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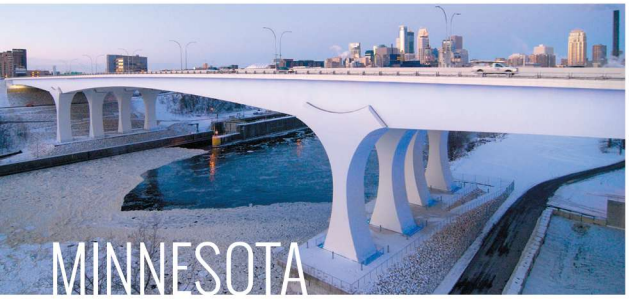

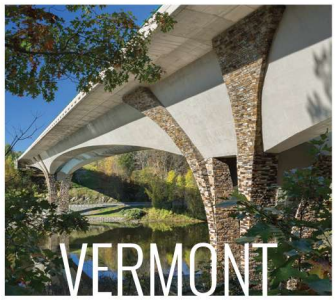

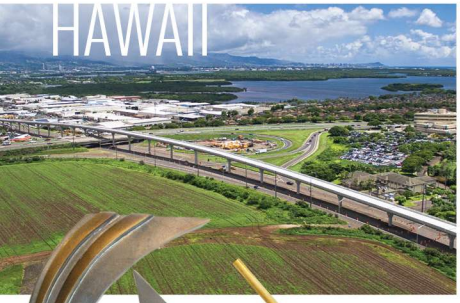

Photo: DSI.

Photo: Johnson, Mirmiran & Thompson.

Photo: Maine Department of Transportation.

Features

Departments

This issue's Concrete Connections will appear online at aspirebridge.org.

Photo: DSI.

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EDITORIAL

Photo: PCI.

Vehicular Trends and the Future of Bridges

Designing for versatility will be key as technology improves life

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

As members of the bridge engineering profession, we want our structures to have a service life of 75–100 years or even longer. None of us wants to see a 20-year-old bridge removed for structural or operational reasons. But, if we are to build long-lasting bridges, we need to anticipate the vehicular trends of the future. My curiosity about those trends was sparked earlier this year when I attended the Transportation Research Board's annual meeting, where participants could observe and ride the Federal Highway Administration's autonomous vehicle. I then attended a chapter meeting of the International Bridge, Tunnel and Turnpike Association, where I was eager to learn what financial professionals and transportation planners envision for the future of bridges and highways.

One crucial variable in bridge design is the fleet of vehicles that will be on the roads in future decades. Trend watchers predict that electric vehicles will be one-third of the inventory by 2040 and one-half of the inventory by 2050. They also anticipate that autonomous vehicles will be on the roads soon, although the widespread impact of these vehicles on the transportation network is harder to predict.[1]

Emerging vehicular technology promises to change human habits in ways that will directly affect transportation solutions, including fundamental aspects of bridge design such as lane geometry and access points. For example, commuters may not care so much about personal time spent in traffic if they can relax while their autonomous cars drive them home. Their incentive to pay tolls for express lanes or to use managed lanes restricted to passenger vehicles may therefore decline. But parcel delivery businesses might be willing to pay a premium to access faster lanes and bridges and shorten the time that their vehicles spend in traffic.

I also wonder whether we will soon be designing bridges for mass-transit, fixed-guide systems with ride-share and autonomous-vehicle technologies. If so, we may need to revisit older transportation solutions. For example, the slab tracks originally designed for buses might be adapted for elevated, personal autonomous vehicle pods. If that happens, we will need to formulate operational protocols, such as how to manage a disabled autonomous vehicle on a confined corridor. Will connected trucks operate in the corridors and on structures? That innovation could have implications for the strength requirements of the structures or the weight limit of the individual trucks.

An area of uncertainty is throughput in our future traffic systems. I have heard it might increase, but it could decrease if computer-controlled autonomous vehicles require increased headways to reduce the risk for collisions. When autonomous technologies control vehicles, exceeding the speed limit will not be an option.

Researchers are currently engaged in studies to model how emerging vehicular technologies will affect the functioning of households, and they have already noted differences between urban and rural areas of the United States. In some cases, autonomous vehicles and phone-based applications will likely select routes that are not on the current mainline arterials, perhaps to avoid tolls or heavy traffic. At the same time, our vehicles and phones will collect data on our driving routines and preferences and use that information to make subsequent decisions about what routes to take and what tolls will be incurred.

Pilot projects in Colorado and Ohio will answer some questions about vehicle-to-vehicle and vehicle-to-road communications. As we gather insights, we'll need to decide what capital programs will be needed to make the transportation system of the future work. Now is a great time for bridge engineers to get engaged so we can understand how new technologies will influence our roadways, bridges, and interchanges. This is also the time to start reviewing the design assumptions underlying our bridge codes. Will our investment in variable toll rate commuter lanes with bridge assets become a feature whose use will need to be redefined to carry heavier trucks or platoons of trucks? With connected vehicles, like a platoon of trucks, what bridge code design assumptions will need to be revisited and how will these new load effects be reconciled with the inherent capacity of our existing inventory? I certainly hope our bridge colleagues in leadership roles are helping to identify consistent and prudent strategies.

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DYWIDAG-Systems International provided the post-tensioning for the Gerald Desmond Bridge in Long Beach, Calif. Photo: DYWIDAG-Systems International.

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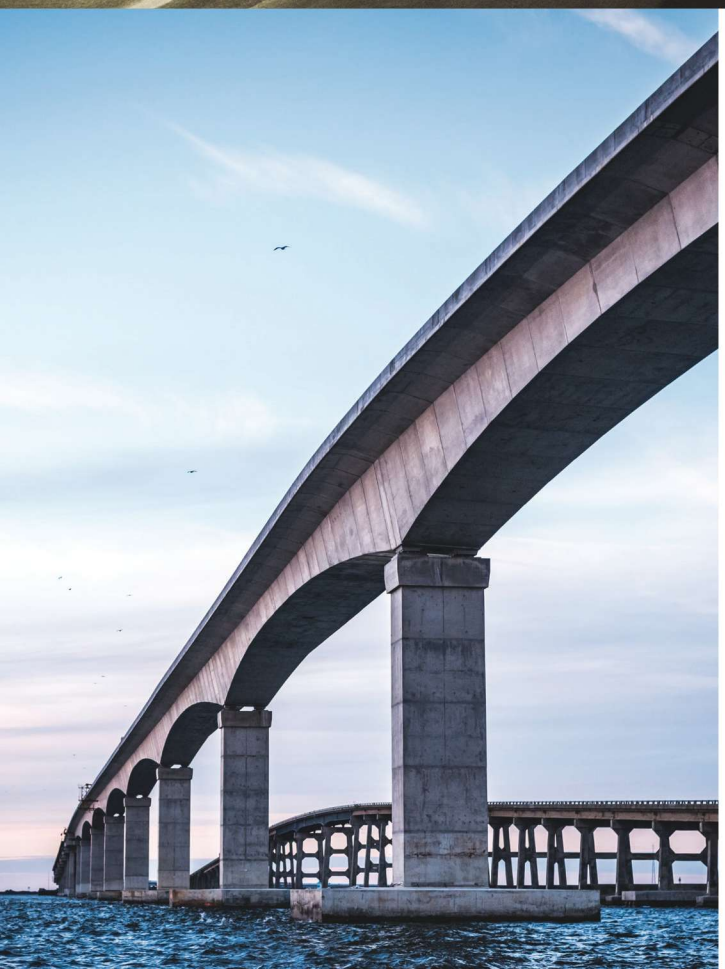
Reference

1. SP Global. 2018. *The Road Ahead for Autonomous Vehicles*. International Bridge, Tunnel and Turnpike Association. https://www.ibtta.org/studies-and-reports.

Photo Credit: New York State Thruway Authority



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Frederick Gottemoeller is an engineer and architect who specializes in the aesthetic aspects of bridges and highways. He is the author of *Bridgescape* and was deputy administrator of the Maryland State Highway Administration.

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Dr. Bruce W. Russell is director of the Bert Cooper Engineering Laboratory and an associate professor of civil and environmental engineering at Oklahoma State University. He has been a PCI member since 1990, and currently serves on the PCI Technical Activities Council and the PCI Bridges, Concrete Materials, and Fire Committees.

CONCRETE CALENDAR 2019–2020

For links to websites, email addresses, or telephone numbers for these events, go to www.aspirebridge.org and select the Events tab.

April 8, 2019
ASBI Grouting Certification Training Course
J.J Pickle Research Campus
Austin, Tex.

April 8–10, 2019
DBIA Design-Build for Transportation Conference
Duke Energy Convention Center
Cincinnati, Ohio

April 24–26, 2019
PTI Level 1 & 2 Multistrand and Grouted PT Specialist Workshop
Baltimore, Md.

April 27, 2019
PTI Level 1 & 2 Multistrand and Grouted PT Inspector Workshop
Baltimore, Md.

May 5–8, 2019
PTI 2019 Convention & Expo
Hyatt Regency Seattle
Seattle, Wash.

May 27–29, 2019
fib Symposium 2019
Krakow, Poland

June 2–5, 2019
Second International Interactive Symposium on Ultra-High-Performance Concrete
Hilton Albany
Albany, N.Y.

June 10–13, 2019
International Bridge Conference
Gaylord National Resort and Convention Center
National Harbor, Md.

June 24–27, 2019
AASHTO Committee on Bridges and Structures Annual Meeting
Renaissance Montgomery Hotel at the Convention Center
Montgomery, Ala.

June 28–29, 2019
PTI Level 3 Unbonded PT Repair, Rehabilitation, and Strengthening Workshop
Chicago, Ill.

July 22–25, 2019
BEI–2019 (Bridge Engineering Institute Conference)
Hyatt Regency Waikiki
Honolulu, Hawaii

August 4–7, 2019
AASHTO Committee on Materials and Pavements Annual Meeting
Four Seasons Hotel Baltimore
Baltimore, Md.

September 4–6, 2019
Western Bridge Engineers' Seminar
Boise Centre
Boise, Idaho

September 22–25, 2019
AREMA Annual Conference
Minneapolis Convention Center
Minneapolis, Minn.

September 25–28, 2019
PCI Committee Days and Technical Conference featuring the National Bridge Conference
Loews Chicago O'Hare Hotel
Rosemont, Ill.

October 1–4, 2019
PTI Committee Days
Hilton Santa Fe Historic Plaza
Santa Fe, N.Mex.

October 20–24, 2019
ACI Fall 2019 Conference
Duke Energy Convention Center & Hyatt Regency Cincinnati
Cincinnati, Ohio

November 4–6, 2019
ASBI 31st Annual Convention and Committee Meetings
Disney's Contemporary Resort and Grand Floridian Resort
Lake Buena Vista, Fla.

December 11–13, 2019
International Accelerated Bridge Construction Conference
Hyatt Regency Miami
Miami, Fla.

January 12–16, 2020
Transportation Research Board Annual Meeting
Walter E. Washington Convention Center
Washington, D.C.

Changing Perceptions

DYWIDAG-Systems International's new high-tech inspection and monitoring tools add to its selection of post-tensioning products and services, enhancing its role as a specialty contractor.

by Craig A. Shutt

While DYWIDAG-Systems International (DSI) is best known for its array of post-tensioning and geotechnical products, its role as a specialty contractor has been growing. Now, with the addition of several high-tech companies, DSI intends to expand its prominence and market penetration as a service provider.

New Scope of Services

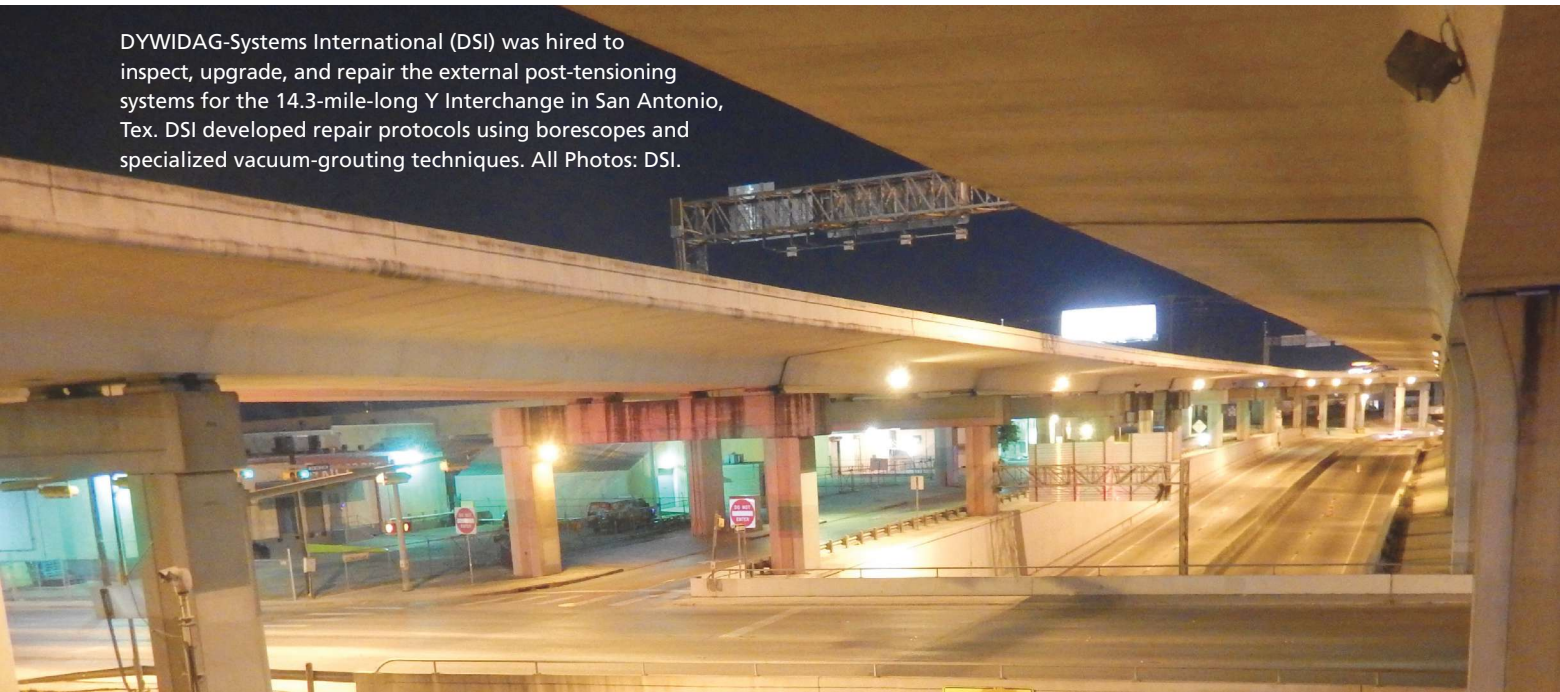
"We're no longer just the post-tensioning product supplier that people knew in the past," says Jason Caravello, director of business operations for DSI's Global Services business unit in North America. "For example, we are active in the inspection, restoration, and maintenance of parking structures, stadiums, high-rise buildings, tanks, and bridges. We are experts at design-build structural-strengthening solutions, often utilizing a variety of systems, including post-tensioning or CFRP [carbon-fiber-reinforced polymer], and even coatings. We obviously use our products where they apply, but we do much more than that."

'We're no longer just the post-tensioning product supplier that people knew in the past.'

Many recent changes at DSI began after Matti Kuivalainen was appointed chief executive officer of DSI Construction in the United States in September 2017. DSI Construction was split from DSI Underground the previous year by the Germany-based parent company DSI. In January 2018, the North American divisions were organized into the Global Services (contracting), Geotechnical, Post-Tensioning, and Concrete Accessories business units. Global Services focuses on three key service offerings: repair and strengthening for structures, robotic inspections, and infrastructure health monitoring, which includes cloud-based data acquisition for structural predictive analysis, among other services.

"Clients don't see a difference in the way they interact with us," Caravello notes of the reorganization. "But it gives us new efficiencies and initiatives for expansion." Those efforts are being aided by recent domestic and international acquisitions:

- In October 2017, DSI Construction announced a strategic partnership in the United States with Infrastructure Preservation Corp., a robotic engineering firm, to develop technology to aid in the nondestructive inspection of bridges.
- In March 2018, the parent company acquired Alpin Technik und Ingenieurservice, which provides robotic inspection and maintenance of infrastructure. Services include corrosion protection, special assemblies, and repairs.
- In October 2018, the parent company acquired Datum Group, a global monitoring company that performs structural health–monitoring services to enhance safety, sustainability, and cost savings.



DYWIDAG-Systems International (DSI) was hired to inspect, upgrade, and repair the external post-tensioning systems for the 14.3-mile-long Y Interchange in San Antonio, Tex. DSI developed repair protocols using borescopes and specialized vacuum-grouting techniques. All Photos: DSI.

"We hope to see significant growth in our capabilities for doing robotic inspections and repairs for tendons and cable stays," Caravello says. The robots can move along cables to make polyethylene repairs, perform water-jet cleaning, inspect via camera links, and document conditions in high resolution. "It's cutting-edge technology."

According to Caravello, the international companies will provide expertise to the U.S. division, and DSI plans to ramp up development throughout 2019. DSI has already used the robotic systems on the cable-stayed Fred Hartman Bridge in Houston, Tex., with excellent results, he notes.

Caravello acknowledges that robotic inspection technology is still developing. "But we're very excited about the potential," he says. "It offers ways to do inspections and repairs that are more efficient and safer." Drones can do some of that inspection work, he notes, but their use raises liability issues related to their potential to distract traffic and the challenges of controlling them in wind currents. "We think robotic options offer much more potential."

The Datum Group acquisition offers DSI the opportunity to expand health-monitoring services, especially for railroad structures, an area of specialization for the U.K.-based company. "Using inclinometers, accelerometers, and specialized sensors to gain real-time data on performance provides a clearer picture of a bridge's status," says Caravello. The firm monitors assets on highways and railroads internationally to assess performance of aging infrastructure, according to former Datum Group owner Rory O'Rourke. Its cloud-based database, Platform Interactive, harvests, validates, interprets, and presents data from any source. The firm's latest addition is an automated pile static-load testing system that can remotely apply a load up to 30 MN (6750 kip) to a pile. Used in Europe for some time, the system was recently shipped to its first U.S. customer.

These new technologies and expertise will be developed and marketed in the coming months, Caravello says. "There will be a period of development, but they add some high-tech tools to our toolbox and allow us to bring more services to the table."

Installation of a split-duct system for permanent repair of external tendons for the Y Interchange project in San Antonio, Tex.

Growth in the Repair Market

The repair market is growing substantially as states look to maximize the service life of bridges in an era of limited federal funding. "It's a critical market that's really growing substantially and yields good opportunities even in tough economic times."

An example of DSI's repair work can be seen in its services for the Y Interchange in San Antonio, Tex. The 14.3-mile-long elevated viaduct, built in the 1980s to merge Interstates 35 and 10, used 755 precast concrete segmental box-girder spans post-tensioned with longitudinal and transverse tendons. In 2014, DSI was hired to inspect, upgrade, and repair the external post-tensioning systems.

The project involved distinctive challenges. "When the bridge was built, project specifications did not reflect the importance of properly grouted tendons," says Caravello. Also, bridge construction was completed in three phases using three design/construction teams and post-tensioning suppliers. DSI inspected the anchors and tendons of each of the three phases and developed repair protocols using borescopes and specialized vacuum-grouting techniques that measured void volumes and then filled the voids with grout.

For external ducts that had incurred major damage over the years through splitting or cracking, DSI designed and installed a special split-duct system for permanent repair. Gaining access inside the precast concrete segments was a major challenge, Caravello notes. It was done from beneath the segments through integrated hatches, which were often located directly above heavy traffic areas, requiring traffic control and night work. DSI logged more than 75,000 work hours to complete the project.

Post-Tensioning Expertise

DSI's new offerings will enhance the company's existing expertise in post-tensioning services and products. "Post-tensioning is becoming more popular and mainstream across the country," says Joseph Salvadori, former Eastern U.S. regional manager for the Post-Tensioning & Reinforcing business unit. "We are seeing more segmental designs that need our products."

'Post-tensioning is becoming more popular and mainstream across the country.'

The trend offers great potential on the West Coast, where the company primarily works as a post-tensioning subcontractor. "California and the West are largely supply-and-install markets," says Bryan Lampe, U.S. Western regional manager for the Post-Tensioning & Reinforcing unit. "Many DOTs [departments of transportation] and the FHWA [Federal Highway Administration] are focusing attention on quality control of post-tensioning and the certification of field personnel, and most general contractors are not equipped to perform this work. As a result, we see an expansion of our installation work here and in states like Texas and Florida."

For example, DSI is providing longitudinal and transverse post-tensioning for the cast-in-place segmental box-girder approach spans for the 5134-ft-long Gerald Desmond Bridge in Long Beach, Calif. This bridge is currently under construction and will be, when completed, the second-highest cable-stayed bridge in the United States. The spans are being

DYWIDAG-Systems International is providing longitudinal and transverse post-tensioning for the cast-in-place segmental box girder approach spans for the 5134-ft-long Gerald Desmond Bridge in Long Beach, Calif. The spans are being constructed using a movable scaffolding system, its first use in the state.

constructed using a movable scaffolding system, its first use in the United States.

The typical span uses four or five tendons with twenty-seven 0.6-in.-diameter strands running full length in each girder web. Partial-length strand tendons, typically twenty-seven or thirty-seven 0.6-in.-diameter strands, pass longitudinally through the top decks, which are also transversely post-tensioned using tendons with four 0.6-in.-diameter strands in flat ducts with flat anchorages. Several units feature partial-length tendons that required tensioning from within the box girder's interior, which was achieved through close collaboration with the contractor.

