO added Article 5.13, Anchorage to Concrete, to the design specifications. Part 1 of this series of articles (see the Summer 2020 issue of ASPIRE) outlined a PCI program sponsored by the Transportation Research Board to educate bridge engineers on the implementation of the new anchorage provisions adopted from the American Concrete Institute’s (ACI’s) Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14).2 Part 2 (see the Fall 2020 issue of ASPIRE) focused on the qualification procedures for post-installed concrete anchors. Any anchor that is designed with the provisions of Article 5.13 of the AASHTO LRFD specifications must be qualified, and its behavior must be shown to be consistent with the design provisions in the AASHTO LRFD specifications. Part 3 (see the Winter 2021 issue of ASPIRE) reviewed the mechanisms that are available to procure concrete anchors indicated in specification sections and on contract drawings once they have been designed. This current article concludes the series by reviewing adhesive-anchor installer certification, explaining an anchor inspector certification program, and discussing guidance on field testing anchors for compliance to design requirements in ACI standards ACI 318-193 and ACI 355.4-19 Qualification of Post-Installed Adhesive Anchors in Concrete (ACI 355.4-19) and Commentary.4

Installer Certification

The origin of many of the current requirements for installation and inspection of anchors can be traced to ACI 318-11,5 when the adhesive anchor design procedures were first introduced into the ACI building code. During the discussions in the ACI 318 committee before publication of ACI 318-11, the committee was aware of the 2006 ceiling collapse in the Interstate 90 tunnel in Boston, Mass., and that the National Transportation Safety Board report identified that the creep properties of the adhesive used in the project were deficient.6 However, others, including members of the ACI 355 committee, believed that although the adhesive did not have adequate creep resistance for the application, the overhead installation was also the most difficult orientation for adhesive installation and inexperienced installers should not be in control of many of the factors that affect the performance of overhead post-installed adhesive anchors under sustained load. A closer examination of the failed anchors revealed that the embedment hole was frequently not filled with adhesive or that the adhesive did not flow well into the threads of the anchor and thus compromised the bond strength.

My two copresenters of the PCI webinar series on concrete anchor design, Dr. Ronald A. Cook and Neal S. Anderson, were members of the ACI 318 committee at the time ACI 318-11 was being written. To get approval of the adhesive anchor provisions in the code, there were some heated committee discussions. Two critical issues were debated: sustained load and overhead installation. The ACI 355.4 standard was about to be published, and the testing protocols it specified to ensure that an adhesive has adequate performance when subjected to sustained load were available. The other critical issue was installation. Installation is not a task for a novice. Adhesive anchor installation is on par with structural welding—welders must be certified to do structural welding. The ACI 318 committee recognized that installation had a part in the failure in Boston and proceeded to incorporate provisions in ACI 318-11 that required any post-installed adhesive anchor carrying a sustained load and installed in an “upwardly inclined” orientation be installed by a certified adhesive anchor installer. ACI has developed an adhesive anchor installer certification program, which was available when ACI 318-11 was published.

Think of designing an adhesive anchor like a milking stool (Fig. 1): you need three legs for it to be stable.

Figure 1. There are three legs to the adhesive anchor quality stool: the design procedure in ACI 318-14 Chapter 17 2 (or Article 5.13 of the AASHTO LRFD specifications1), the qualification protocol in the ACI 355.4 material standard,4 and an operational installer certification program. Figure: Neal S. Anderson.
The first leg is the material standard, ACI 355.4, which qualifies the anchor as satisfying the requirements that are in the design code. The second leg is the design requirements found in Chapter 17 of ACI 318 and Article 5.13 of the AASHTO LRFD specifications, which define how the anchor carries load. The third leg for the successful use of adhesive anchors is an installer who can install them correctly in accordance with the manufacturer's printed installation instructions. That third leg is supplied by a certification program that ensures that the installer understands the installation procedures in the manufacturer's printed installation instructions and can perform the installation as intended. It is critical to install anchors according to the manufacturer's printed installation instructions because if the instructions are not followed, the anchor manufacturer is not obligated to ensure the published load values.

**Inspection Requirements and Inspector Certification**

The flowchart in Fig. 2 shows anchorage installation activities. Anchor installation requires a certified installer who knows how to install an anchor in any orientation, inspection of the anchor installation as required by ACI and AASHTO specifications, and construction documents that specify proof-load requirements specifically for adhesive anchors—also required by ACI and AASHTO specifications. Acceptance testing is discussed in the final section of this article.

Construction inspection is required by the *International Building Code* for all post-installed anchors used in building construction. The AASHTO LRFD specifications and ACI code also require inspection of adhesive anchor installation. This requirement has been in the ACI code since 2011. The new requirements in ACI 318-19, which are pending adoption by the AASHTO Committee on Bridges and Structures, are for all post-installed anchors (torque-controlled and displacement-controlled anchors), concrete screw anchors, undercut anchors, and adhesive anchors to be inspected by a certified inspector or a qualified inspector specifically approved for that purpose by the licensed design professional. ACI offers a program to certify such inspectors as ACI post-installed concrete anchor installation inspectors.

The certification program requires that the inspector has demonstrated the knowledge required to properly inspect the installation of post-installed adhesive and mechanical anchors, and understands the responsibilities of the qualification requirements for the installer. A post-installed concrete anchor installation inspector does not have to be a qualified installer but is obligated to understand the procedural steps for the anchor installation. A certified inspector must understand inspection requirements cited in the documents governing the construction specifications as well as the inspection requirements in the product evaluation service report (ESR).

A licensed design professional can specify periodic or continuous inspection of installed anchors in their specifications. The type of inspection required by code also depends on what the anchor manufacturer states in its ESR. All ESRs for the installation of adhesive anchors in overhead orientations carrying sustained load require continuous inspection.