On the Desmond project, DSI has already installed more than 3300 tons of bonded tendons ranging in size from four to thirty-seven 0.6-in.-diameter strands with anchorages. In all, nearly 10 million ft or about 7.3 million lb of tendons will be installed. DSI has also performed tensioning and grouting operations, including transverse deck post-tensioning. DSI's work on this project will be completed in mid-2019.

On the East Coast, where general contractors typically undertake post-tensioning duties, DSI's most frequent role remains that of product supplier. For example, DSI supplied its multi-strand post-tensioning products for the Interstate 91 Brattleboro Bridge in Brattleboro, Vt. This design-build project in an environmentally sensitive area featured cast-in-place segmental construction using the balanced-cantilever method with a form traveler (see the Project article in the Spring 2018 issue of *ASPIRE*®). The three-span, 1036-ft-long structure employed a straightforward post-tensioning design with only minor variations, Salvadori says. The 814 transverse tendons in the top slab used four 0.6-in.-diameter strand flat-anchor bodies with stainless steel local-zone anchor reinforcement.

The cantilever and continuity post-tensioning consisted of eight tendons with twelve 0.6-in.-diameter strands, 57 tendons with nineteen 0.6-in.-diameter strands, and 90 tendons with twenty-seven 0.6-in.-diameter strands, all of which used corrugated polypropylene ducts. The design also utilized permanent 36-mm-diameter (1.4-in.-diameter) Grade 150 threaded-bar tendons for longitudinal post-tensioning in the superstructure and in the tops of piers.

"There are always challenges with major projects, but Brattleboro was successful for both DSI and the contractor, with no major hiccups," Salvadori says. DSI is currently undertaking a growth initiative to expand the scope of its programs to furnish and install products in the Eastern region, he notes.

Upholding Grouting Standards

To ensure its expertise can extend a bridge's service life, DSI prefers to do its own grouting installations. Nationally, requirements have definitely become more stringent since bridges like the San Antonio Y Interchange were built. "Today, grouting specifications are a key focus for designers and contractors," says Caravello. "They understand the value it [grouting] provides."

Lampe agrees. "Owners and designers understand today that grouting is a critical component to the durability of a structure. It's become an area of focus in the past 15 years, with significant

Electrically isolated tendons (EITs) were included in the top tendon (of four) in each of the five girder lines making up the main channel unit of the Coplay-Northampton Bridge between the Pennsylvania boroughs of Coplay and Northampton. Their use in this bridge marks the first application of EITs in the United States.

upgrades to standards along with awareness and training on techniques and inspections. Grouting used to be an afterthought, and that's changed. Now it's recognized as arguably the most important element."

Execution is at least as important as specifications, he notes. "In the Western United States, post-tensioning is performed almost exclusively by specialty subcontractors, and their field personnel have extensive expertise. Workmanship is often the weak link, and it's critical. We're also seeing a major push to revise grout materials and methods and focus more attention on new technologies, which will help extend the structure's life."

'Owners and designers understand today that grouting is a critical component to the durability of a structure.'

Innovation

New technology was a key element of the design for the Coplay-Northampton Bridge in Coplay, Pa. The eight-span, 1118-ft-long precast concrete girder bridge features a three-span (181-181-186 ft), continuous post-tensioned, spliced prestressed concrete bulb-tee girder river crossing, the first of its type in the state. Five girder lines make up the main channel unit cross section, with girders in the pier segments varying in depth from 6 ft 7 in., the depth of the constant depth drop-in and end segments, to 9 ft 7 in. at the piers. All girders are made continuous by four longitudinal post-tensioning tendons with fifteen 0.6-in.-diameter strands.

DSI supplied engineering services, materials, and field-installation contracting for this project. It also installed electrically isolated tendons (EITs). This installation, which was requested by FHWA as a demonstration project, was the first use of EIT technology in the United States and its first use ever in a bridge with spliced precast, pretensioned concrete girders.

Because domestic electrically isolated anchor bodies are not available, DSI used its own ETAG (European Technical Approval Guideline[1])-approved anchors. An EIT was installed as the top tendon (of four) in each of the five girders. "The EITs provide the highest level of corrosion protection possible, as they isolate the strands completely so no electrical path exists from the strand to the structural concrete for corrosion to develop," Salvadori explains.

Post-tensioning strands have been installed and tensioning operations are underway for the main spliced-girder unit on the Coplay-Northampton Bridge between the Pennsylvania boroughs of Coplay and Northampton.

Engineers at Lehigh University deployed a monitoring system for impedance and capacitance, which is intended to provide an indication of performance of the corrosion protection system. Initial results at 28 days met expectations, and designers expect electrical resistance readings will rise as the grout ages. A significant drop in resistance would be indicative of leakage or ingress of moisture.

DSI participated in an FHWA-sponsored workshop on the EIT technology hosted by Lehigh University in October 2018, shortly after the last tendon was grouted. DSI is also looking to help update specifications related to EIT anchoring and duct ventilation. (For more information on EITs, see the Concrete Bridge Technology article in this issue of *ASPIRE* on page 36.)

DSI is also focusing on ways to strengthen members with carbon fibers and meet seismic requirements with AASHTO-type girders. The need to control long-term strand costs to reduce market volatility and allow for more precise cost estimates also remains a key need, Salvadori notes. "Challenges will always arise, and we intend to continue to work on improvements and advances."

Innovative approaches provided by its new partnerships will help DSI reach that goal. "We see these new technologies as a way to pull us into the market with expanded services," says Caravello. "We want the industry to see DSI as a bridge-repair contractor as well as a post-tensioning product supplier. It's hard to change perceptions, but we expect that to change as our offerings grow."

Reference

1. European Organisation for Technical Approvals (EOTA). 1997. *Metal Anchors for Use in Concrete.* European Technical Approval Guideline (ETAG) 001. Brussels, Belgium: EOTA.

DSI's 150-Plus Years of Service

Dyckerhoff & Widmann AG, later shortened to DYWIDAG International and then to DYWIDAG Systems-International (DSI), was founded in 1865 as Lang & Cie by German concrete manufacturer Wilhelm Gustav Dyckerhoff. The company's name and business strategy changed in 1866 when Dyckerhoff's son, Eugen, joined forces with his father-in-law, Gottlieb Widmann, to refocus on construction engineering.

The firm worked on some of Europe's earliest rammed-concrete bridges and developed an array of products and technical innovations for post-tensioning concrete. By the 1950s, it was offering licenses and consulting contracts for its products and construction methods around the world.

The firm employs about 2300 people worldwide. For more on its history, see the Partner Spotlight in the Summer 2016 issue of *ASPIRE*.

Quantitative Assessment of Resilience and Sustainability

by Dr. Jeremy Gregory, Massachusetts Institute of Technology

Sustainability is a word that is ubiquitous, yet often vague. At the Massachusetts Institute of Technology Concrete Sustainability Hub (CSHub), we have a concise mantra: "We want our world to last." This saying helps us clarify what we mean by sustainability with regard to the structures we build, including bridges. Such structures will remain sustainable only if civil engineers are farsighted. Many aspects of a structure, including its future economic impact and the environmental conequences of construction, repairs, or replacement, affect its sustainability. Our research finds that quantitative assessment of these factors can lead to alternatives that improve a structure's sustainability (see http://cshub.mit.edu/news/lca-research-brief-quantifying-hazard-life-cycle-cost for more information).

Sustainability analyses are usually conducted by considering a structure's typical loading conditions throughout its life. An additional important consideration is the structure's response to atypical loading conditions, such as those caused by natural or human-made disasters. A structure's response to such conditions is a measure of resilience that is vital to understanding sustainability, particularly in regions where climate-related events are becoming increasingly frequent and destructive.

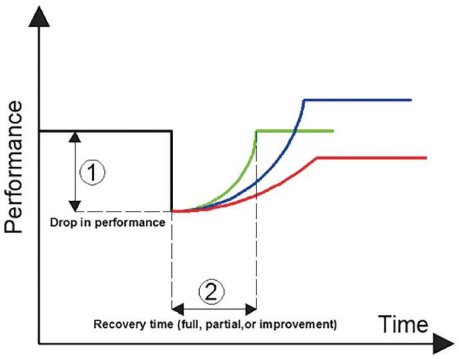


Figure 1. Resilience has two parts—initial response to a damaging event (the drop in performance) and recovery time. All Figures: Dr. Jeremy Gregory.

Resilience, an important element of sustainability, can be quantifiably measured in two stages, response and recovery (**Fig. 1**). First, we assess the system's immediate response to an event. For example, let's imagine that a city experiences a hurricane. Before the storm arrives, the city operates at peak performance, as indicated by parameters like economic productivity, transportation efficiency, and so on. As the storm ensues, the city's performance will drop because of economic inactivity, disabled infrastructure, and other damages. The extent of the drop in performance is the first measure of resilience—a smaller reduction in performance indicates greater resilience. The second measure of resilience is the time to recovery to a performance level that may be lower than, the same as, or higher than the performance level before the disaster. A rapid and strong recovery signifies effective resilience.

A recent example of resilience can be seen in the aftermath of Alaska's November 2018 7.0-magnitude earthquake. While the photos of crumbled highways were dramatic, the actual devastation was not extensive. Authorities reported no fatalities and no collapsed buildings, and seriously damaged roadways were repaired within days.[1,2] Pundits therefore declared the state's response to the disaster a success.

Alaska has not always enjoyed such resilience. In 1964, the state experienced the largest earthquake in U.S. history. This 9.2-magnitude quake shattered roads and bridges, destroyed neighborhoods, disrupted livelihoods, and killed an estimated 139 people.[3] In response to the earthquake, authorities established more rigorous building codes intended to protect buildings from future seismic activity. Alaska's successful response to the 2018 earthquake is due, in large part, to the changes implemented in building and bridge design codes.[1]

Alaska's rapid recovery demonstrates how efforts to improve resilience can deliver a return on investment (ROI). If stakeholders remain unaware of the ROI of resilience, success stories like that of Alaska in 2018 will occur less frequently. Calculating this ROI is therefore a key aspect of CSHub's work, and, to do it, a probabilistic hazard repair estimation model has been developed.

The crux of this model is the probability curve, which allows us to analyze the uncertainties of disaster and predict outcomes. Using government and other scientific data, we can generate a probability curve that estimates the likelihood of a hazard in a given location. We refer to this as a *hazard curve* (**Fig. 2**).

The hazard curve's counterpart in resilience analyses is the *fragility curve* (**Fig. 3**), which estimates the damage a hazard will inflict on a structure. To generate fragility curves, we employ a technique inspired by molecular dynamics. We model the integrity of a structure's components like we would the bonds of atoms. This methodology allows us to efficiently estimate the effects of hazard-induced loads on these components. What makes this technique unique is its versatility and precision. Whereas conventional models of building fragility approximate

damage for a broad class of buildings, we can efficiently generate fragility curves of the damage to different elements of specific building designs. Fragility curves for specific building designs give stakeholders a more detailed and quantitative understanding of building damage for proposed mitigation solutions.

By integrating hazard and fragility curves, we can create a comprehensive damage model that assesses potential damage from hazards of different intensities. When we assign costs to these damages and add them to the normal lifetime maintenance and construction costs of a structure as part of a life-cycle cost analysis, we can determine whether investments in resilience will prove cost effective over a structure's life when compared to conventional designs.

Our research has shown that the value of hazard resistance can be quantified and that investments in hazard mitigation can pay off, particularly in locations where hazard repair costs approach the value of initial construction costs. Within expectation of sustainable infrastructure, such resilience analyses for structures, including bridges, are critical. It is essential that we create a culture of resilience by educating stakeholders that quantitative resilience assessments are possible and necessary for infrastructure. Bridge engineers are well positioned to carry this mantle.

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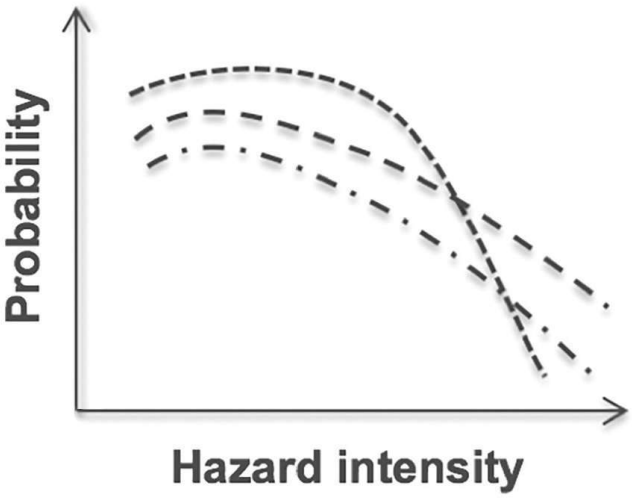


Figure 2. A generic hazard curve.

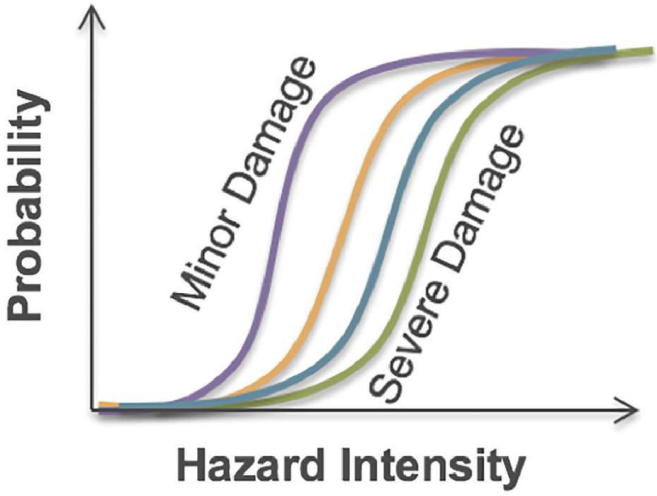


Figure 3. A generic fragility curve.

REHABILITATING MARYLAND ROUTE 195

(CARROLL AVENUE) BRIDGE OVER SLIGO CREEK

by Fred Braerman, Johnson, Mirmiran & Thompson, and Maurice Agostino, Maryland Department of Transportation, State Highway Administration

The Maryland Route 195 (Carroll Avenue) Bridge over Sligo Creek, located in the Washington, D.C., suburb of Takoma Park, Md., is a nonredundant, open-spandrel, reinforced concrete arch bridge that carries one lane of traffic in each direction, with sidewalks on both sides. Nearly 80 years after being constructed in 1932, the bridge became rated structurally deficient in 2011, when routine inspections found extensive and worsening deterioration of the bridge deck. While evaluating options for this aging structure, Maryland Department of Transportation (MDOT) officials recognized the functional and historic significance of the bridge and concluded that replacing it with a girder-type structure would not be the right solution. Instead, MDOT decided that rehabilitation was the appropriate strategy, a decision that was well received by the community. A construction contract was awarded to the prime contractor for $9.3 million in October 2015, and the project was completed less than one year after the bridge was closed to traffic. The bridge was reopened ahead of schedule in June 2017.

Planning and the Decision Process

Originally built with non-air-entrained concrete and black reinforcing steel, the 220-ft-long Carroll Avenue Bridge has span lengths of 65, 90, and 65 ft and traverses Sligo Creek Parkway, Sligo Creek, and a county-owned trail for hiking and biking. In 2011, the MDOT State Highway Administration (SHA) Office of Structures contracted the bridge design engineer to perform an in-depth field inspection, structural analysis, and assessment of the bridge. Because the bridge was eligible for inclusion on the Historic Register and on MDOT SHA's Priority 1 list for preservation, the bridge design engineer focused on rehabilitation strategies.

The bridge design engineer's findings indicated that replacement of all portions of the bridge above the arch ribs would be required to address the extensive deterioration of the deck, floor beams, columns, and spandrel arches. The plan was to repair and preserve the existing substructure units and arches while reconstructing the remainder of the bridge from the arch ribs up. The complete reconstruction of the bridge above the arch ribs would allow for the structure to be widened curb-to-curb by 2 ft, creating 11-ft-wide travel lanes. The community wanted wide shoulders to make biking safer in this residential area.

Additionally, the rehabilitation would include new sidewalks on both sides of the bridge, with new sections of sidewalk extending past the ends of the bridge to provide improved pedestrian access to all four approaches to the bridge.

During the planning stage, several project-specific challenges were noted. The bridge's location, 50 ft over a ravine within the floodplain of Sligo Creek, constrained the project site. Existing utilities on the bridge and adjacent overhead utilities on the west side of bridge would have to be temporarily relocated to allow for construction. Sligo Creek Parkway, located beneath the southernmost span, was a major commuter route within this congested suburb, and a heavily used hiker-biker trail located beneath the northernmost span would need to remain open during rehabilitation.

Project leaders conducted extensive public outreach to gain input and support from the community. For example, they held multiple public meetings and coordinated plans with Washington Adventist Hospital on the north side of the bridge, the Takoma Park City Council, the Montgomery County Transit bus system, Montgomery County Parks, and local emergency responders.

Design Challenges

When a bridge is rehabilitated, its geometry must be maintained. The existing Carroll Avenue Bridge was

profile

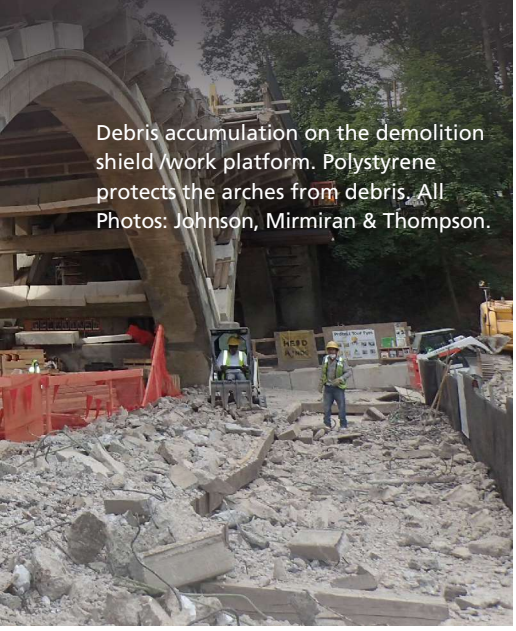
MARYLAND ROUTE 195 (CARROLL AVENUE) BRIDGE OVER SLIGO CREEK / TAKOMA PARK, MARYLAND

BRIDGE DESIGN ENGINEER FOR REHABILITATION: Johnson, Mirmiran & Thompson, Hunt Valley, Md.

PRIME CONTRACTOR: Kiewit Infrastructure South, Omaha, Neb.

REINFORCING STEEL SUPPLIER: Gerdau, York, Pa.

OTHER CONSULTANTS (ROADWAY AND TRAFFIC DESIGN): Mercado Consultants Inc., Ashton, Md.; Sabra and Associates Inc., Columbia, Md.



Debris accumulation on the demolition shield /work platform. Polystyrene protects the arches from debris. All Photos: Johnson, Mirmiran & Thompson.

on the crest of a vertical curve, which resulted in a sump at each abutment just past the end of the bridge. The City of Takoma Park requested that drainage improvements be made to alleviate flooding from these sumps. During design, the bridge design engineer found that the existing sump inlets were severely undersized as the result of increased development over the years. The inlets were therefore replaced with larger, more efficient curb-opening-at-grade inlets to minimize the potential for flooding in the future.

Maintenance of traffic during reconstruction of the cast-in-place reinforced concrete bridge was a significant challenge. Unlike a typical redundant, multi-girder bridge arrangement, removing one of the two arches during a staged construction scheme would render the bridge unstable. After this inherent problem with the structure configuration was explained to the stakeholders, all parties agreed that closing the bridge during construction and detouring traffic was the only feasible construction solution. However, this decision presented a problem for pedestrians because there was no alternative way to safely cross the ravine on foot. Therefore, MDOT SHA required the contractor to provide a 295-ft-long temporary pedestrian bridge along the east side of Carroll Avenue.

A major structural concern involved maintaining a balanced-loading condition during the sequence of demolition and build back to avoid overstressing the existing arches that were to remain. The project leaders were especially mindful of this issue because three workers had died on this site in 1932, when a previous reinforced concrete bridge built in 1904 collapsed during demolition prior to construction of the arch bridge.[1,2] In the rehabilitation project, every precaution was taken to ensure safe conditions. The contract plans included requirements for a 22-stage systematic construction sequence working from the center of each arch outward in a concentric fashion about the crown of the arch. A structural analysis was performed using software under each condition of demolition and build back to verify that the arches were not overstressed during any of the stages.

The reconstruction plans were painstaking in recreating and replicating the details of the existing bridge, which included extensive architectural features for the spandrel arches, floor beam overhangs, and columns. These architectural details were matched identically. However, details could not be exactly matched for the existing open-balustrade railings, which were significant to the Maryland Historical Trust. Fortunately, the Texas Department of Transportation Traffic Railing Type C411, which was crash tested and met the test requirements for the design speed and classification of Carroll Avenue, closely resembled the existing railings. This barrier received approval without comment.

Construction

The primary construction challenge for the project was access. Steep side slopes at each end of the bridge precluded the provision of construction entrances from Carroll Avenue. The solution was to provide an entrance off Sligo Creek Parkway; however, this location complicated matters because the parkway also served as the temporary detour route. Flaggers were employed daily to facilitate traffic flow while construction vehicles entered and exited the site.

One of the construction innovations that made the project such a success was the prime contractor's use of a work platform at the elevation of the arch supports. This work/demolition platform consisted of steel beams and timber flooring supported by shoring towers 15 to 20 ft above the floodplain of Sligo Creek. This platform solved

Concrete being placed for new floor beams at the top of one of the arches.

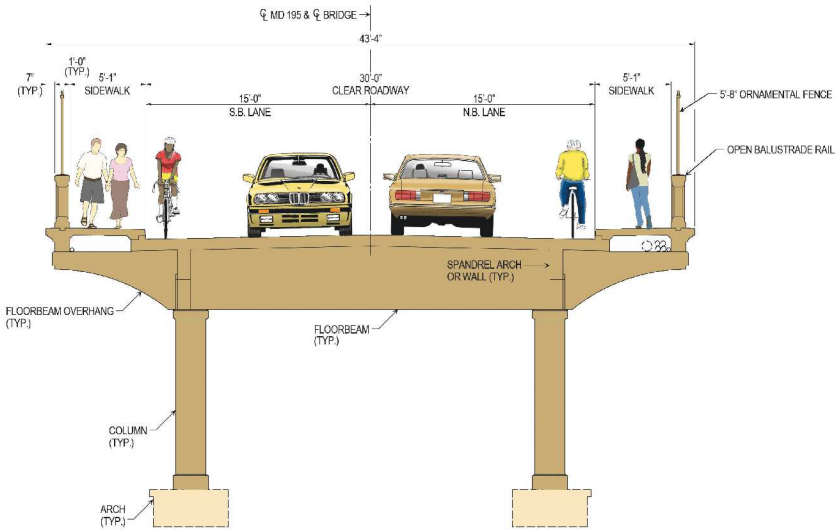
MARYLAND DEPARTMENT OF TRANSPORTATION STATE HIGHWAY ADMINISTRATION, OWNER

BRIDGE DESCRIPTION: Three-span, 220-ft-long, open-spandrel concrete arch

STRUCTURAL COMPONENTS: Repair of substructure units and arch ribs and replacement of columns, floor beams, and deck using 712 yd^3 of concrete MDOT SHA Mix No. 6 (28-day compressive strength of 4500 psi); 64 yd^3 of concrete MDOT SHA Mix No. 3 (28-day compressive strength of 3500 psi); 172,000 lb of epoxy-coated reinforcing steel; and 15,000 lb of black reinforcing steel

BRIDGE CONSTRUCTION COST: $9.3 million

AWARDS: American Council of Engineering Companies Maryland 2018 Outstanding Project Award; Portland Cement Association 16th Concrete Bridge Awards (2018) Award of Excellence



Rendering of proposed bridge section shows roadway with space for bicycles and sidewalks for pedestrians. Figure: Maryland Department of Transportation, State Highway Administration.

the problem of trying to work on the uneven terrain beneath the bridge and greatly facilitated construction demolition and build back. It also minimized the project's environmental impact. As concrete was demolished and dropped to the platform, small excavators transferred the material to the staging area, where the reinforcing steel was separated and loaded onto trucks for hauling.

The contractor made extensive use of polystyrene during demolition of the existing superstructure to protect the existing arches during construction and provide fillers in the formwork for creating the architectural details on the floor-beam overhangs, spandrel arches, and openings in the bridge railing. Most of the new concrete for the floor beams and deck slab was pumped from behind the abutments of the bridge. However, bringing in a concrete pump truck was not economical for placing the small volume of concrete needed for the short columns. Instead, the concrete placements were made in a fashion similar to the original construction in 1932, with workers lifting buckets and placing concrete by hand.

Design Criteria and Materials

All new elements of the bridge were designed in accordance with American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications,* sixth edition. The existing arches and substructure units were analyzed and load-rated in accordance with allowable stress design methods, which more accurately correlated to the original design.

All superstructure concrete consisted of MDOT SHA Mix No. 6 (which has a 28-day concrete compressive strength of 4500 psi). All reinforcing steel in the deck slab, floor beams, bridge barrier, and sidewalks on the bridge was epoxy coated. To extend the life of the new bridge deck slab, an epoxy-polymer concrete overlay was applied after the bridge deck slab was fully cured.

Pervious concrete was used on the sidewalk approaches to the bridge. This porous concrete feature allowed for stormwater infiltration and minimized runoff, providing additional stormwater management, which was considered a value-added improvement for the project because the project was exempt from stormwater management improvements per state requirements.

References

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2. Hendrix, Steve. 2016. "Three Workers Once Died Rebuilding This Historic Md. Bridge. Now a New Makeover Is Under Way." *Washington Post,* May 13, 2016.

Fred Braerman is vice president, structures transportation, for Johnson, Mirmiran & Thompson in Hunt Valley, Md., and Maurice Agostino is deputy director, Maryland Department of Transportation, State Highway Administration, in Baltimore, Md.

View of completed Carroll Avenue Bridge from Sligo Creek trail.

PCI Offers FREE *Bridge Geometry Manual* and Training

A *Bridge Geometry Manual* has been developed as a FREE resource that presents the basics of roadway geometry and many of the calculations required to define the geometry and associated dimensions of bridges. Four FREE instructor-led training (ILT) courses have been developed to facilitate the use of the Bridge Geometry Manual. The target audience for the *Bridge Geometry Manual* and instructor-led training courses is bridge engineers with two or less years of experience and CAD technicians who have not had previous training in roadway and bridge geometry. Again, there is no cost to enroll in and complete any of these courses being taught by the contracted primary author Corven Engineering Inc. or to download the new manual.

Register for these ILT Courses

Go to pci.org/PCI/Education/Webinars

Fundamentals of Roadway Geometry

(T505) on April 16, 2019 at 2:00 PM Eastern time

Fundamental to establishing bridge geometry and the resulting geometry and dimensions of bridges is the ability to work with roadway geometry provided by highway designers. Terminology used in roadway geometry is discussed. The three primary aspects of roadway geometry (horizontal alignment, vertical profile, and cross-section super elevations) are presented in this course. Stationing along horizontal alignments is also defined. Mathematical formulae and graphical representations are presented to define these three primary aspects, along with worked example problems.

Working with Horizontal Alignments

(T510) on April 30, 2019 at 2:00 PM Eastern time

Locating points along or offset from a horizontal alignment is necessary to defining bridge geometry. This course presents a vector-based approach to locating the North and East coordinates of point defined by a horizontal alignment. The vector-based approach was taken as it closely follows the way CAD software is used. Vector solutions are presented for 1) Locating points on tangent runs and circular curves, 2) Locating points offset from tangent runs and circular curves, and 3) Project points onto a tangent run or circular curve.

Geometry of Straight Bridges

(T515) on May 14, 2019 at 2:00 PM Eastern time

The third course begins by introducing terminology common to bridges elements. The material presented in the first two courses is then used to define the geometry of straight bridges. Concepts of skew crossings and the impact on bridge geometry are presented. Haunches that are cast over girders along with the bridge deck are defined. Factors effecting haunch thickness are defined using mathematical equations, which are demonstrated through examples. Substructure geometry in the form of top of bearing seat elevations is presented. Example problems are used to reinforce the information presented.

Curved Bridge Geometry

(T520) on May 28, 2019 at 2:00 PM Eastern time

This course advances the approach to bridge geometry developed for straight bridges to curved bridges. The geometry of curved bridges using both straight, chorded girders and curved girders is presented. The geometry of two curved bridge types (precast segmental box girders and curved u-girders) is introduced and an approach to computing their geometry is outlined.

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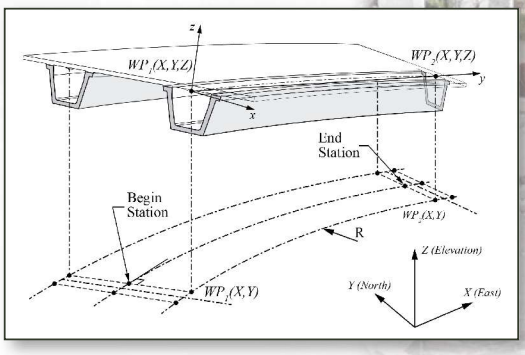


AASHTO THE VOICE OF TRANSPORTATION

PCI® Precast/Prestressed Concrete Institute

All Photos: Corven Engineering Inc.

PROJECT

Northwest Corridor Express Lanes

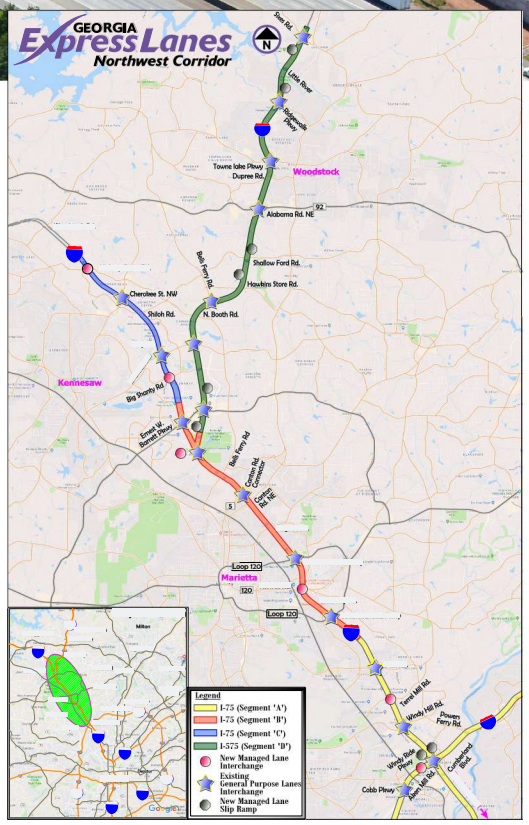
Aerial view of a mainline bridge. Bridge 25 parallels existing Interstate 75 southbound lanes and is over 1 mile long. Photo: Aerial Innovations of Georgia Inc.

by Alan Kite, Parsons Corporation

The Interstate 75/Interstate 575 (I-75/I-575) corridor northwest of the Atlanta Beltway, or Interstate 285 (I-285), runs through a densely populated and rapidly growing section in the Atlanta, Ga., metropolitan area. This corridor currently accommodates 297,000 vehicles per day, which almost meets the traffic projection of 310,000 vehicles per day by 2035 per the project's environmental assessment. Several planning studies over the past 15 years have looked for economical alternatives to alleviate daily traffic congestion, increase traffic capacity, and provide a construction scheme with minimal impact on existing traffic. In the fall of 2013, a public-private partnership, design-build-finance contract was executed to construct reversible managed toll lanes that will allow drivers to choose to pay a dynamically priced toll to bypass congestion.

The project extended from the I-285 (Atlanta Beltway)/I-75 interchange parallel to I-75 to Hickory Grove Road (approximately 17 miles) and from the I-75/I-575 interchange parallel to I-575 to Sixes Road (approximately 12 miles). The project scope included 39 bridges, two major interchanges, roadway, signing, tolling facilities, intelligent transportation systems, and all ancillary structures for the project. Flyover ramps within the I-75/I-285 interchange are up to 2827 ft long with span lengths up to 175 ft and pier heights up to 90 ft to cross numerous roadways, existing flyover ramps, and irregular terrain.

Prestressed concrete girders were used on several of the interchange bridges to construct cost-effective and low-maintenance structures. North of the I-75/I-575 interchange, mainline viaduct superstructures are 39.25 ft wide with four prestressed concrete bulb-tee beams, 72 to 74 in. deep, supported on single-column hammerhead piers. Bridge lengths range from 1298 to 5981 ft to span over existing local roads, railroads, streams, wetlands, and



Project location and vicinity map.
All Figures: Parsons.

profile

NORTHWEST CORRIDOR EXPRESS LANES / MARIETTA, GEORGIA

BRIDGE DESIGN ENGINEER: Parsons, Peachtree Corners, Ga.

OTHER BRIDGE CONSULTANT: Heath & Lineback Engineers, Marietta, Ga.

PRIME CONTRACTOR: Northwest Express Roadbuilders, a joint venture of Archer Western and Hubbard Construction Company, Marietta, Ga.

PRECASTER: Standard Concrete Products, Atlanta, Ga.—a PCI-certified producer

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

OTHER MATERIAL SUPPLIERS: Mechanically stabilized earth wall panel fabricator: Reinforced Earth Company, Atlanta, Ga.; noise barrier panel fabricator: Oldcastle Precast Inc., Newnan, Ga.—a PCI-certified producer

on/off ramps to I-75. Throughout the project corridor, smaller bridges were used to span over local roads and streams.

The $654 million design-build-finance project required a fast-paced design schedule, with all bridge and structure design to be completed in 15 months. To meet this schedule, design teams were established in six Parsons offices (Atlanta, Ga.; Baltimore, Md.; Boston, Mass.; Chicago, Ill.; Orlando, Fla.; and Washington, D.C.) and the local office of Heath & Lineback Engineers to create the design and generate contract documents. Each office was assigned bridges with similar superstructure and substructure types and similar span arrangements to provide design consistency and an efficient design that met the design schedule. Another advantage of using multiple offices was that it allowed independent design checks on major bridge elements to be performed by engineers not directly involved in the initial design.

Prior to the start of design, a project-specific design criterion was written that included contract-specific requirements, Georgia Department of Transportation (GDOT) bridge standards, and contractor-preferred materials (for example, concrete strengths), where applicable.

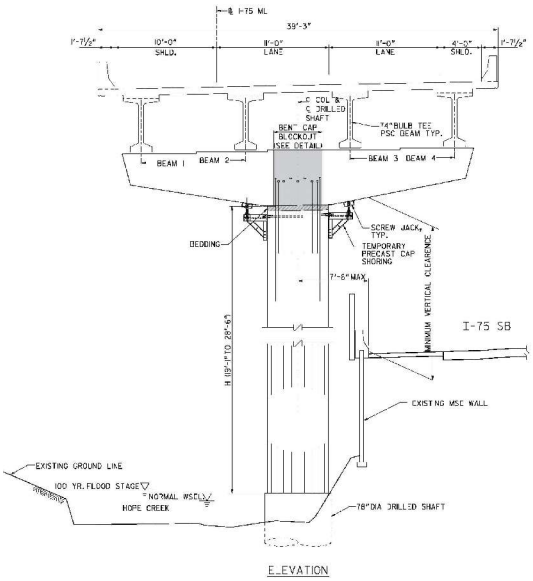
Precast concrete girders were selected because rapid construction was critical to meeting the schedule, and because of their additional benefits of high durability, low maintenance, excellent quality, and low cost relative to other construction materials. In addition, standard details were established and provided to each bridge design team to ensure project-wide design and detailing consistencies. Weekly conference calls with all offices were held to discuss design issues, the use of similar details, and adherence to the design schedules.

Precast Concrete Pier Caps

Numerous project challenges related to site conditions and economical construction methods had to be resolved at various locations along the alignment. At one project location, a proposed 3500-ft-long bridge would be adjacent to the existing I-75 roadway. A mechanically stabilized earth (MSE) wall supported the existing fill, with a stream and environmentally sensitive wetlands located at the bottom of the wall and running parallel to the interstate. Eighteen single-column piers along the proposed bridge were to be constructed in the narrow wetlands area between the stream and the wall.

During the early stages of the project, it was evident that access to the site, traffic maintenance, and minimizing impact on the wetlands would be a major challenge. Two alternatives were evaluated: building a conventional temporary work bridge, and using accelerated bridge construction methods with precast concrete pier elements. Ultimately, it was decided to perform all major construction activities from the existing southbound interstate shoulder with limited nighttime lane closures.

The piers are founded on 78-in.-diameter drilled monoshafts. Excavated spoil was contained with permanent casings that were installed with drilling equipment staged on the southbound I-75 shoulder. The maximum offset from the existing I-75 southbound curb to the centerline of the pier shaft was limited to 7.5 ft. This distance was dictated by the practical reach of the drilling rig as well as the maximum allowable surcharge



Cross section of a 3300-ft-long bridge adjacent and parallel to southbound Interstate 75, the existing retaining wall, and wetlands. Figure: Parsons.

pressure behind the existing MSE wall. This system minimized the impact on the environmentally sensitive wetlands area, remained within the required vertical and lateral structural loading capacities, and limited traffic interruptions.

Precast concrete caps were cast by the contractor at a yard located approximately 5 miles from the pier construction site. The precast concrete units were transported individually using a single specialized-permit vehicle. A rapid setup/takedown type of crane was used to erect the precast concrete caps onto a temporary support bracket attached to the top of the column. The temporary shoring bracket included screw jacks mounted on top of the pier column to support and provide geometry control to the precast concrete cap before the connection was cast.

A cast-in-place concrete bedding layer between the top of column and bottom of cap was used to provide

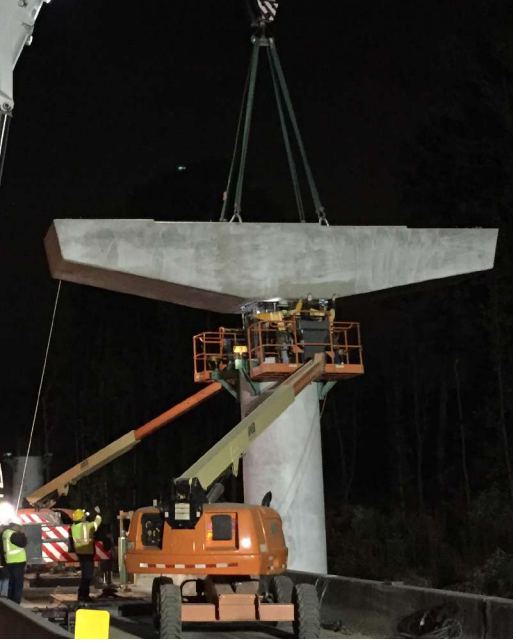
GEORGIA DEPARTMENT OF TRANSPORTATION, OWNER

BRIDGE DESCRIPTION: The project included 39 bridges; mainline bridges were between 807 and 5981 ft long and 29 ft 3 in. to 39 ft 3 in. wide. Mainline bridge superstructures have a 7½ in. thick cast-in-place concrete deck and prestressed concrete girders.

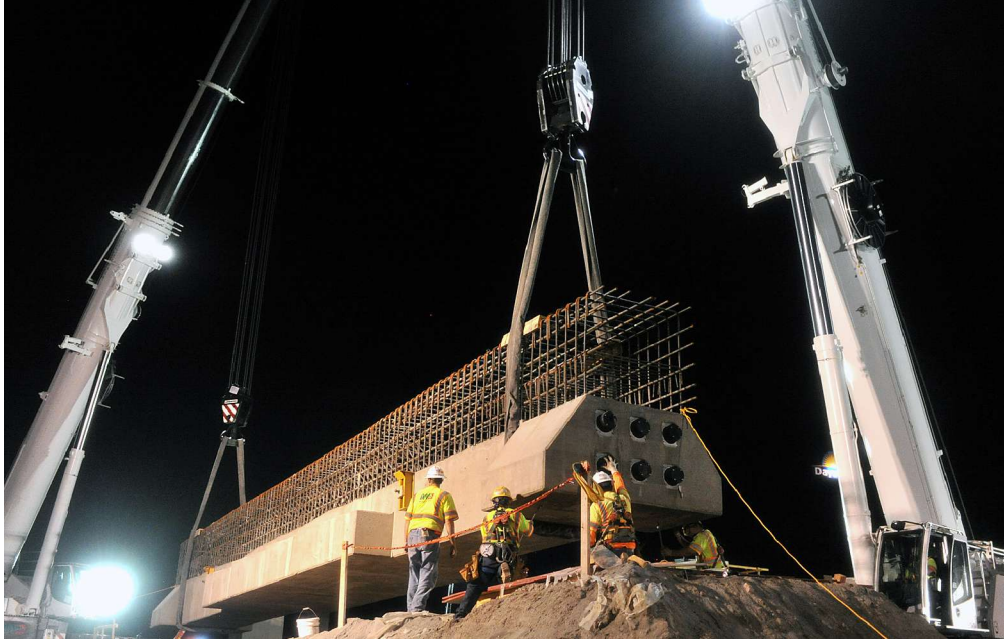
STRUCTURAL COMPONENTS: Mainline bridges used BT72 and BT74 prestressed concrete bulb-tee girders supported on single-column hammerhead piers and drilled shafts or piles; precast concrete pier caps; and precast concrete post-tensioned straddle-beam piers

BRIDGE CONSTRUCTION COST: $131 million

AWARDS: 2018 *Roads & Bridges* #1 Road; Georgia Engineering Excellence Award Winner in Transportation; Georgia Partnership for Transportation Quality Preconstruction Design Award Grand Prize Winner; Precast Concrete Institute 2019 Design Award Main Span More than 150 Feet



Erection of a precast concrete pier cap from the adjacent Interstate 75 shoulder. Photo: Northwest Express Roadbuilders.



Lifting the post-tensioned precast concrete portion of a straddle bent cap from the fill embankment. The top of the embankment was constructed at the same slope as the final cap geometry to control the geometry of the formwork and post-tensioning ducts. The bottom segment of the cap was cast on the ground and lifted into place after post-tensioning. The top portion of the cap was then cast in place. Photo: Northwest Express Roadbuilders.

construction tolerance and to position the precast concrete cap. The cap-column connection was established using a filleted rectangular blockout (with 7.375 in. chamfers at each corner) in the precast concrete cap. Headed dowels, confined by heavy transverse reinforcement, reinforced the connection zone.

The precast concrete cap reinforcement was detailed to avoid conflict with the headed dowels. The size and reinforcement detailing of the connection were designed to utilize standard strength concrete (f'_c = 4.5 ksi). Constructing the pier column from the I-75 shoulder and using precast concrete elements efficiently minimized both traffic disruptions and the environmental impact of the project.