ACI 318-19 requires certification of installers and inspectors. AASHTO is considering revising its requirements.
and adopting both Chapter 17 and the inspection and certification requirements in Chapter 26 of ACI 318-19. Certifications obtained through the ACI programs are valid for a period of five years.

**Construction Compliance**

**General**

Construction compliance includes both construction inspection and construction testing, as shown in Fig. 2. The question the designer or the inspection agency must ask is, what frequency is required for inspection and field testing? And for the inspector’s part, the inspector must know if periodic inspection or continuous inspection is required. Guidance for inspection is found in the project specification section on anchors or in the ESR for that anchor. Another excellent general reference on field inspection is in an article by Silva and Mattis. Reading this article before writing specifications or planning field-testing programs is highly recommended.

**Inspection**

Inspection can be required for either cast-in-place anchors or post-installed anchors. For a cast-in-place anchor, inspection is usually done before the placement of concrete. Inspection of cast-in-place anchors is similar to inspecting formwork and reinforcing bar placement before placing concrete.

For post-installed anchors, requirements for inspection are found in the appropriate ESR for the anchor. The AASHTO LRFD specifications list the resistance factor φ used for design. The φ-factors vary depending on the anchor reliability. Lower φ-factors are specified for anchors that receive periodic rather than continuous inspection. Periodic special inspection requires a certified inspector and is performed intermittently—the inspector is not present every time anchor installation steps are executed. Code for overhead installation requires continuous inspection, and this inspection is performed by a certified inspector who is continuously present. It is critical to have overhead installations continuously inspected to provide assurance that the design assumptions are justified.

**Field Testing**

Field testing can take place in one of two formats. In the first format, expendable anchors are installed and tested to failure to either develop site-load criteria or for direct comparison with the average capacity that is embedded in the design equations. In this second format, many tests must be conducted—30 or more for a given size (diameter) and embedment depth. The second format is a proof test on the anchor to some load such that the anchor can still be used in the structure if it passes the proof-load test.

The first format of field testing anchors as a means of establishing in-place strength is often confused with proof loading. Establishing in-place strength is not the intent of a proof-loading program. Proof loading is to make sure that there are no gross errors in the installation.

A fitting definition for proof loading is the application of tension load to fully cured adhesive anchors or, in the case of torque-controlled expansion anchors, application of a torque to an installed anchor to verify proper set of that anchor. Proof loading of screw anchors may be performed for anchors that have been prequalified for resetting. However, proof loading screw anchors should be approached with caution because tension testing may damage the screw threads cut into the concrete. It may be more appropriate to torque test an installed screw anchor.

The proof-load level is selected to be sufficiently high to provide assurance of correct installation, but the objective is that the proof-loaded anchor can still be used in the structure. Therefore, the load on the anchor should not be so high as to result in damage such as yielding or, for an adhesive anchor, any permanent slip. Currently there is no AASHTO or ACI standard for conducting proof-load tests.

- **Proof-load magnitude.** The magnitude of the proof load is typically set as a percentage of the anchor’s tested tension capacity. The proof load is not the load used on the load side of the design equation. Proof-load magnitude has historically been about twice the allowable tension for the anchor in concrete. Considering that anchors have a global factor of safety of about 4, resulting in the “allowable” load being about 25% of the average ultimate capacity, the proof load should be about 50% of the mean ultimate anchor tension strength using the traditional criteria. The proof load should not be influenced by edges and member thicknesses. So, a typical proof load can be set at about 50% of the mean ultimate tension strength of the anchor or 70% of the “characteristic” design value, but not more than 80% of the nominal yield stress of the steel components. The proof load only has to be maintained on the anchor for about 10 seconds.

- **Deformation acceptance criteria.** Acceptable displacements at the proof load depend on the anchor type, diameter, and embedment. For an adhesive anchor, the rule of thumb is that no displacement should be allowed at the proof load. For a post-installed mechanical anchor, the anchor should not slip (deform) unless the proof load exceeds the preload in the anchor at installation. For mechanical expansion anchors, it has been suggested that a slip of as much as 10% of the anchor diameter is acceptable. A remedial action plan should exist in the event of failure.

- **Frequency of proof-load testing.** The frequency of testing is up to the designer. The suggested minimum is 5% to 10% of the total number of anchors installed, and the anchors to be tested are to be randomly selected. If any anchors fail, there might be an issue with the installation and the number tested should be increased.

**Summary**

This is the last article of a four-part series discussing the content of a webinar series prepared by PCI as a comprehensive training program for highway bridge engineers to explain the implementation of the provisions in the AASHTO LRFD specifications for the design of post-installed anchors in concrete. This final article discusses certification for
installers and inspectors, inspection requirements depending on anchor type and installation orientation, and compliance testing of installed anchors.

Access to the five-session webinar training series on concrete anchor design is available at PCI.org/AnchoringToConcreteImp. For each of the five webinars, the following are available to download: the PowerPoint slides used in the presentation, a video of the presentation, the text of the presentation, course resource documents to amplify the written discussion, and a transcript of all the questions asked at the end of each webinar and the answers provided. These materials are available at no charge to anyone desiring more in-depth learning on the provisions for design of concrete anchors now in Article 5.13 of the AASHTO LRFD specifications.

References
2. American Concrete Institute (ACI) Committee 318. 2014. Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14). Farmington Hills, MI: ACI.
4. ACI Committee 355. 2020. Qualification of Post-Installed Adhesive Anchors in Concrete (ACI 355.4-19) and Commentary. Farmington Hills, MI: ACI.
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