Precast, Post-Tensioned Straddle Bent Caps

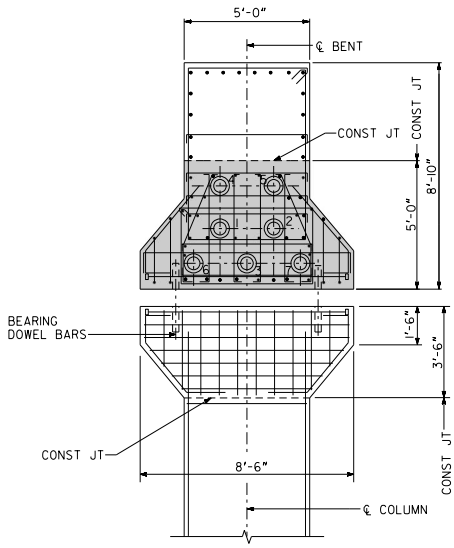
At a second site, an 806.5-ft-long, six-span bridge carries the I-75 express roadway over the existing I-75 southbound lanes. The roadway's horizontal alignment at the bridge is highly skewed with respect to the roadway below and consists of two reverse curves with a short tangent between the curves. The overall width of the bridge varies from 29 ft 3 in. to 35 ft 3 in.

Given these geometric constraints and the presence of existing roadways under the bridge, three types of intermediate piers were used: conventional hammerhead on a single column, inverted-tee hammerhead on a single column, and inverted-tee straddle bent on two columns. The inverted-tee hammerhead pier caps were used in two of the intermediate bents to mitigate the limitations on the available vertical clearance under the bridge.

A straddle bent was selected primarily because of the sharp skew angle and to keep the maximum span lengths around 150 ft, which would make the best use of precast concrete girders. The location of the straddle bent was chosen to minimize its length and to balance loads from the adjacent spans.

To make the straddle bent, a fill embankment with the same slope as the final slope of the precast concrete bent cap was constructed; this facilitated geometric control of the formwork and the post-tensioning ducts. The precast concrete bent cap was cast on site, adjacent to the final bent location, and post-tensioned and grouted while on the ground. The precast concrete cross section was a "U" shape to allow for seven ducts, each with nineteen 0.6-in.-diameter strands tensioned in a single-stage post-tensioning operation. The concrete for the cap was at least 14 days old and had reached a compressive strength of 7 ksi (which was also the 28-day design strength) before tensioning. The "U" shape also minimized the lifting weight (210 kip) and provided part of the formwork for the cast-in-place concrete after the precast concrete member was erected on the columns. The prestressed beam seats were constructed as part of the precast concrete section, with 2-ft 2-in.-wide ledges on each side of the 5-ft-wide core precast concrete section. During a temporary nighttime (11:00 p.m. to 4:00 a.m.) shutdown of the existing southbound I-75, traffic was detoured to adjacent roadways and I-285. During this time, two 300-metric-ton mobile hydraulic cranes were used to lift the cap and place it on top of the columns. The formwork for the cast-in-place portion of the bent cap was then lifted and connected to the precast concrete section. The cast-in-place concrete was placed later with protection measures in place while traffic was maintained below.

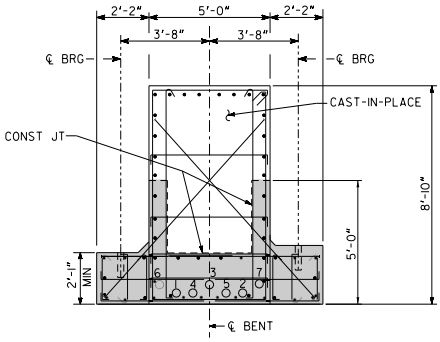
The cast-in-place portion of the bent cap reached a minimum strength of 5 ksi before the prestressed concrete girders were erected. To maintain structural stability of the precast concrete section after erection on the columns, the top of the 5-ft-wide column was flared to 8 ft 6 in. wide in the longitudinal direction. The precast concrete cap width was similarly flared to 8 ft 6 in. to accommodate two bearings at each column.



SECTION AT COLUMN

PRECAST CONCRETE

Cross section of a straddle bent cap. The shaded area is the precast concrete section with seven post-tensioning ducts and longitudinal girder bearing seats. The cast-in-place concrete portion of the section was placed after cap erection and over traffic.



TYPICAL SECTION OF STRADDLE BENT

PRECAST CONCRETE

Conclusion

The Northwest Corridor Express Lanes opened to traffic in September 2018 after 5 years of design and construction. During the first month of operation, more than 20,000 vehicles per day were using the express lanes, far exceeding initial projections. The 29.7-mile-long corridor is the longest installation in the Georgia express lane system and has received glowing reviews from the traveling public and GDOT, which noted that the project has set the bar for future mega-design-build project delivery within Georgia and around the country.

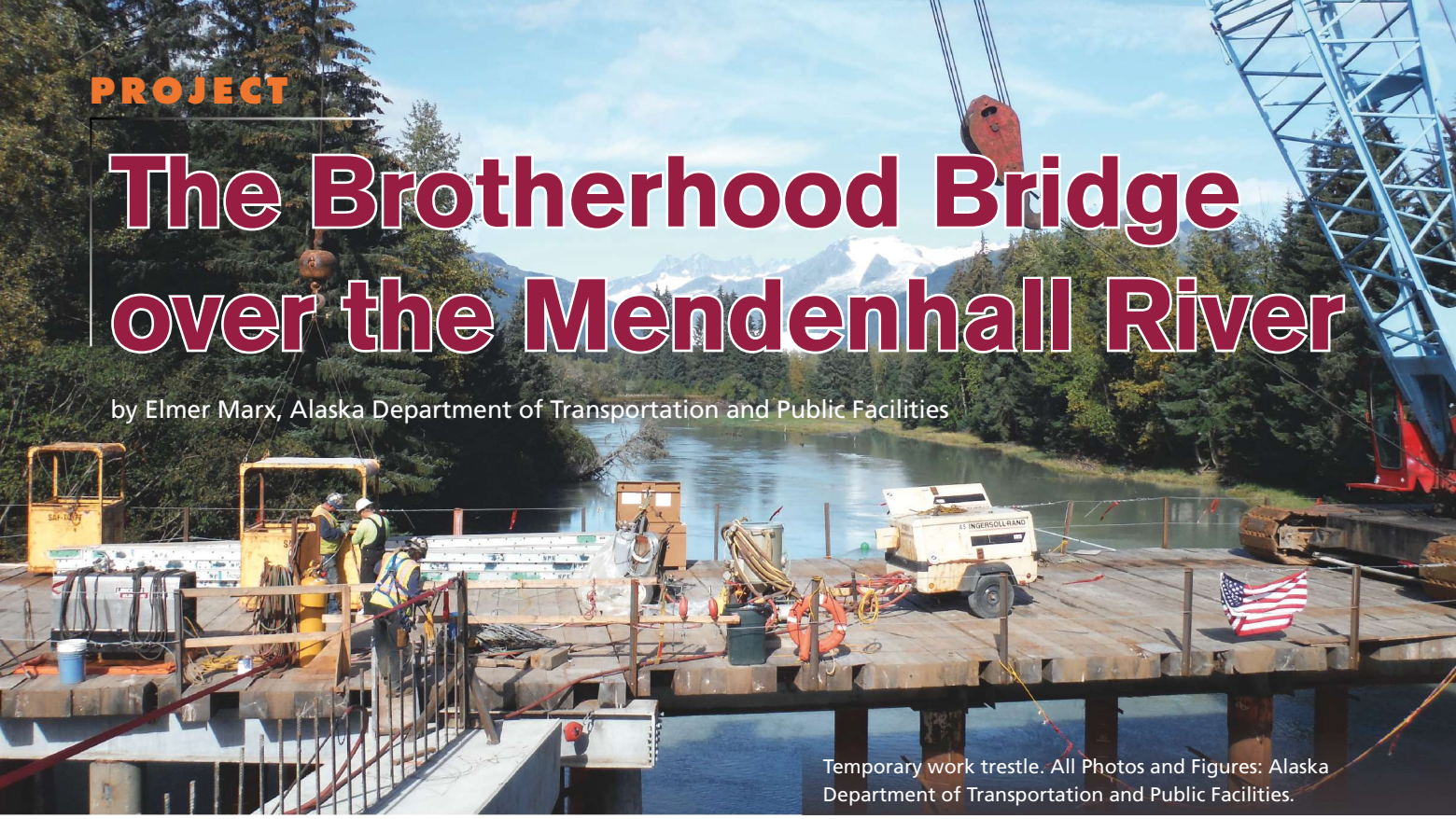
Alan Kite is a senior project manager with Parsons Corporation in Baltimore, Md., and was the lead structural engineer for the Northwest Corridor Express Lanes project.

A straddle bent with flared ends to support two bearings at each end of the pier cap. Photo: Parsons.

PROJECT

The Brotherhood Bridge over the Mendenhall River

by Elmer Marx, Alaska Department of Transportation and Public Facilities



Temporary work trestle. All Photos and Figures: Alaska Department of Transportation and Public Facilities.

The Brotherhood Bridge (also known as the Mendenhall River Bridge) is located near Juneau, Alaska, about five miles downstream of the Mendenhall Glacier. The project, which has interesting historical connections to the region's Native Alaska community, faced several unique design challenges not often encountered outside of Alaska and incorporates many features commonly used by the Alaska Department of Transportation and Public Facilities (AKDOT&PF).

History

Five bridges have been built to cross the Mendenhall River at this location. The first road crossing at the river, a timber trestle, was constructed in 1903 for a total cost of $1700. The timber trestle was reconstructed in 1919 and then replaced with a single-lane, two-span steel truss in 1931. In 1965, the fourth bridge, a two-lane, three-span continuous steel girder structure, was built and named the "Brotherhood Bridge" to commemorate the 50th anniversary of the Alaska Native Brotherhood (ANB).

Roy Peratrovich Jr. was an engineer for the Alaska Department of Highways team that designed the 1965 structure. He is a Tlingit of the Raven Lukaax.ádi clan, the first Alaska Native person registered as a professional civil engineer in Alaska, and the son of prominent Alaska Native civil rights leaders Elizabeth and Roy Peratrovich Sr. In honor of the ANB anniversary, Peratrovich Jr. sculpted 10 bronze medallions with the ANB crest and imagery, which were incorporated into the bridge rail of the 1965 structure.

The bridge served the community well for about five decades. But, as the weight of trucks increased, traffic demands grew, and design code requirements became more stringent, the need for a replacement bridge became apparent. Construction of the replacement bridge began in April 2014. The new bridge, which was rededicated in October 2015, retains the name and decorative bronze medallions that are a key part of the Brotherhood spirit.

Geological and Hydrological Considerations

The Mendenhall Glacier is viewed by more than 500,000 tourists every year. Currently, the face of the glacier is located about 1.5 miles from the Mendenhall Glacier Visitor Center; however, several hundred years ago, the face of the glacier was located near the bridge site. As the glacier has retreated, the surrounding ground surface has risen due to isostatic glacial rebound. At the bridge's location, the ground surface is rising at a rate of about 1 in. per year. Consequently, the river is incising and

profile

BROTHERHOOD BRIDGE (MENDENHALL RIVER BRIDGE) / JUNEAU, ALASKA

BRIDGE DESIGN ENGINEER: Alaska Department of Transportation and Public Facilities Bridge Section, Juneau

PRIME CONTRACTOR: Orion Marine Group, Anchorage, Alaska

PRECASTER: Concrete Technology Corporation, Tacoma, Wash.—a PCI-certified producer

GEOTECHNICAL AND FOUNDATION CONSULTANTS: Jacobs (formerly CH2M Hill), Bellevue, Wash.

OTHER MATERIAL SUPPLIERS: Formliner (enlarged version of medallions): Spec Formliners Inc., Santa Ana, Calif.; elastomeric bearing pads: Seismic Energy Products LP, Athens, Tex.

Ten bronze medallions from the 1965 bridge commemorating the 50th anniversary of the Alaska Native Brotherhood were reused in the new bridge railing. The sculptor, Roy Peratrovich Jr., was a member of the Alaska Department of Highways team that designed the 1965 structure. He was the first Alaska Native person registered as a professional civil engineer in Alaska.

lowering the streambed surface below the bridge—that is, the exposed length of the piers will increase over the life of the bridge. This phenomenon had to be considered when planning for pier scour over the design life of the structure.

The recession of the Mendenhall Glacier has resulted in several unusual flood-like events that have followed the outbursts of glacier-dammed lakes. Such events are known by the Icelandic term *jökulhlaups* (pronounced "yo-KOOL-lahps"). The Mendenhall River jökulhlaups have caused floodwater surface elevations above those of the 50-year flood event and accelerated erosion along the riverbanks (which are partially lined by residential structures). Predicting these events can be difficult given the uncertainties of lake volumes, fill rates, glacier buoyancy, and other contributing factors. In addition to the transport of large wood debris, jökulhlaup floodwaters may include large chunks of ice.

The previous bridge substructure was founded on driven steel H-piles that penetrated up to 70 ft beneath the streambed. The effects of local pier scour, isostatic glacial rebound, and jökulhlaups undermined the pier footings of the previous bridge and were among the contributing factors for why it had to be replaced.

Geotechnical and Seismic Considerations

Much of Alaska's coastline is located on the Pacific "Ring of Fire." Consequently, Alaska is subjected to frequent and strong ground shaking. Although the seismic hazard at the bridge site (peak ground acceleration ~0.2*g* for the 7% probability of exceedance in the 75-year design event) is not particularly large compared to other places in Alaska, it was predicted to be enough to generate soil liquefaction below the bridge.

Geology at the bridge site is composed of over 150 ft of nonplastic silts and sands. Blow counts from standard penetration tests within the silts and sands were generally under 15 blows per foot; cone penetrometer test soundings and shear wave velocity measurements confirmed very soft to loose soil conditions.

Preliminary analyses showed that sediment liquefaction could be triggered at depths in excess of 100 ft and might result in free-field lateral spreading of between 4 and 6 ft at bridge abutments and center piers. At this level of deformation, the bridge could not be guaranteed to satisfy the American Association of State Highway and Transportation Officials (AASHTO) requirements for noncollapse. Therefore, some form of liquefaction mitigation was required.

ADOT&PF worked with CH2M Hill (now Jacobs) to examine various approaches to address the seismic demands at the site. Pseudostatic slope stability analyses using limit equilibrium methods were conducted to evaluate the seismic stability of the abutment during and immediately after the design seismic event.

The response was evaluated for two conditions. The first considered flow failure. In this case, the soil liquifies and loses strength when the ground is not shaking. This case has been observed when liquefaction is slow to develop, sometimes occurring several minutes after the end of the earthquake. The second condition considered lateral spreading, where reduction in soil

Installing the precast, prestressed concrete decked bulb-tee girders during nighttime construction. During the summer in Alaska, there is still daylight or twilight during "nighttime" construction.

ALASKA DEPARTMENT OF TRANSPORTATION AND PUBLIC FACILITIES, OWNER

BRIDGE DESCRIPTION: Three-span, prestressed concrete decked bulb-tee girder superstructure on reinforced-concrete-filled steel pipe pile extension bents

STRUCTURAL COMPONENTS: Forty-two 65-in.-deep decked bulb-tee girders; twenty-eight 2-ft-diameter reinforced-concrete-filled steel pipe piles at the abutments; ten 4-ft-diameter reinforced-concrete-filled steel pipe piles at the piers; cast-in-place concrete pile caps, diaphragms, traffic barriers, and approach slabs

BRIDGE CONSTRUCTION COST: $11 million ($294/ft^2)

AWARD: Outstanding Project of the Year Award, American Society of Civil Engineers, Alaska Section, Juneau Branch

strength and ground shaking occur in combination. These analyses were performed without consideration of the "pinning" effects from the abutment or pier piles. Both circular and noncircular failure surfaces were evaluated.

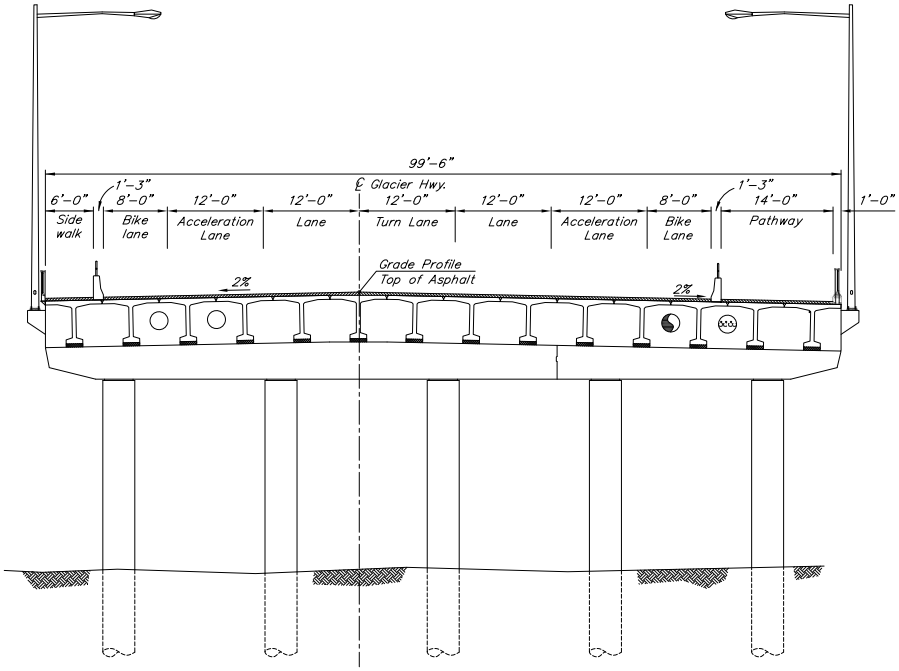
Once the "free field" condition had been better quantified, the option of pile pinning was examined. In the pile-pinning approach, restraint is provided by the bridge superstructure (acting as a strut) and by the lateral resistance of reinforced-concrete-filled steel pipe piles displaced during slope movement, which results in stabilizing forces to slope movement. This approach provided bridge stability without the need for expensive ground improvement work. Preliminary assessments found that the depth and location of ground improvement would cost over $2 million. Given the costs, environmental constraints, and schedule limitations, pile pinning was evaluated to provide a cost-effective approach for the liquefaction hazard. The use of pile pinning has become a frequent approach for other AKDOT&PF bridge projects.

Design and Construction

The Brotherhood Bridge is located near a popular pedestrian trailhead and area for viewing/photographing the Mendenhall Glacier. The new three-span structure is 378 ft long, with two lanes in each direction and a center turn lane. In addition to traffic, the bridge also carries water, electric, and telecommunication utilities. It has 8-ft-wide shoulders, a 6-ft-wide sidewalk on the downstream edge of the bridge, and a 14-ft-wide multiuse pedestrian pathway on the upstream edge for a total bridge width of 99.5 ft.

Steel and concrete options of various span configurations were considered for the replacement bridge. Ultimately, a three-span, precast, prestressed concrete decked bulb-tee girder superstructure supported on reinforced-concrete-filled steel-pipe-pile extension bents was found to be the most cost-effective solution for the site.

Precast, prestressed concrete decked bulb-tee girders have been used in Alaska since the mid-1970s. In addition to being more durable than bridges with cast-in-place decks, bridges with



Typical section of the precast, prestressed concrete decked bulb-tee girder superstructure that is supported on reinforced-concrete-filled steel pipe pile extension bents.

predecked girder cross sections can be constructed rapidly, an important consideration given Alaska's cold climate and short construction season. The girders were fabricated at Concrete Technology Corporation's Tacoma, Wash., facility. This facility is located on a maritime waterway and has direct access from the fabrication floor to an awaiting barge. All 42 girders were shipped on one barge. The barge arrived at the Juneau dock where the girders were loaded on trucks and driven approximately ten miles to the bridge location. In Alaska, girders up to 165,000 pounds are commonly transported in this manner.

AKDOT&PF designs these sections to a zero-tension stress limit with the AASHTO LRFD Service III load combination using gross-section properties. The specified 28-day concrete compressive strength of these sections is typically a minimum of 7.5 ksi, with fabricators commonly achieving strengths of 10 to 12 ksi. This high-strength, high-durability concrete is used in the entire concrete section, including the monolithic deck. The three precast producers that provide decked girders for projects in Alaska (two in Anchorage, Alaska, and one in Tacoma, Wash.) have the capability to deflect (sag) the forms downward to offset prestress camber as a method to achieve the roadway profile grade needed for the roadway surface. Leveling inserts are provided in the girders to provide a method of adjusting differential camber between girders, if needed. High-strength (9 ksi) grout is used in the longitudinal keyways between the girders. The combination of using the zero-tension stress limit for design and high concrete strength has resulted in a very durable, nearly maintenance-free superstructure that is well suited to Alaska's severe environment.

For several decades, AKDOT&PF has used reinforced-concrete-filled steel

Completed first-stage pier using reinforced-concrete-filled steel pipe piles with reinforcement extended from top of cap to engage the diaphragm between girders.

Aerial view of the bridge with the first stage of construction nearly complete; the work trestle is on the left, and the old bridge is on the right.

pipe pile extension substructures (piles typically range from 2 to 4 ft in diameter for conventional bridges). Because this type of substructure does not require cofferdams, it can be built in less time than more traditional, pile-supported footings. The steel pipe acts as a stay-in-place form for the reinforced-concrete core and is included in the cross-sectional capacity of the members. The upper lengths of the pipe (exposed length and 10 ft below the minimum streambed surface) are hot-dip galvanized. AKDOT&PF requires the use of steel pipes that comply with either the American Petroleum Institute (API) 5L PSL2 X52 or ASTM A709 Grade 50T3 fabricated to API 2B or AKDOT&PF spiral (helical) weld specifications to provide superior seismic performance. Research performed at North Carolina State University developed strain limits of reinforced-concrete-filled steel tubes (RCFSTs) associated with the onset of pipe wall buckling and fracture (for more information, see the Concrete Bridge Technology article on RCFSTs in this issue of *ASPIRE®* on page 32). These composite concrete-steel sections are stiffer, stronger, and more ductile than conventional reinforced concrete or hollow steel members and are better able to accommodate large lateral demands from ice, earthquakes, and vehicle collisions.

The project included construction of paths below both sides of the Brotherhood Bridge to allow pedestrians and bicyclists to safely cross under the highway along the riverbanks. Mechanically stabilized earth walls are utilized at each abutment to improve pedestrian access under the bridge. The bridge abutments are supported by fourteen 2-ft-diameter concrete-filled pipe piles under each cap beam to accommodate the large seismic demands, including predicted pile-pinning action and lateral soil movement.

Above the bedrock at the site was 290 ft of very loose soils. Based on the two-dimensional, site-specific seismic analysis, the five 48-in.-diameter pipe piles driven to bedrock and partially filled with concrete were found to be the most cost-effective foundation type for the bridge piers. Only the top 85 ft of the pipe piles are filled with reinforced concrete. The design required concrete to be placed below the ground line to a point where the moment is less than half the maximum plastic hinging moment accounting for the effects of scour, liquefaction, and other factors that lower the plastic hinge location.

Because the AKDOT&PF right-of-way is relatively narrow at this crossing, staged half-width construction was selected to allow traffic during construction. About one-third of the new bridge was built upstream of the old steel girder bridge without affecting traffic. Once the first stage of construction was complete, vehicular and pedestrian traffic was routed over the new bridge. The old bridge was demolished, and the steel was sent to a recycling facility outside of Alaska. The rest of the new bridge was constructed and traffic was placed in the final configuration.

Since construction was staged, full designs were completed for the two bridge configurations, including evaluation of the seismic response of both stages, to ensure safe and adequate performance during the entire construction process.

During construction of the bridge, a *jökulhlaup* occurred, carrying large amounts of wood and debris. Fortunately, the bridge contractor was able to quickly respond to the unexpected event and removed debris around the work trestle during the outburst and for several days after it.

The deck and cast-in-place approach slabs are covered with a waterproofing membrane and 4 in. of asphalt concrete. Concrete formliner replicas of the ANB medallions were fabricated and used in the wingwalls. Cast-in-place end, pier, and intermediate diaphragms are used in the superstructure. The abutments employ a semi-integral abutment concept that results in a jointless bridge and subsequently reduced maintenance needs.

The total cost of the project, including demolition of the existing bridge, was approximately $11 million, which corresponds to about $294 per square foot of bridge deck area. Conventional highway bridges in Alaska typically cost between $300 and $350 per square foot. Considering the use of staged construction, decorative railing, long piles, and poor soil conditions, the Brotherhood Bridge proved to be a very cost-effective structure. The high-strength precast concrete decked bulb-tee girders will provide a durable bridge requiring minimal maintenance over the structure's design life.

Elmer Marx is a senior bridge design engineer with the Alaska Department of Transportation and Public Facilities in Juneau.

Aerial view of the bridge carrying traffic while the second stage of construction was underway.

PROJECT

Sarah Mildred Long Bridge

The movable midspan, elevated clearance, and reduced number of piers of this replacement bridge over the Piscataqua River improve marine navigation

by Jeffrey S. Folsom, Maine Department of Transportation

Built in 1940, the original Maine–New Hampshire Bridge spanning the Piscataqua River between Portsmouth, N.H., and Kittery, Me., served as the critical backup route between Maine and New Hampshire if traffic were disrupted on the Interstate 95 Bridge upstream. In 1987, the Maine–New Hampshire Bridge was renamed to honor Sarah Mildred Long, an employee of the Maine–New Hampshire Interstate Bridge Authority for 50 years.

Three decades later, on March 30, 2018, a replacement bridge, also named for Long, opened. Its innovative design would have made her proud. The previous structure had five main truss spans, including the vertical lift span, along with 15 roadway spans on the Portsmouth side and seven on the Kittery side. The new Sarah Mildred Long Bridge has twin segmental-concrete approach spans in a stacked configuration leading to a single-deck hybrid movable span at its center.

The new structure's design, which provides a unique concept for a segmental bridge, improves the skew angle and width of the navigation span as well as safety and efficiency for vehicles, rail lines, and maritime vessels. With the new bridge, maritime vessels can navigate the Piscataqua River while the upper segmental-approach superstructure carries U.S. Route 1A (Bypass) and the lower segmental superstructure connects Pan Am Railways, a railroad system for the Northeastern United States, to the Portsmouth Naval Shipyard. The movable hybrid span rises to allow passage of tall vessels and lowers to railroad-track level for trains to cross, remaining at the vehicle level in its resting position. The Portsmouth, Me., approach spans extend over Market Street and the Pan

The new Sarah Mildred Long Bridge between Maine and New Hampshire over the Piscataqua River features twin segmental-concrete approach spans in a stacked configuration leading to a single-deck, hybrid movable span at the bridge's center. Photo: FIGG Engineering Group.

profile

SARAH MILDRED LONG BRIDGE / PORTSMOUTH, NEW HAMPSHIRE, AND KITTERY, MAINE

BRIDGE DESIGN ENGINEERS: FIGG Bridge Engineers, Tallahassee, Fla., and Hardesty & Hanover, New York, N.Y., a joint venture

PRIME CONTRACTOR: Cianbro, Pittsfield, Me.

POST-TENSIONING CONTRACTORS: GZA GeoEnvironmental, Boston, Mass.; Sebago Technics, Portland, Me.

Am Railways' Newington Branch railroad tracks. The Kittery approach spans cross Bridge Street.

Selection of a segmental concrete solution helped project leaders achieve many of their goals for the bridge. It allowed them to fine-tune both horizontal and vertical alignments to improve the navigation span, meeting geometric standards related to both railroad and vehicular traffic, and it minimized the impact of the project on the environment, historic and archaeological sites, and nearby properties. The use of concrete also offered a context-sensitive, aesthetically pleasing solution.

Throughout the design process, the team gained insight from extensive public involvement and government agency outreach to the community, regulatory interests, and ship pilots. This input helped guide designers and contractors to a design that meets all of the project goals while creating a striking design with a unique lift concept that can serve as a template for future projects of this type. Details of the design and construction of the lift tower foundation can be found in a Concrete Bridge Technology article on page 34 of this issue.

Project Partners, Funding, and Vendors

The replacement bridge project was a partnership among the Federal Highway Administration, the Maine Department of Transportation (MaineDOT), and the New Hampshire Department of Transportation, with MaineDOT serving as the lead agency. Funding for the $165 million project was aided by a $25 million Transportation Investment Generating Economic Recovery (TIGER) grant for the railway portions. The project was the second in a "Three Bridge Agreement" between Maine and New Hampshire to address their jointly owned bridges spanning the Piscataqua River.

The new bridge was designed by a joint venture of the specialty bridge design firms FIGG Bridge Engineers and Hardesty & Hanover, and it was delivered through a new construction manager/general contractor method, in which a construction manager (Cianbro Corporation), chosen through an RFP process, was an active participant in the design process.

Design work began in late 2012. Early on, Cianbro provided constructability reviews and input on scheduling parameters, cost elements, and other factors. The goal was for the construction manager to provide a reality check on the constructability and economy of the design as it evolved.

When Cianbro became construction manager, the owners and Cianbro negotiated a construction contract, with the intent that if they could agree on a price, Cianbro would also be the general contractor. If no agreement were reached, the owners would advertise for competitive bids. Cianbro prepared cost estimates at 30%, 60%, and 100% design plan reviews. The estimate prepared at 100% was the basis of negotiation. The final pricing and negotiations resulted in a few plan and specification changes that were incorporated into one final released-for-construction set of contract documents. It was at that point that a construction contract was signed and Cianbro became the general contractor. Construction commenced in late 2014.

Improving Maritime Clearance and Accessibility

Clearance and accessibility for maritime vessels were among the primary concerns tackled by the design team. The original bridge offered just 10 ft of clearance in the closed position, which meant that bridge openings were frequently required so maritime traffic could pass. In 2008, the bridge opened 2637 times, with an average traffic delay of 9.5 minutes per opening. The new bridge offers 56 ft of vertical clearance in its resting position. As a result of the increased vertical clearance, 68% fewer openings will be required with the new structure. The bridge can remain at its resting position, the vehicle level, almost all of time because only 10 trains typically use the lower-level tracks each year.

The previous bridge provided only 175 ft of useable width for maritime traffic, forcing tugboats to disengage from larger ships before passing through the span. To improve accessibility for ships approaching the open span, a new alignment was created that provides a 15-degree skewed crossing to the bridge (the original crossing skew was 25 degrees). The alignment change increases useable width to 250 ft.

To improve accessibility for ships approaching the open span, a new alignment was created that provides a 15-degree skewed crossing to the bridge.

The larger opening was achieved with a 4770 ft reverse curve on the approach bridge on the New Hampshire side and a 5200 ft radius curve on the Maine side, which connects back into the existing alignment. This design allows the next generation of cargo vessels to pass through the span with tugboats engaged.

Span Design

The new bridge features 11 upper vehicle spans: 160-270-270-320-320-35 (tower span)-300 (lift span)-35 (tower

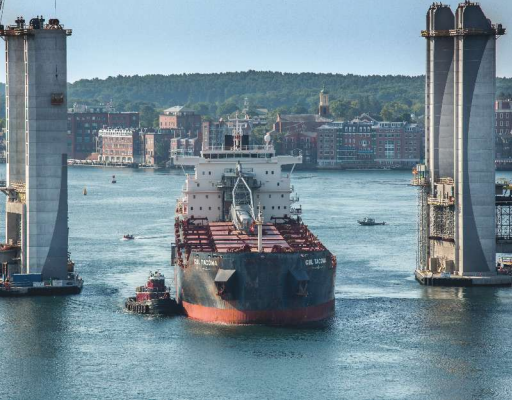
MAINE DEPARTMENT OF TRANSPORTATION AND NEW HAMPSHIRE DEPARTMENT OF TRANSPORTATION, OWNERS

BRIDGE DESCRIPTION: 1990-ft-long bridge with twin segmental-concrete approach spans in a stacked configuration leading to a single-deck, hybrid movable span at the bridge's center. The lift span is supported by four 200-ft-high precast concrete towers at the span's corners.

STRUCTURAL COMPONENTS: 229 precast concrete segments in 11 upper vehicle spans: 160-270-270-320-320-35 (tower span)-300 (lift span)-35 (tower span)-200-320-200 ft; and 126 precast concrete segments in 16 lower railway spans: 60-100-160-160-160-160-135-65-35 (tower span)-300 (lift span)-35 (tower span)-65-135-160-160-100 ft. Each tower consists of 21 precast concrete match-cast segments.

BRIDGE CONSTRUCTION COST: $165 million

AWARDS: 2018 International Bridge Conference Special Award of Merit; 2018 *Roads and Bridges Magazine* Top Ten Bridges



A ship passing through the movable span during construction shows the added width for maritime traffic and the size of ship that needs to be accommodated. Photo: Maine Department of Transportation.

A new alignment was created for the replacement bridge, reducing the crossing skew to 15 degrees for approaching ships. Photo: Maine Department of Transportation.

The upper vehicular spans feature precast concrete segmental box girders with depths varying between 8 and 13.5 ft and base widths varying from 15 to 16.75 ft. Photo: Maine Department of Transportation.

span)-200-320-200 ft. The lower railway level consists of 16 spans: 60-100-160-160-160-160-135-65-35 (tower span)-300 (lift span)-35 (tower span)-65-135-160-160-100 ft. Many of the railway level's span lengths are approximately half the length of the vehicle level to accommodate the heavier (Cooper E80) required live load. The goal was to maximize span lengths on each level to reduce the number of piers in the waterway.

The 160 ft length for the railway spans is significant for a segmental bridge and allowed for only an intermediate monoshaft pier to be used at the midspans of the upper level's 320 ft lengths. As a result, 11 fewer piers were needed in the waterway, and an existing median pier on Market Street was eliminated, improving the gateway span leading into downtown Portsmouth.

The upper vehicular spans feature precast concrete segmental box girders with depths varying between 8 and 13.5 ft and base widths varying from 15 to 16.75 ft. The lower railway spans consist of precast concrete segmental girders varying in height from 9 to 11 ft, with top flanges of 19 ft and base widths varying from 11.33 to 12 ft. The single-level lift-span deck is 42 ft 7 in. wide, with two lanes separated by a 5-ft 7-in.-wide median that contains the railroad tracks.

Each lift tower consists of 21 precast concrete segments cast at the project site using match-casting procedures. Photo: Maine Department of Transportation.

Modified Tower Drive System

Several options were considered for the drive system to operate the central 300-ft-long streamlined box-girder lift span. Typically, two options are available. With a span drive, the machinery is located on the span, raising and lowering itself with the span. With a tower drive, the machinery, located at the tops of the towers, raises or lowers the span. For this project, a hybrid design provided the best of both worlds.

The operating equipment is located at the base of the four 200-ft-tall precast concrete towers, on each corner of the span. The machinery was placed at the bases, providing easy access for maintenance and inspection. Similar to a span drive, the equipment uses uphaul and downhaul ropes to physically pull the span up and down and does not rely on the friction of the counterweight sheave to move the span. As with a tower drive, the equipment is enclosed within the towers, including mechanicals, wiring, and even the maintenance stairway. The counterweights were built with steel plates inside a steel box. Steel was selected after evaluating it against other options, such as lead, to determine the cost-benefit ratios and compare densities to the total heights of the materials that would be needed. The towers feature glass windows that provide ambient lighting for maintenance crews and also allow drivers to see the counterweights moving when the span is raised and lowered.

The lift span features guide rollers that

run on plates attached to the towers to maintain the transverse and longitudinal position of the span as it is raised and lowered. The span joins at the primary (vehicle) resting level with a standard finger joint. Openings allow the tower rails to pass through the span and continue down to the railroad level. At that point, a plate with mitered rails connects to the tracks. It was challenging to detail this configuration because joints usually are not designed to have tracks pass through them and continue below.

Cost Efficiency of Precast Concrete Towers

Precast concrete was chosen for the towers after all possibilities were evaluated. This option was most efficient and cost effective because all the equipment and setup needed for erection, post-tensioning, and epoxy joining for the tower segments were already on site for the precast concrete segmental superstructure.

Each tower consists of 21 precast concrete match-cast segments. Tower segments were cast at the project site using standard segmental concrete match-casting procedures. Forms were stripped from wet-cast segments and moved into place to serve as a match-cast segment. After the first three segments were erected and adjusted to the proper line and grade, a 3 ft starter segment was cast below to establish the baseline erection geometry. This was necessary to begin erection of the tower segments. The match-cast precast concrete tower segments needed to be placed on top of the cast-in-place footing. To ensure the correct starting geometry, the segments were erected on temporary supports and properly adjusted, with a short closure pour between the precast concrete tower segments and the cast-in-place footing. Erection geometry was monitored and adjusted as necessary to maintain erection tolerances. The towers were capped with a precast concrete tower cap with cast-in-place concrete topping.

Jeffrey S. Folsom, P.E., is assistant bridge program manager with the Maine Department of Transportation in Augusta.

AESTHETICS **COMMENTARY**

by Frederick Gottemoeller

The Sarah Mildred Long Bridge is an impressive bridge that meets a set of complex functional requirements while achieving a high level of visual quality suitable for its location within an area that encompasses two historic cities and an attractive natural setting. Meeting both the functional and aesthetic requirements of the project within a reasonable budget required innovation in layout, design, contracting arrangements, and construction. That the design, construction, and client team was able to meet all of these requirements so successfully is a credit to all involved.

Projects with this kind of functional complexity often have a corresponding and unattractive visual complexity. The old bridge was an example of that type of design. On the new bridge, concrete segmental construction offered a solution to the aesthetic challenges. On the spans, it allowed for fewer piers and simplified the appearance of the girders. On the towers, it eliminated the usual cross bracing and concealed the lifting equipment. At the same time, the haunched girders, which are deeper at the piers where the forces are the greatest, and the solid towers, rising from a massive base, provide an impression of great strength. This impression is reinforced by the simple but robust modulation of the concrete piers.

There is also a kind of delicacy at the tops of the towers, which taper to reveal the counterweight sheaves. Because the sheaves are a visual feature, the design conveys that the bridge is meant to move. The sheaves are round, and things that are round rotate. Why else would they rotate but to lift the center span? Finally, the vertical strips of tower windows that show the counterweights moving are visual compensation to drivers stuck in the traffic backup as a ship or train passes through the crossing. Sarah Mildred Long would indeed be proud of the bridge bearing her name.

Observations from 30 Years of Inspecting Post-Tensioned Structures

by Bruce Osborn, Structural Technologies LLC/VSL

During the 1970s, the construction of post-tensioned (PT) concrete structures rose in popularity as the technology allowed for the efficient construction of longer spans with smaller concrete sections and less mild-steel reinforcement. While this technological advancement has provided contractors and owners with a durable and cost-effective solution, it has not been without its challenges.

Since their inception, post-tensioning technology and specifications for PT bridges have evolved to ensure that long-lasting structures are being built with higher-quality installation procedures. Early failures of PT systems such as the Niles Channel Bridge (1999), Sunshine Skyway Bridge (2002), Varina-Enon Bridge (2007), and Mid-Bay Bridge (2011) pushed owners to implement closer inspection of PT systems.

The following article shares the author's firsthand accounts of inspections of over 70 bridges and approximately 18,500 tendons. The author has inspected PT structures in some of the most benign and harshest environments from Alaska to Puerto Rico, and in many locations in between. The condition of each PT bridge was determined through analysis of data gathered from destructive and nondestructive investigations performed on external and internal post-tensioning systems. In the former type of system, tendons run within the void in box girders or cable-stayed bridges; in the latter type, ducts for tendons are cast within the structural concrete section. The findings illustrate the evolution of PT structures since the 1970s, common types of deficiencies, and how shifts in quality-control procedures have improved installation and extended the life of PT structures.

Figure 1. Bare strand in a tendon at a grout void. All Figures: Bruce Osborn.

Unforeseen Challenges

In the early years of post-tensioning, stressing was regarded to be the most important aspect of the operation, and other details were believed to not be as structurally significant. Even though grouting served as another layer of protection against corrosion of the PT strands, it was often treated as an afterthought and performed with limited supervision or inspection from an outside party. Because the construction and inspection standards of past decades do not meet today's requirements, inspection of older PT structures is important.

Prior to 2000, grout used for tendons was a simple mixture of Type I or II cement and water. In some cases, additives were used to try to improve the protective characteristics of the grout. In later years, it was proven that this combination of materials had segregation issues that caused excessive bleeding in the tendons. The presence of bleed water increased the risk of voids during the grouting process, thereby making the strands vulnerable to corrosion. Voids discovered in this time period were typically credited to excessive bleed water, improper venting, poor grouting practices, or inadequate grout materials.

Notably, when inspections identified voids in the grout within tendons, those voids were not always associated with significant corrosion. In most cases, voided areas could be regrouted to extend the service life of the structure and the PT system within it. **Figure 1** shows a borescope picture taken from the inside of a voided tendon in a structure whose exterior condition was well maintained. The void is located at the top of the duct, and the steel within the tendon has not undergone any major corrosion. This type of void is most

Figure 2. Bare strand with clear signs of corrosion in a tendon with a grout void.

Figure 3. Operation to find cracks in the bridge that were leading to moisture in post-tensioned tendons.

Figure 4. Equipment used to pressurize tendons and find the source of water.

often caused when improper grout venting results in air entrapment at a high point. To repair this type of condition, grout ports are drilled into the duct and vacuum grouting is used to ensure a proper seal. When there is seemingly no corrosion, this manner of repair is very beneficial.

Of the bridges inspected, tendon deterioration could typically be attributed to severe cracking of the deck surface at pier locations, which allowed contaminants to enter the metal duct and access the grout and tendon. As water entered the duct, it often traveled the length of the tendon in a process known as "wicking"; this same process can also occur in the seam of a metal duct. **Figure 2** shows an example of corrosion caused by water traveling along the strand and duct into the voided area. In areas where road deicers are used, this phenomenon is a major concern. Once project stakeholders became more aware of this issue, inspections were performed more regularly on this structure's PT tendons. Analysis of the damaged condition by qualified engineers is also prudent.

Noteworthy Findings

Inspections of structures constructed during an era with lower grouting installation standards and supervision illustrate how tendons that were poorly grouted have held up over time. These structures were of different construction types, ranging from precast segmental concrete to cast-in-place concrete boxes on falsework and straddle bents. In one Midwestern state, 17 structures and 233 tendons were inspected. The tendons contained 466 anchorages and 340 high points. For all of the bridges and tendons inspected in this state, 61 voids were located—with some being in the same tendon at different locations. Despite the number of voids found, there were no documented strand or tendon failures in these structures, most likely because of details of the structural high quality of the structure's upkeep. The owner is currently in the process of having the voided areas grouted and the decks sealed to lengthen the service life of these structures.

Inspections by the author have also revealed that a substantial number of the observed tendons with section loss arose from conditions outside of the PT system itself. Cracks, which occur throughout the life of the structure, can accelerate the intrusion of environmental risks to the PT system. Environmental conditions or the general upkeep of the structure surrounding the PT tendons, and not the PT tendons themselves, were typically the root cause. Tendons that are sealed performed much better.

Figures 3 and **4** show two photos from a structure in which large amounts of water collected in the PT ducts. It was not known how the water was infiltrating the system, but there were fears that the PT system was at fault. As part of the investigation, ports were installed into the ducts, which were then pressurized, to seek the source of the water intrusion. After reaching around 40 psi, water began discharging out of the top deck of the bridge (soap was placed around area to make any exiting air visible). This evidence demonstrated that the water-filled ducts were being recharged through cracks in the bridge deck.

One of the most prevalent design problems in PT bridges is scuppers and deck drainage systems that run inside the structure or terminate at critical locations on the superstructure. Over time, these drains fail, transferring road deck debris and water either to the interior of the structure or

Figure 5. Signs of tendon corrosion near a bridge drainage system.

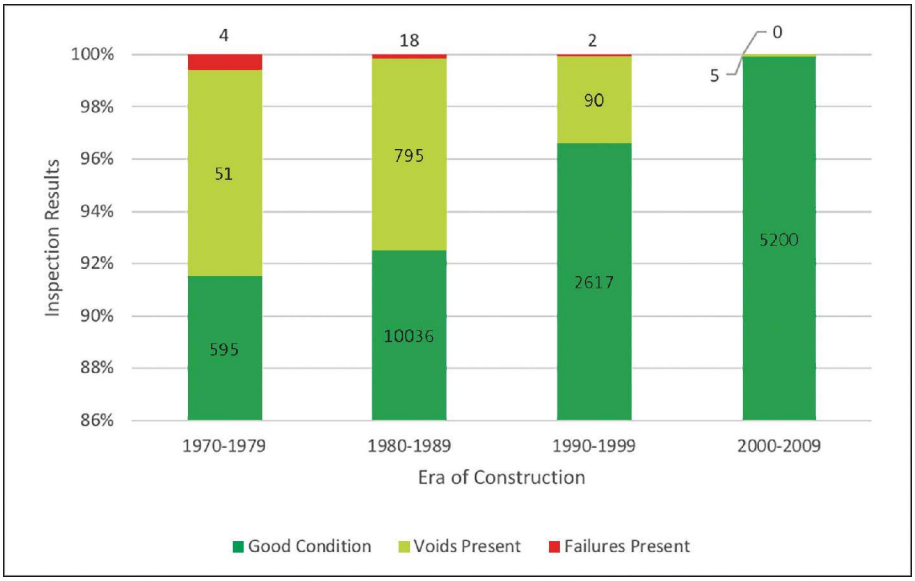


Figure 6. Condition of post-tensioning tendons inspected by the author by construction time period.

onto the superstructure (**Fig. 5**). Cracks consistently exposed to outside contaminants such as road salts and chlorides will grow, further endangering the tendon.

One example of a bridge with a PT system with severe tendon corrosion was the Cline Avenue Bridge, a cast-in-place concrete box structure built on falsework. For several years before the tendon failure was found, inspections had noted deterioration, which was related to a leaking drainage system. Staining was visible on the wall where the water collected because the structure did not contain drain holes. Also, layers of salt were visible on the floor from the leakage and rehydration process. At some point, workers tied rags wrapped with duct tape to the leaking drainage pipe in an attempt to seal it. When it was determined that tendons were located in the same area as the leak, an inspection was performed.

When the Oneida River Crossing structure was inspected in 2007, a few tendon failures were identified. This structure is one of the oldest PT girder-type structures in the United States, built in the 1960s. In this case, the drain scuppers were terminated just above the girder flange and the tendons were encased in a galvanized metal duct. Years of deicing salts applied to the roadway and spreading onto the concrete flange caused the concrete to deteriorate, followed by corrosion of the duct and ultimately corrosion of the tendon. There was no indication of voids being originally present in the failed area.

Advancements in Post-Tensioning

Although tensioning of the tendons is perhaps the most important factor for the strength of the structure, grouting is the most critical for the lifespan and durability of the structure. In the current era, several modifications have been made to improve PT installation and help eliminate issues in the tendons. One major achievement was the introduction of the plastic extruded duct material with airtight sealed connections. This innovation helps prevent intrusion of water or any outside contaminants into the tendons. Another advancement was the introduction of prepackaged thixotropic grouts. These new grouts have limited chloride content, low permeability, zero bleed and excellent pumpability in normal grouting operations.

Another achievement over the past 30 years has been the proactive approach of the American Segmental Bridge Institute (ASBI) and the Post-Tensioning Institute (PTI) to improve the quality of grouted PT structures. These two institutions have developed and continue to refine grouting specifications and procedures as well as field certification programs aimed at eliminating problems with improper materials and incorrect installation of grouted systems. These efforts have served to better educate all parts of the industry and to improve grouting quality. PT tendons are now being grouted under strict guidelines, and detailed reports are required on the grouting operation itself. Testing throughout the operation is required in most states; therefore, if any issue arises in the future, it should be possible to retrieve the original records for investigations. The current state of the practice for grouting can be found in PTI M55.1-12, *Specification for Grouting of Post-Tensioned Structures*, as well as PTI/ASBI M50.3-12, *Guide Specification for Grouted Post-Tensioning*. These documents, in addition to ASBI and PTI installer certification programs, are major resources for owners that are building and maintaining PT structures.

Figure 6 presents the conditions of the PT tendons investigated by the author grouped by the date of original construction. The vertical axis has been shortened to focus on the upper limits to allow the small quantities to be visible. Even though data in the figure are just for structures inspected by the author, it is evident that the condition of the structures significantly improved over time with refinement of PT systems and procedures, as indicated by the reduced incidence of voids in tendons.

Conclusion

After decades of inspecting PT systems, the author can comfortably assert that, when properly designed, detailed, and installed under the guidelines and procedures published as consensus-based best practices by industry leaders, PT structures are durable, reliable, and cost-effective options for many bridge projects. Within the last 20 years, significant innovations and improvements in PT systems have resolved challenges encountered when the technology was new and spurred growth in the industry. As a result, today's PT systems are designed and constructed to withstand severe conditions.

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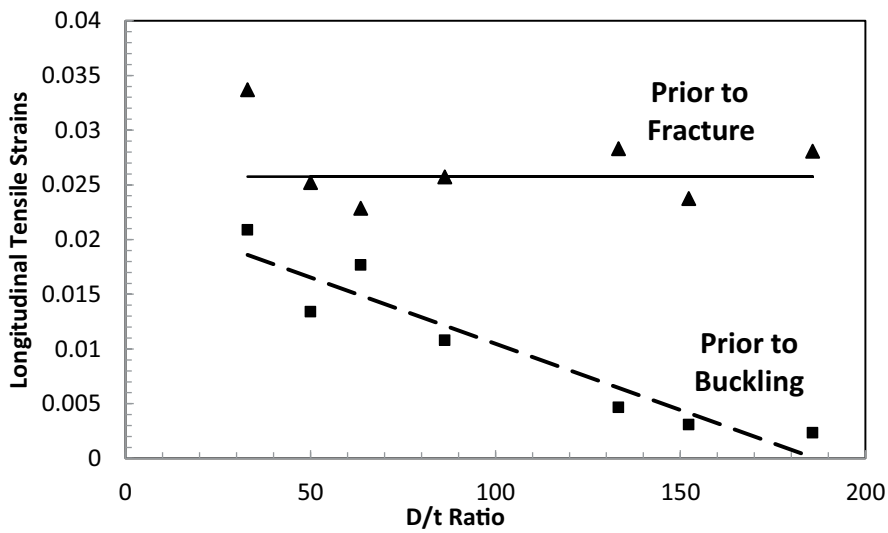
Seismic Behavior of Reinforced-Concrete-Filled Steel Tubes

by Dr. Mervyn J. Kowalsky, North Carolina State University

Research on concrete-filled steel tubes is vast and has been conducted for many decades. Over the last 10 years, a research program undertaken at North Carolina State University (NC State) has aimed to investigate the seismic performance of reinforced-concrete-filled steel tubes (RCFSTs). The system involves a reinforcing bar cage that is placed inside a steel shell, which is then filled with concrete. In the research done at NC State, both spirally welded and straight-seam pipe have been studied, with diameter-to-wall thickness ratios (*D/t*) varying from 32 to 192. As noted in the companion article in this issue of *ASPIRE*® on the Brotherhood Bridge near Juneau, Alaska, the use of RCFSTs is common in Alaska, as well as other states. In the case of Alaska, the RCFST system is preferred for bridge design for several reasons:

- The steel tubes serve as the formwork for the piles/columns.
- The steel tubes can be used in driven or drilled foundation applications.
- The steel tubes provide high levels of confinement, thus resulting in large deformation capacity, which is important for good seismic performance.
- The system has less environmental impact than conventional substructures requiring larger cofferdams.
- Construction is simplified because the steel tubes serve as piles below the soil surface and as columns above the ground level.

When subjected to lateral demands, these RCFST systems may develop plastic hinges at the connection to the cap beam as well as below grade at the point of maximum moment. Therefore, research has been directed toward developing guidelines for the designs of these regions. Among the parameters that have been investigated are below-grade plastic hinge length, energy dissipation capacity, and quantitative measures of key limit states such as buckling and rupture of the tube wall.



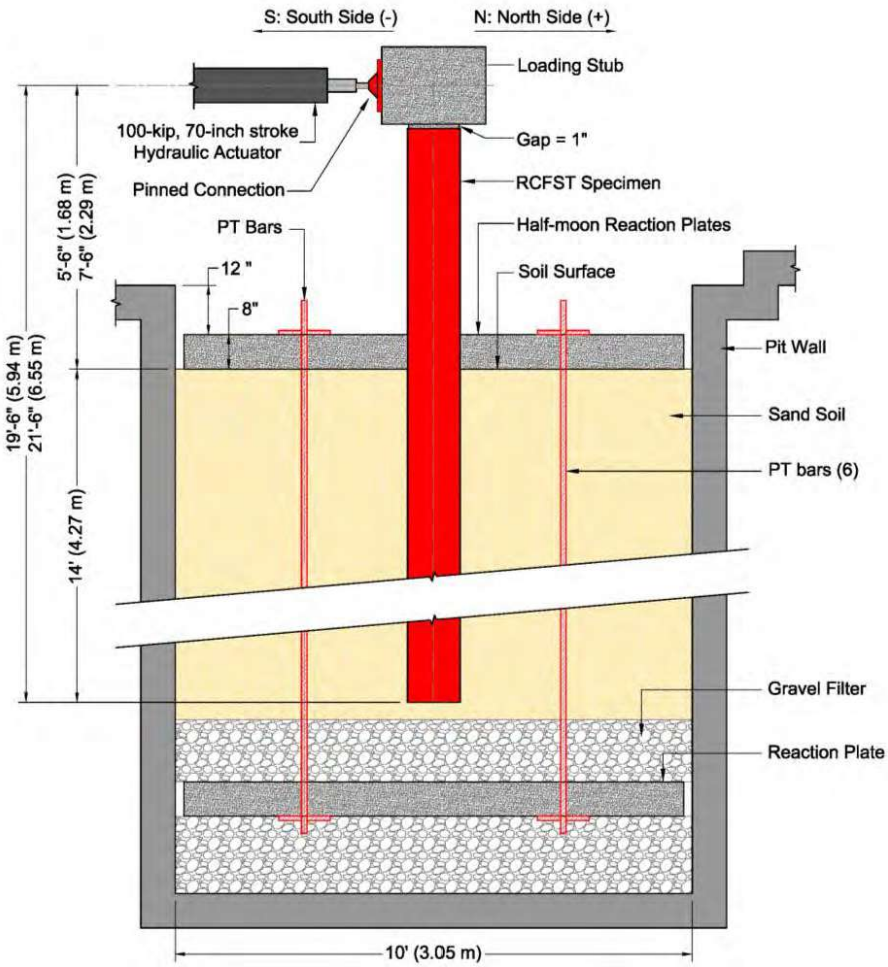
Strain limit states as a function of tube diameter/wall thickness ratio (*D/t*). Figure: Nicole Brown.

Initial tests at NC State on RCFSTs were conducted where the test units were subjected to reversed cyclic four-point bending.[1–4] The span between actuators was adjustable to account for different constant-moment regions, which in turn model different soil stiffnesses. In total, 30 RCFST specimens were tested: 22 were 2 ft in diameter and 33 ft long, and eight were 10¾ in. in diameter and 14 ft long. Four of the tests were conducted at –40°C (–40°F) to investigate the RCFST behavior at wintertime temperatures typical of Alaska.

The testing program showed that tube wall buckling is strongly dependent on the *D/t*, while fracture occurs at an approximately constant strain, irrespective of *D/t*. In all cases, tube wall local buckling was followed by tube fracture. The energy dissipation capacity of these systems was also shown to be large.

The next series of tests focused on RCFSTs in soil using the soil-structure interaction facility at NC State.[5,6] This setup allowed the soil stiffness to be modified by prestressing the sand between two concrete plates. Each specimen was subjected to reversed cyclic lateral loads during testing. Damage levels were similar to what was observed for the initial four-point bending tests.

In addition to tests on the below-ground hinge, the RCFST column-to-cap connection, which was developed at the University of California, San Diego, in the 1990s, was subjected to simulated seismic forces at NC State at temperatures as low as –40°C (–40°F). The connection, which leaves a gap between the RCFST shell and the bottom of the cap, was shown to perform well at low temperatures. Recommendations on material strength and plastic hinge lengths as a function of temperatures typically encountered in Alaska were also proposed.[7] Combined with the extensive research on the RCFST itself, the system has been deployed in numerous Alaska bridges over the last 20 years. Approximately 20 multispan bridges with



Test setup to model reinforced-concrete-filled steel tubes in soil. Figure: Diego Aguirre.

RCFST piers were located within areas having a peak ground acceleration greater than 0.25*g* during the 7.0-magnitude earthquake event on November 30, 2018. Even though many of these bridges were located in liquefiable soils, none experienced any damage or signs of significant movement.

In summary, based on research over a 10-year period that indicated good performance of these systems, design guidelines have been developed for RCFST seismic design. Recommendations have included strain limits, plastic hinge lengths, and analysis methods.

Deformation of reinforced-concrete-filled steel tube during test in soil. Photo: Diego Aguirre.

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Sarah Mildred Long Bridge Tower Foundation Construction

by Joseph Orlando, Cianbro Corporation

The new Sarah Mildred Long Bridge, which spans the Piscataqua River from Portsmouth, N.H., to Kittery, Me., opened on March 30, 2018. The structure has two-level precast concrete segmental box-girder approach spans with a 300-ft-long lift span. Additionally, the 200-ft-tall towers of the lift span are vertically post-tensioned segmental towers, a first in the United States. The foundations for the lift span towers are 125 ft long, 65 ft wide, and 15 ft thick. The design of the foundation includes a precast concrete tub used as a cofferdam and form. An allowance of 18 in. for the tub-wall thickness and 3 ft for the bottom thickness was included in the design. Eight 10-ft-diameter drilled shafts socketed 35 ft into competent rock support each foundation. The top of the foundation is at elevation 8 ft, about 2 ft above the highest astronomical tide expected during construction. Each of the structural footings contains 3200 yd^3 of cast-in-place concrete and 500,000 lb of reinforcement along with 14 post-tensioning ducts for the segmental towers.

The project was jointly sponsored by the Maine and New Hampshire departments of transportation. The project delivery method was the construction manager/general contractor method, and the Cianbro Corporation of Pittsfield, Me., was the general contractor (for a profile of Cianbro, see the Spring 2017 issue of *ASPIRE*®). FIGG Bridge Engineers designed the approach spans, and Hardesty & Hanover designed the lift span and towers as a joint venture.

Tower Foundation Design Challenges

The Piscataqua River is a deep and fast-flowing tidal river, one of the fastest in the United States. This factor and others presented many challenges to the construction of the tower foundations. The following were among the primary considerations that influenced the ultimate method of foundation construction:

- The tidal current could exceed 4 knots.
- The maximum crane capacity at working radius using a Manitowoc MLC300 Lattice Boom Crawler, which was the largest crane the contractor had in its fleet, on a 72 ft by 200 ft deck barge was 180 tons.
- The foundation had to be submerged 13 ft at the highest astronomical tide.
- The water was 65 ft deep, with little overburden on the river bottom.
- There were significant construction and environmental loads, such as water pressure due to the tidal current alternating in direction.
- Worker and diver safety had to be ensured.

Sarah Mildred Long Bridge approaches, lift span, towers, and tower footings. Photo: Cianbro/Richard Hopley.

Others have approached the construction of these types of foundations using the following methods:

- Casting a complete tub and setting it on drilled shafts using a large capacity crane
- Casting and launching a complete tub, floating it over the drilled shafts, and then submerging it into place
- Suspending a precast concrete segmental soffit above high water, building a traditional cofferdam on the soffit, and using synchronous jacks to lower the system into place

Cianbro analyzed these options and ruled them out because it did not have sufficient crane capacity to lift 1100 tons, and a dry dock or launching system was not available to fabricate and launch such a large floating tub. Additionally, the Piscataqua River's extreme current and a slack tide of only 30 minutes made the prospect of floating and accurately positioning a large tub over the drilled shafts too risky. Similarly, slowly jacking a tub down into the current was not ideal.

Cianbro Solution

A precast, sand-lightweight concrete, segmental post-tensioned tub addressed the design challenges. The MLC300 crane could quickly erect the tub. The combination of strength and density of the sand-lightweight concrete was critical to the success of the design because loads were close to crane capacity. Additionally, each piece could be set quickly to the design elevation and secured during slack tide. The tubs were free draining, which gave workers two hours to work inside the tubs at low water. Workers grouted and bolted the tubs from the top. There was no need for divers to go beneath the tubs, thereby improving safety.

Each precast concrete segmental tub, one for each tower, was composed of nine segments: five drilled-shaft units and four fill units. The drilled-shaft units used strongbacks with hangers to support the segments from the top of the drilled shafts. For erection, the fill

Installation of the drilled-shaft units. Photo: Cianbro.

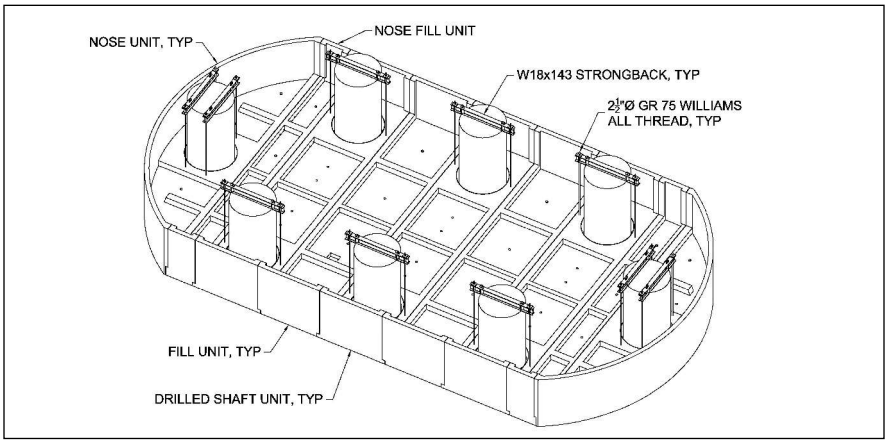
Fill unit installation. Photo: Cianbro.

Nose unit installation over a drilled shaft. Photo: Cianbro.

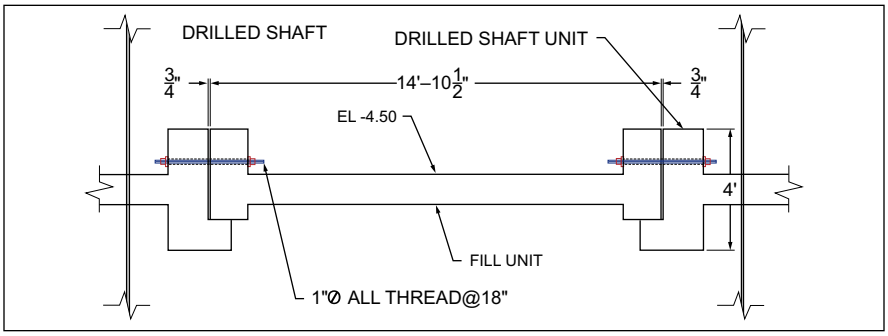
units were placed between the drilled-shaft units and were supported by an integral ledge cast into the drilled-shaft units. The drilled-shaft segments could be precisely located in elevation and plan by using threaded hangers and horizontal alignment fixtures cast into the concrete around the penetrations for the drilled shafts. The horizontal alignment fixtures also stabilized the segments from the pressure of the water current. A ¾ in. space between segments provided erection tolerance and space to grout and seal between segments. Grade 75, 1-in.-diameter sleeved all-thread bolts were spaced at 18 in. along the entire length of the joint between segments. Each all-thread bolt was post-tensioned to 30 kip after the joints were grouted, allowing the entire load of the fill unit (weight of first lift of concrete plus dead load of the fill piece) to be transferred to the drilled-shaft unit through shear friction. An extensive finite element model was developed of the tub using RISA-3D software to determine the structural behavior of the tub.

The success of the system depended on the connection of the tub to the drilled shafts. The connection was required to transfer large uplift forces from the dewatered tub as well as the loads imposed from concrete placement. The design shear load transferred to each drilled shaft was 850 kip. A grouted shear key design, typically used in the offshore oil industry, provided the necessary capacity. The connection was designed using grouted pile-to-structure connection criteria from the American Petroleum Institute's *Recommended Practice 2A-WSD.*[1] Installing forms for grouting before a segment was set allowed the divers to grout the connection from the top. High-strength (8 ksi), structural underwater concrete, with washout additives, provided the required strength in a timely manner in the cold river water.

The dewatered tub with the drilled shafts visible. The work platform has been attached to the top of the tub. Photo: Cianbro.



The precast concrete segmental tub. The use of sand-lightweight concrete for the segments allowed the tub to be installed using readily available equipment. Figure: Cianbro.



Typical segment joint detail. The fill unit of the tub was placed on an integral ledge of the drilled-shaft unit during erection. Later, the joint was grouted and post-tensioned. Figure: Cianbro.

Conclusion

The precast concrete, post-tensioned segmental tub provided a cofferdam/form system that was constructed using readily available erection equipment. Tub erection, grouting, and dewatering took only 30 days, which helped meet an aggressive project schedule. Exposure to the harsh environmental conditions was minimized, thereby reducing risk. The use of sand-lightweight concrete was also critical to the success of the design.

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Joseph Orlando is senior structural engineer with Cianbro Corporation in Portland, Me.

EDITOR'S NOTE

More information on the Sarah Mildred Long Bridge is contained in a Project article in this issue of ASPIRE *on page 24.*

Implementation of an Electrically Isolated Tendon Post-Tensioning System in the United States

by Dr. Clay Naito, Lehigh University, and Dr. Christina Cercone, Manhattan College

Post-tensioned concrete bridges represent an important component of the current U.S. bridge inventory. These systems, whether cast-in-place or precast concrete construction, can be used to create long-span continuous bridge superstructures with long service lives. However, ensuring the quality of the post-tensioning duct system and grout material used to encapsulate the strands has been a concern. The post-tensioning ducts are conventionally located within the web of the concrete girders, and critical sections—such as splices and anchorages—are heavily reinforced to provide adequate transfer of forces into the members. These practical requirements of post-tensioning (PT) make visual and nondestructive inspection techniques difficult to implement during construction and throughout the life of the bridge. Quality control methods are well established for PT installation and grouting, but traditional inspection methods are not easy to implement during regular service-life inspections of internal tendons.

To address this issue, electrically isolated tendon (EIT) systems have been developed. These systems, which can be easily integrated into new construction, provide full electrical isolation of the steel PT tendons from the surrounding concrete of the bridge structure. With isolation, the PT steel cannot form a corrosion cell with the exterior reinforcement, and corrosion of the tendon is nearly eliminated because the amount of oxygen within the duct is low.

System Components

The EIT system is composed of an enhanced tendon and anchorage system. The anchorage includes an isolation plate, a plastic trumpet, and proper encapsulation of the anchorage head. The tendon system uses a plastic duct, with plastic duct couplers and plastic half shells to protect the integrity of the corrosion protection system along the length of the tendon. The plastic half shells are used between the plastic duct and the transverse reinforcement to provide additional protection to the duct during member fabrication and tensioning.

Several companies commercially produce the required components of the EIT system for the European market, making this an off-the-shelf technology. For the past 20 years, EIT systems have been used in various post-tensioned concrete bridges in Europe, including the Piacenza Viaduct and Marchiazza Viaduct in Italy and the P.S. du Milieu and the Wiesebrücke Basel in Switzerland.

With the EIT system, the encapsulation system is assessed from time of construction through the service life of the bridge by using commercially available inductance/capacitance/resistance (LCR) meters to measure the AC resistance, capacitance, and loss factor between the isolated PT strands and the reinforcing steel located in the surrounding bridge structure. These readings can be compared to limiting acceptance criteria values[1] to provide owners with an assessment of the quality of the encapsulation and any unexpected corrosion initiation. These measurements are often taken at key stages of construction, such as during tensioning or tendon installation, before and after grout installation, during deck placement, and during service. The EIT system has the potential to provide long-term condition assessment of the tendon system over the life of the bridge.

First U.S. Demonstration Project

The many benefits of the EIT system and its successful use in Europe have prompted research and development efforts geared toward implementation in the United States, including a series of

The main unit of the Coplay-Northampton Bridge consists of a 540-ft-long, three-span section over the Lehigh River. The superstructure consists of five prestressed concrete PA bulb tees, which were spliced and post-tensioned. All Photos: Dr. Clay Naito.

Post-tensioning anchorages in precast concrete beams during spiral reinforcement installation, with an electrically isolated tendon trumpet (top) and a traditional trumpet (bottom).

Post-tensioning splice prior to placement of the closure joint. No difference can be seen between the electrically isolated tendon at the top and the three conventional tendons below.

demonstration bridge projects sponsored by the Federal Highway Administration (FHWA). The first demonstration project integrates the EIT system into the Lehigh County–owned Coplay-Northampton Bridge between Coplay and Northampton boroughs in Pennsylvania. This construction project is currently underway. Deck placement is expected in spring 2019, and the bridge is scheduled to open in fall 2019.

The primary portion of the Coplay-Northampton Bridge consists of a three-span, 540-ft-long unit over the Lehigh River. The superstructure consists of prestressed concrete PA bulb tees, which were spliced and post-tensioned in the field. The beams were precast at Northeast Prestressed Products in Cressona, Pa. The bridge was designed by AECOM and constructed by Trumbull Corporation, with project management provided by Pennoni. Each beam has four PT ducts. As a demonstration, only the top PT duct in each beam line has the EIT system, which was provided by DYWIDAG-Systems International (see the Focus article in this issue of *ASPIRE*® on page 6).

As mentioned previously, the EIT system requires the use of a plastic duct with plastic duct couplers and plastic half shells. In the demonstration project, the addition of the required isolation plate and plastic trumpet in the EIT anchorage was not a significant change to a traditional PT system. All the PT ducts, including the single EIT duct, were spliced in the field using plastic couplers combined with heat-shrink tubing. To ensure electrical continuity of the beam reinforcement located outside of the PT ducts, two uncoated pretensioning strands were left protruding from each end of each beam. Electrical continuity along the entire span is important because the measurements are taken at one end of the bridge. Failure to maintain electrical continuity of some spans would limit the effectiveness of the EIT system. The continuity strands were tied to the uncoated steel shear reinforcement within each beam during precasting and were tied together at each splice after erection. The two electrical continuity strands were located at the bottom of the beams.

The EIT system was monitored during selected stages of the construction project (following tensioning of the tendons, prior to and following grouting, and 28 days after grouting), and monitoring will continue to evaluate the long-term performance of the PT system. Measurements are taken by connecting the LCR meter to one of the exposed continuity strands located at the bottom of the flange and to the PT strands of the EIT system, using an exposed wire connection. These measurements are compared to the threshold performance limits and indicate the level of electrical isolation of the tendons.

Initial measurements indicate that the EIT system meets the current isolation requirements needed to achieve the highest level of corrosion protection, PL3, as defined in *PTI/ASBI M50.3-12: Guide Specification for Grouted Post-Tensioning*[2] and additional recommendations developed by Angst and Büchler for FHWA.[1]

According to discussions with the precast concrete producer, project manager, contractor, and Pennsylvania Department of Transportation, the use of the EIT system required minimal added construction effort, has had nominal impact on project costs, and appears to be a viable approach for providing additional quality assurance of the PT system.

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Measurement of the electrically isolated tendon (EIT) system after grouting. The inductance/capacitance/resistance meter is connected to the exposed steel located at the bottom flange and to the post-tensioning strands using the wire connection to the top EIT anchorage.

Sweep in Precast, Prestressed Concrete Bridge Girders

by Dr. Bruce W. Russell, Oklahoma State University

Sweep is defined in the *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products* (PCI MNL-116)[1] as "a variation in horizontal alignment from a straight line parallel to centerline of member (horizontal bowing)." A generalized representation for sweep is shown in **Fig. 1**.

Decades of industry experience inform us that very few girders are produced with noticeable or out-of-tolerance sweep. However, when sweep is large enough to be noticed—or when the amount of sweep exceeds tolerance—producers, contractors, and owners may be understandably concerned about how sweep could occur in production, and what corrective actions can, or should be taken. Regardless the amount of sweep, the truth is this: most prestressed concrete bridge girders, once they are in place with composite concrete decks cast upon them, will function effectively for the design life of the bridge without concern for limits on strength, durability, or serviceability.

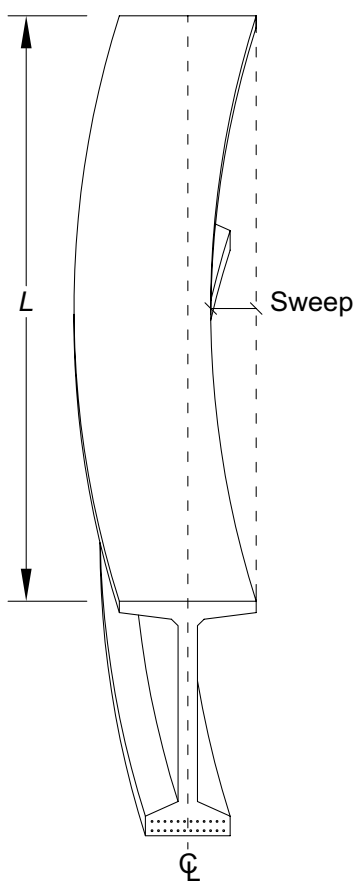

Figure 1. A generalized representation for sweep in a girder. Figure: Dr. Bruce Russell.

Prestressed concrete bridge girders that are built in accordance with our design standards, and are made from conforming materials (concrete and steel) within a geometry described by the contract documents, will support the intended design loads for millions of cycles, in all kinds of climates, for the design service life of the structure. This article focuses on the sweep of bridge girders that may occur during initial fabrication, lifting, or storage in the yard. One probable cause is explored with a hypothetical example and corrective actions that can be taken while the girder is still in the precast producer's yard. Subsequent articles will cover transportation, erection, and construction issues and discuss whether permanent sweep in girders should cause concern for long-term stresses, strength, or performance.

Measurements and Tolerances

The initial measurement of sweep is made after the side forms are removed and prestressing force is transferred, or after the girder is removed from the forms and is placed on dunnage in the precast producer's yard. When measuring sweep, it is important that the girder is level, the sun is not shining on one side, and the actual flange widths have been verified at the ends and middle. Commonly, sweep is measured at the midspan of the girder. Admittedly, accurate measurement may be difficult for long girders. However, a measurement to ¼ in. precision should be sufficient (⅛ in. would be preferable). The measurement(s) should be recorded even if zero.

Alternatively, it may be easier to measure the sweep at approximately 20 ft increments. In this case, measurements should be made accurate to ±1/16 in. A sweep measurement made at the center of a 20.0-ft length, or the average of several such measurements, can be converted to an overall sweep by the following:

$$f = f_{20}\left(\frac{L}{20.0\text{ ft}}\right)^2$$

where

f = sweep projected over the entire girder length L

f_{20} = sweep measured over a 20.0 ft length

Tolerances for precast, prestressed concrete members have been established to allow for production and erection variances, as well as the interface of the member with the overall construction of the structure. Tolerances are to be used for guidelines and not as reasons for rejection. If the girder can be brought within tolerance in an acceptable manner, or if exceeding the tolerance does not affect the integrity or performance of the girder, the girder should be accepted. Later articles will show that girders with sweep in excess of tolerances can function without compromising the completed bridge system.

The tolerance for sweep for bridge girders given in MNL 116-99[1] is ⅛ in. per 10 ft. For example, a bridge girder with a length of 130.5 ft would have a sweep tolerance of 1.63 in. The calculation for sweep tolerance follows:

$$\text{Sweep Tolerance} = 130.5\text{ ft}\left(\frac{\frac{1}{8}\text{ in.}}{10\text{ ft}}\right)$$

$$= 1.63\text{ in.}$$

What Causes Sweep and What Should Be Done?

Sweep can be caused by a number of factors, including eccentric prestressing force, the misalignment of forms, anomalies in locations or positions of lifting devices, differences in material properties within the girder, variation in concrete maturity at early ages, exposure to sunlight, out-of-level supports, or differing curing conditions.

Based on previous experience, each producer should be aware of what amount of sweep is normal for a particular girder type. When the initial measured sweep is larger than this "experience" value, or if measured sweep exceeds tolerance or is projected to exceed tolerance, then an investigation should be undertaken immediately by the precaster and sweep should be monitored on a regular basis while the girder is stored. The investigation might uncover a problem that can be corrected before subsequent girders are produced or, if mitigating or corrective action on the existing girder is required, lead to a preferred course of action. As time goes by, measured sweep may worsen because of creep and concrete maturity, with fewer opportunities for corrective action. An investigation by the producer should include checking or reviewing the following:

- Form alignment.
- Cross-section dimensions in several locations along the length of the forms.
- Strand locations—ensure templates are centered and that holes have not elongated.
- Uniform pretensioning of strands and uniform application of prestressing forces.
- Plumbness of girder and level bearing surfaces in storage—a girder that is stored out of plumb can develop sweep.
- Exposure to sun—thermal differences between the sides of the girder in storage can cause sweep.
- Curing temperatures—if heat is applied, ensure uniform application of elevated temperatures.
- Quality control records—search for differences in concrete batches.
- The design—strand pattern and items such as blockouts in a flange or localized changes in the web.

In many cases, the cause of sweep is not easily identified. It could involve a combination of two or more of the listed items.

Table 1. Girder Design Details for Example

Bridge:	SH 99 over Deep Fork River, Lincoln Co., Okla.	
Design Engineer:	Bruce W. Russell	
Span:	130.5	ft
Type of Girder:	PCI-AASHTO BT-72	
Spacing, s =	7.17	ft
A =	767	in.^{2}
h =	72.0	in.
c_b =	36.60	in.
I_{xx} =	545,113	in.^{4}
I_{yy} =	37,543	in.^{4}
w_b =	0.799	kip/ft
f'_c =	10.0	ksi
f'_{ci} =	7.0	ksi
E_{ci} =	4821	ksi
Width of bottom flange	26	in.
Width of top flange	42	in.
Strand type:	0.6-in.-diameter Gr. 270 ASTM A416, low relaxation	
N =	38	strands
e =	29.02	in.

Calculating Sweep for Eccentrically Located Strands

For the purposes of this discussion, one possible cause of sweep is explored. The example is based on a PCI-AASHTO bulb-tee BT-72 girder[2] designed for State Highway 99 (SH 99) over the Deep Fork River in Lincoln County, Okla. The original designs were performed by the author. Spans for this bridge were 130 ft 6 in., measured from center-to-center of bearings. Girders were spaced at 7 ft 2 in. Each BT-72 girder was made with thirty-eight 0.6-in.-diameter Grade 270 prestressing strands conforming to ASTM A416. Design details are listed in **Table 1**. Please note that the example used for this article is hypothetical and created only for purposes of illustration. During fabrication of the actual girders for the SH 99 Bridge, there were no reports of sweep in the bridge girders.

For purposes of illustration, let us compute the sweep as if all strands in the girder were located ¼ in. off center, or with a lateral eccentricity e_x of ¼ in., which is the tolerance in PCI MNL-116 for strand placement. The following calculations compute the amount of sweep f that results from the minor-axis bending moment induced by the ¼ in. eccentricity of strands assuming thirty-eight 0.6-in.-diameter strands with an effective prestressing force of 1512 kip after elastic shortening loss:

$$F_{pe}e_x = (1512\text{ kip})(0.25\text{ in.}) = 378\text{ kip}-\text{in.}$$

$$\phi_y = \frac{F_{pe}e_x}{E_{ci}I_{yy}} = \frac{378\text{ kip}-\text{in.}}{(4821\text{ ksi})(37{,}543\text{ in.}^4)} = 2.088\times10^{-6}\text{ rad/in.}$$

$$f = \frac{\phi_y L^2}{8} = \frac{(2.088\times10^{-6}\text{ rad/in.})(130.5\text{ ft})^2\left(144\dfrac{\text{in.}^2}{\text{ft}^2}\right)}{8} = 0.640\text{ in.}$$

From these calculations, one can see that a strand pattern that is placed eccentrically by ¼ in. can produce an estimated sweep of 0.64 in. immediately after transfer. This sweep is less than the sweep tolerance of 1.63 in. for a 130.5 ft girder, which was calculated earlier in this article.

The sweep created by the eccentric prestressing force creates a theoretically uniform minor-axis curvature ϕ_y over the length of the bridge girder. As can be seen in the calculations, sweep is proportional to the square of the girder length. Therefore, in the author's opinion, for longer girders, it may be more appropriate to set a larger tolerance for sweep than the ⅛ in. in 10 ft provided in MNL 116-99. For example, the same strand eccentricity that creates a ¾ in. sweep for a 100 ft girder (which exceeds the tolerance of 1.25 in.) also produces a 3.0 in. sweep for a 200 ft girder, which exceeds the tolerance of 2.50 in. The point is that longer girders are more susceptible to noticeable or out-of-tolerance sweep simply because they are longer, and the effect that may have caused sweep in the first place is squared in proportion to length.

Stresses Caused by Sweep

Generally, if sweep exists, it is most often small, and its effects on stresses can be ignored. Also when considering the magnitude of the stresses, readers should be aware that the stresses computed in this manner are temporary stresses and,

accordingly, the importance of temporary stresses should not be overemphasized in judging the performance of precast, prestressed concrete bridge girders.

The earlier computations of the sweep caused by an eccentric prestressing force resulted in a lateral curvature of 2.088 × 10^{-6} rad/in. If this curvature is extended to the outer edges of the bottom fiber, strains at the extreme fibers would be:

$$\varepsilon = \pm\ 2.088 \times 10^{-6}\,\frac{\text{rad}}{\text{in.}} \times 13\ \text{in}$$
$$= \pm\ 27 \times 10^{-6}\,\frac{\text{in.}}{\text{in.}}$$

This strain would produce 131 psi of additional compressive stress on one side of the bottom flange and reduces the compressive stress on the opposite side by 131 psi. At the top flange, an additional tensile stress of 211 psi will theoretically be produced on one side.

Now, some engineers may be tempted to include additional sources of tension in their calculations. However, one must bear in mind that these calculations are approximations and represent mere estimates for the stresses and strains that may actually occur within bridge girders. Accordingly, I argue that additional calculations for stress and strain based on possible sweep deformations are relatively unimportant and should not be used to make additional requirements for computation, for load conditions, or for handling. Instead, this calculation serves as a reminder *to place reinforcement where tension exists* in order to control cracking and provide better distribution of stresses and strains throughout the cross section. It is my firm opinion that if the engineer has provided sufficient reinforcement in the tensile zone of a precast, prestressed concrete bridge girder, and if that design is in accordance with current American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[3] for temporary stresses, the additional tensile stresses created by sweep in girders can be ignored.

Table 2a. Summary of assumptions.

$RH =$	70	%
$V/S =$	3.01	in.
$k_f =$	0.625	
$k_s =$	1.06	≥ 1.0
$k_{hc} =$	1.00	

Creep Considerations

Because of concrete's relative immaturity at early ages, we know that the concrete will experience the largest creep strains at early ages. Creep also can have significant effects on prestressed concrete because the prestressing forces remain active over the life of the bridge. Similarly, the girder sweep measured initially is affected by the creep properties of concrete, and so discussion of creep within this article is important.

In the example provided, the estimated sweep of 0.64 in. that would be caused by a ¼ in. eccentricity of prestressing force is less than the 1.63 in. tolerance. Those involved must draw from their own experience to judge whether sweep is "normal," but this author will argue that the estimated sweep is not "excessive." Nonetheless, even "normal" amounts of sweep may merit consideration for a proactive approach when considering the effects of creep. Immediately after transfer, and upon initial storage at the precast producer's yard, the concrete is young and the internal crystalline structures that provide concrete strength are maturing rapidly. As the concrete creeps with time, the sweep is also expected to increase.

Table 2a provides a summary of assumptions and **Table 2b** are the calculations performed to estimate the increase in sweep caused by creep at various ages. Data in the table show the additional sweep that can occur within the first 28 and 90 days for the BT-72 girder being used as an example in this article. Considering only the "within tolerance" sweep of 0.64 in. caused by offset prestressing strands, and using provisions in the current AASHTO LRFD specifications[3] for creep calculations, we can see that the sweep deformation nearly doubles in 90 days, to 1.24 in. At this level of sweep, the bridge girder is approaching the tolerance limit of ⅛ in. per 10 ft. of length (that is, 1.63 in., as computed previously). Additionally, various other potential effects may combine to increase sweep, and many of these effects may be unknown. Therefore, we should conclude that it is possible that tolerance for sweep may be reached and exceeded, although, as mentioned earlier, the project from which the example girder is taken had no reported issues with sweep, which is typical of the great majority of bridge projects.

Recommendations to Mitigate Sweep

Sweep in a girder is not typically a large concern, but it can affect handling, transportation, erection, and stability issues even when the measured sweep is within tolerance.[4] For many reasons, the engineer, owner, or precaster may want to reduce the sweep of the girder to ease any handling, lifting, transportation, or erection issues. When a decrease in the amount of sweep in a girder is desired, the early age of the concrete and its self-weight enable corrective actions. The recommendations for corrective action fall into two categories:

1. Straighten the girder with bracing and forces, or
2. Store the girder on slightly sloped supports and allow its self-weight to straighten the girder.

To straighten the girder using direct application of force, the girder must be braced at the ends to provide a reaction to the straightening force applied at midspan or intermediate points. A structural frame will likely be required to provide both straightening forces and reactions, or the girder with sweep can be braced against other girders. It is also important that the straightening force be applied through the centroid of the cross section; if it is not, the application of force will cause torsional deformations. In the example provided for the 130.5-ft-long BT-72 girder, the lateral midspan force required to straighten the girder would be about 1450 lb. The sweep must continue to be monitored, and it is likely that the force will have to be maintained over the full time of storage to straighten the girder prior to transportation.

Alternatively, the girder can be straightened by storing the bridge girder on a slight

Table 2b. Effects of Creep at 28 and 90 Days Using AASHTO LRFD Article 5.4.2.3.2.

Age (days)	Creep coefficients		Sweep f from ±¼ in. eccentric prestressing force and creep (in.)
	Time Development Factor k_{td}	Creep Coefficient $\Psi(t,\ t_i)$	
1	–	–	0.64
28	0.467	0.587	1.02
90	0.738	0.929	1.23

angle. In this method, the girder's self-weight will work over time to straighten the beam. At the very least, storing the girder with a slight tilt will prevent creep effects from worsening the initial sweep.

The method for straightening a girder by storing on canted dunnage is described as follows. (Calculations shown are for the BT-72 beam with an initial sweep of 0.64 in.)

1. Calculate the required slope of the support to produce a tilt to counteract the measured sweep. For the BT-72 girder, the uniform load (amount of lateral self-weight) w_y to cause a midspan deflection equal to the 0.64 in. sweep is:

$$w_y = \frac{384 E_{ci} I_{yy}}{5L^4} f(\text{sweep})$$
$$= \frac{384(4821\text{ ksi})(37{,}543\text{ in.}^4)}{5(130.5\text{ ft})^4\left(1728\frac{\text{in.}^3}{\text{ft}^3}\right)}(0.64\text{ in.})$$
$$= 0.0178\text{ kip/ft}$$

The slope of the supports required to straighten the 130.5-ft-long BT-72 beam during storage can then be computed as the ratio of w_y to the weight of the girder w_b:

$$\theta = \frac{w_y}{w_b} = \frac{0.0178\text{ kip/ft}}{0.799\text{ kip/ft}} = 0.0223\text{ rad}$$

Using this procedure, the supports for this bridge girder in storage should be sloped at θ, or approximately ¼ in. per 12 in., or approximately ½ in. for every 24 in. (about the width of the bottom flange). My recommendation is to measure the angle against the web of the girder using a 2- or 4-ft level.

2. Monitor the sweep regularly during storage to ensure that sweep is sufficiently reduced before transportation.

Conclusion

In most bridge girders, sweep, if it exists, is likely to remain small and unnoticed. However, as longer and lighter bridge girders are fabricated, sweep may become more noticeable. This article provides an example for one cause of sweep in a girder, discusses increase in sweep that can occur at early ages when the girder is most susceptible to time-dependent changes, and provides an outline for corrective action, if that is desired.

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Managing Structures Affected by Delayed Ettringite Formation and Alkali-Silica Reaction

by Dr. David Rothstein, DRP, a Twining Company

Delayed ettringite formation (DEF) and alkali-silica reaction (ASR) are durability mechanisms that can lead to cracking that reduces the service life of concrete structures. An overview of these mechanisms was provided in the Summer 2018 issue of *ASPIRE*®. This article will discuss how to manage structures with ASR and DEF, with an emphasis on ASR because it is far more common than DEF.

Investigating Damage

The first step in managing ASR or DEF is to identify whether a structure has been damaged. Investigations are done visually on site, using petrographic techniques, and with other laboratory methods.

Field Investigations

The starting point for all investigations is on-site visual inspection of the structure and documentation of deterioration such as cracks, deformations, displacements, joint deterioration, pop-outs, efflorescence, and discoloration of surfaces.[1] Although map or pattern cracking is a classic indicator of internal expansion, the presence of reinforcement and prestressing (pretensioning or post-tensioning) forces can influence cracking patterns, making them more linear and aligned. The concrete should be sampled for petrographic examination, and the samples should represent a suite of different conditions ranging from cracked to intact. Other samples may be collected to test compressive strength, splitting tensile strength, and other engineering properties that may be of interest for evaluating performance.

Petrographic Investigations

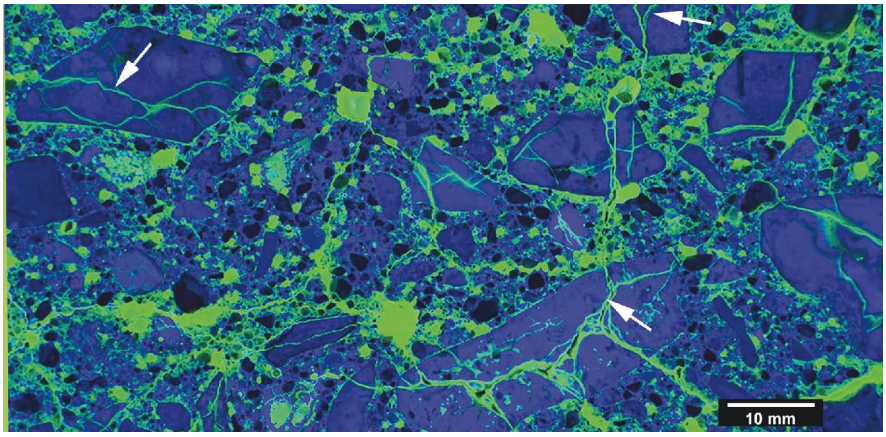
When damage is observed in the field, petrographic examination is used to determine whether there is evidence of internal expansion. An experienced petrographer can determine if internal expansion from ASR is occurring and whether it is linked to macroscopic cracking. In many structures, cracks observed in the field are from early-age mechanisms such as drying shrinkage rather than ASR. Early-age cracks tend to cut orthogonally to exposed surfaces and to cut around aggregate particles. They also lack secondary deposits. In contrast, cracks caused by DEF or ASR contain significant deposits of ettringite or ASR gel, respectively. When internal expansion progresses to the point of visible cracking, a petrographer will see microcracks filled with gel or ettringite that coalesce and feed into macroscopic cracks.

Petrographers can categorize the severity of ASR qualitatively or quantitatively. In qualitative rankings, severe ASR is indicated by clear evidence of macroscopic cracking; moderate ASR by the presence of associated microcracks; and minor ASR by the presence of gel without cracks or microcracks. Quantitative methods include the Damage Rating Index (DRI), which assigns numerical "scores" to indicate the degree of damage.[2] There is no standardized threshold score to indicate a particular level of damage, but the DRI can nevertheless be a useful tool to compare different parts of a structure or assess the same parts of a structure at different times.

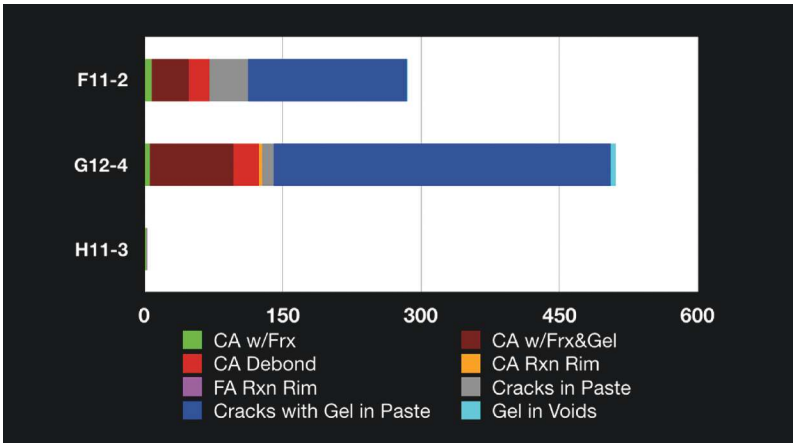
Other Laboratory Investigations

Determining the chemical composition of the gel may help investigators understand whether the ASR is at an early or late stage.[3] Other laboratory tests can help determine the likelihood that ASR will cause continued expansion. For example, residual expansion tests can be used to monitor cores for length change

Photograph of map cracking on concrete pavement affected by alkali-silica reaction. All Photos and Figures: Dr. David Rothstein.



Ultraviolet light photograph of polished slab from a concrete core affected by alkali-silica reaction. The core was impregnated with a fluorescent epoxy. The yellow-green areas are filled with the epoxy. The white arrows indicate examples of cracks.



Summary diagram of Damage Rating Index (DRI) results, showing how the method documents variations in the amount of damage. The bar chart summarizes the amount of distress associated with different indicators of distress associated with alkali-silica reaction (ASR). An entire bar represents a total DRI score, with higher numbers indicating more damage. The different colors within a bar indicate the different forms of damage that contribute to the total score. F11-2 and G12-4 show extensive damage, primarily associated with cracks with gel in the paste, and H11-3 shows essentially no damage. Abbreviations are as follows: CA, coarse aggregate; FA, fine aggregate; Frx, fracture; Rxn, reaction.

under controlled conditions.[4] Measuring the water-soluble alkali content of the concrete can show whether alkali in the system is sufficient to sustain ASR. Remember that in cases where concrete is exposed to deicing salts, the alkali content of the concrete can increase over time because of the ingress of the salts.[5]

Managing the Structure

Once ASR is identified as a concern, the appropriate approach to manage it depends on the severity of the ASR and the potential for future expansion. The first option is to simply monitor the structure by conducting field inspections at regular intervals. Crack index maps and strain gauges can be used to determine if cracks are growing. Also, additional cores can be extracted to be examined petrographically (DRI works well for this) and compared to cores evaluated previously.

If remediation is needed, several approaches are available. The most viable strategy for slowing or stopping the progression of ASR is to cut off the water that feeds it. Studies have shown that an internal relative humidity (RH) of 80% to 85% is essential to sustain ASR in concrete.[6] Often, the first step in keeping concrete elements below this RH threshold involves repairing or modifying drainage systems to redirect water flows away from and out of structures. Waterproofing membranes, filling cracks and construction joints with appropriate materials, and applying sealers or other surface treatments may be adequate. Coatings that permit water vapor to escape are preferred because they will facilitate the drying of the structure. In some cases, sealers that do not allow the structure to breathe can make ASR worse because they trap moisture entering from untreated surfaces. Topical applications of lithium-based compounds have been used with mixed results to arrest the progression of ASR. A major concern with such materials is the depth of penetration, which may be negligible in some cases.

Other measures to prolong the serviceability of structures affected by ASR may include structural repairs such as external post-tensioning, use of carbon- or glass-fiber wraps, or other methods to provide physical restraint. In cases where there is significant damage, cutting expansion joints or removing areas around damaged joints may reduce stress. However, such measures are only temporary, and they do not mitigate future expansion.

Conclusion

The diagnosis of ASR or DEF in a structure does not necessarily mean that immediate removal, replacement, or even repair is required. It is essential to conduct careful field and petrographic investigations of the structure to obtain an understanding of the severity of the problem. In most cases, it is prudent to implement field-based programs to monitor and evaluate the behavior of the structure over time. Preventing the ingress of water remains the most effective method to mitigate ASR in structures. In cases where damage is significant, structural repairs may be necessary. In all cases, experienced engineers, petrographers, and contractors should be engaged to ensure that the proper strategies and methods for this work are deployed.

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Advancing Bridge Repair and Preservation Using Ultra-High-Performance Concrete

by Dr. Zachary B. Haber and Dr. Benjamin A. Graybeal, Federal Highway Administration

Ultra-high-performance concrete (UHPC) is an advanced, fiber-reinforced, cementitious composite material that has gained significant attention from bridge owners and designers because of its excellent mechanical and durability properties. The rapid growth in UHPC's popularity in the U.S. bridge market has been spurred in part by its ability to provide a much-needed, robust solution for closure pours in prefabricated bridge element (PBE) connections.[1] This application has also been a common entry point for owners interested in the opportunities UHPC provides for bridge design and construction. To date, in the United States, nearly 200 bridges in 27 states and the District of Columbia have been constructed using UHPC materials, and 93% of these projects used UHPC for PBE connections.

Moving Beyond PBE Connections

As UHPC-class materials become more prevalent, new and innovative applications for these materials in bridge design and construction will continue to emerge. One emerging application for UHPC in the U.S. highway bridge sector is for bridge repair, retrofit, and rehabilitation. Since 2016, at least 10 U.S. bridges have been repaired or rehabilitated using UHPC-class materials, and, for the most part, each application has had unique aspects. (Refer to the *FHWA Interactive UHPC Bridge Map* for additional details.[2])

Four of these projects (in Colorado, California, Florida, and Rhode Island) marked the first use of UHPC on state-owned highway bridges. Each of these projects used relatively small volumes of UHPC material and were viewed as relatively low risk by the bridge owners. As such, repair applications are becoming an alternative entry point for bridge owners looking to generate institutional knowledge and familiarize local

Ultra-high-performance concrete repair of connections between precast, prestressed concrete voided-slab beams in Florida. Photo: Florida Department of Transportation.

The first installation in the United States of an ultra-high-performance concrete bridge deck overlay was in Iowa. Photo: Federal Highway Administration.

Ultra-high-performance concrete was used for the repair of a midspan spliced U-girder closure pour. Photo: SEMA Construction Inc.

contractors with UHPC.

The following innovative UHPC-based repair solutions have been implemented, conceptualized, or laboratory tested:

- UHPC overlays for bridge deck rehabilitation
- Repair of existing connections between PBEs
- Structural patching of reinforced concrete bridge decks, prestressed concrete girders, and bearing pedestals
- Expansion joint repair with UHPC headers or replacement using UHPC link slabs
- Seismic retrofit of deficient bridge column-footing lap splices
- Structural strengthening of deteriorated girder ends

The first three of these applications are summarized here.

UHPC Overlays for Bridge Decks

In one of most promising UHPC-based bridge repair applications, UHPC is used as a thin, bonded overlay for bridge deck rehabilitation. As an overlay, UHPC can provide structural strengthening, protection from chloride and water ingress, and good resistance to mechanical abrasion, which reduces the tendency for rutting. This deck rehabilitation can be achieved with a 1- to 3-in.-thick layer of UHPC, which minimizes material volume and additional dead load. This repair method, which was pioneered in Europe, and has been deployed on more than 20 European bridges, is particularly competitive when an owner is looking to extend the life of a structure that is otherwise difficult to replace, such as a life-line structure, a historically significant bridge, or a structure in a challenging geographic or urban area. To date, there have been three UHPC overlays installed in the United States. The first of these installations was on a reinforced concrete slab bridge in an agricultural region in Iowa.[3,4] UHPC overlays are an active research topic at the Federal Highway Administration (FHWA) Turner-Fairbank Highway Research Center (TFHRC), and additional information can be found in references 3 through 6.

Repair of Existing Connections

UHPC-class materials are known to be excellent candidates for closure pours for PBE connections because these materials can provide durable connections and very compact reinforcement details.[1] The Florida Department of Transportation (FDOT) leveraged these characteristics to repair an existing bridge (constructed in 1978) with precast, prestressed concrete voided-slab beams. The units were transversely post-tensioned and employed partial-depth, conventionally grouted shear keys. After years of service, reflective cracking became apparent in the bridge's asphalt overlay, and there was evidence of differential deflection between adjacent slab units. Instead of replacing the superstructure, FDOT decided to remove the existing connection regions using hydrodemolition, install additional reinforcement, and replace the removed concrete with UHPC. Laboratory testing of similar connections using UHPC between adjacent box beams has demonstrated that UHPC has the ability to create full composite action and robust performance.[6]

Structural Patching and Localized Repairs

UHPC-class materials have been used for localized repair of expansion joint headers, structural patching of bridge decks, and structural patching of prestressed concrete bridge girders. An example of structural patching was performed on a recently constructed, multispan interstate flyover. This bridge had a spliced U-girder superstructure. During construction, poorly consolidated concrete was identified within a closure pour at a midspan girder-to-girder splice location. The poorly consolidated concrete was removed and replaced with UHPC. UHPC was selected as the repair material because it bonds very well with steel reinforcement and existing concrete, and because it is highly flowable and self-consolidating. The region to be repaired was highly congested; thus, the repair material needed to have excellent flow properties.

Moving Forward

The advancement of bridge repair and preservation practices using innovative materials such as UHPC is a current area of focus for research within the Structural Concrete Research Group at FHWA TFHRC. We are striving to develop new solutions to solve some of the continual challenges related to bridge preservation. We hope this article spurs innovative thinking as it relates to concrete bridge repair, and we look forward to assisting owners with the identification, design, and installation of innovative solutions.

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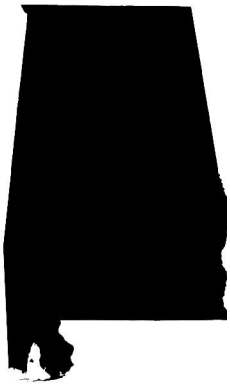

Alabama

by William (Tim) Colquett, Alabama Department of Transportation

Precast concrete caps were set onto precast concrete columns for the Interstate 59/20 Central Business District Bridge project. Photo: ALDOT.

A segment for the Interstate 59/20 Central Business District Bridge project is removed from the casting bed. Photo: ALDOT.

Micropiles were used in some locations of the Interstate 59/20 Central Business District Bridge project because of the varying soil conditions in the area. This was the first use of micropiles in the state. Photo: ALDOT.

Concrete has been a mainstay of Alabama's highway bridge infrastructure for many years. There are concrete-superstructure standard drawings that date to the 1920s, and some of the bridges built from those drawings are still in service. With advances in technologies, materials, and construction techniques, bridges that would have traditionally been constructed of steel are now designed using concrete.

Alabama's bridge inventory consists of 15,980 bridge structures, including 5757 bridges and culverts that are owned and maintained by the State. Approximately 69% of those structures (excluding culverts) have concrete superstructures. Concrete construction has proved to be efficient, economical, and aesthetically pleasing.

Concrete construction has proved to be efficient, economical, and aesthetically pleasing.

These benefits are nowhere more apparent than with the current Interstate 59/20 Central Business District bridge replacement project currently underway in downtown Birmingham, Ala. The largest contract ever let by Alabama Department of Transportation (ALDOT), the $475 million project replaces 6600 ft of steel girders and several fracture-critical steel bent caps for structures that run through the downtown area. The circa-1971 bridges were designed for average daily traffic (ADT) of 80,000; however, ADT is projected to reach 225,000 in 2035. The old bridges lack shoulders and have obsolete geometry, with some entrance and exit ramps placed on the left.

The new bridges, which have wider cross sections to accommodate additional lanes and full shoulders, eliminate some of the entrance and exit ramps and utilize segmental concrete box girders. Constructability, schedule, and aesthetics were among the reasons that this approach was chosen. The segmental girders provide longer span lengths, eliminating substructure and providing more space beneath, which city officials intend to use for public events. Also, noise levels, both above and below, will be reduced because concrete provides much-improved noise abatement under the bridges and because longer span lengths and continuity will eliminate joints, reducing noise from wheel strikes.

The segments are being cast off site and stockpiled until a section of the bridge is closed, at which time the contractor will set the segments. Prior to closure, while the existing bridges are still in place, foundation work is being completed using low-overhead equipment that can operate in the confined space.

Because of the area's karst geology, subsurface conditions vary within small distances; therefore, foundations include micropiles, driven-steel piling, and drilled shafts. Micropiles are being used for the first time in a widespread application on a bridge project in Alabama. Precast concrete columns and caps are further expediting construction, with grouted splice sleeves being used for connections between precast concrete elements and the footings.

The contractor has 14 months (from January 2019 to spring 2020) to complete and open the new bridge system. It will be erected using the span-by-span method with ground-based shoring. An aesthetics package includes Wi-Fi-controlled multicolored lighting installed beneath the bridge that can be programmed for events at the bridge. To further expedite construction, electrical infrastructure was preinstalled in the segments.

Use of Accelerated Bridge Construction Techniques

Accelerated bridge construction (ABC) techniques have been in use in Alabama for many years. In some ways, the state was advancing ABC methods before ABC became an industry buzzword. For example,

Construction of one of the new twin bridges on the Ross Clark Circle (U.S. Route 231) over Beaver Creek culvert in Dothan, Ala. The rollers and steel tracks shown in the photo on the right were used with hydraulic jacks to slide the new bridge into place. Photos: ALDOT.

short-span precast concrete bridges and substructures have been a common feature of Alabama's off-system roadways for years.

In some ways, the state was advancing ABC methods before ABC became an industry buzzword.

Perhaps the most prominent use of ABC on state highways came in 2002 and 2004, when two different structures in the Interstate 59/20 interchange were damaged beyond repair after tractor trailers wrecked under the bridges and their fuel loads were ignited. These bridges, located in what is sometimes called the "Malfunction Junction" due to the heavy traffic and diverging-diamond geometry, had to be replaced as rapidly as possible to minimize travel disruptions on an already difficult route. Steel spans were replaced with prestressed concrete bulb-tee girders on an accelerated schedule. One span was replaced in 36 days, and the other was back in service in 38 days. Prefabrication of span components while the site was prepared was a key to that speed. ALDOT has incorporated similar techniques into other projects to help reduce user costs and keep traffic moving smoothly.

Recently, another ABC technique was successfully deployed for the bridge slide in the $2.43 million culvert replacement on Ross Clark Circle (U.S. Route 231) in Dothan, Ala. (see the article on this project in the Fall 2016 issue of *ASPIRE*®). The objective was to widen the structure from two to three lanes in each direction as quickly as possible to minimize traffic disruptions in an area with high ADT. In lieu of constructing a new culvert, ALDOT's solution was to retain the existing culvert, partially remove the fill from the culvert's top, and span over it with parallel 120-ft-long bulb-tee girder spans, 54 ft and 63 ft wide, each supported by steel piles and cast-in-place concrete abutments.

The contractor constructed the abutments beneath live traffic by using steel trench boxes with removable lids that accommodated traffic during peak hours and then could be removed in off-peak times to allow piles to be driven. Once the piles were driven, the lids were put back in place while workers beneath completed construction of the abutment caps.

The superstructure was constructed on roller bearings, so it could be rolled into place during specific windows of closure times. When the slide was ready, traffic was shut down, the trench boxes were removed, and the spans were rolled into place. The first bridge was rolled into place and completed in three days, while the second was completed in two.

Successfully completing this project required ALDOT to take a proactive approach with the contractor. This close relationship allowed dialogue and suggestions to flow easily, facilitating the quick processing of issues and creating a pattern that can be used on future projects where such ABC techniques may be appropriate.

Although these projects demonstrate ALDOT's willingness to use new techniques when needed, Alabama's bridge projects typically use conventional construction methods. Most bridges are built with traditional concrete designs, typically using AASHTO-type and bulb-tee girders. These designs usually provide the best economy, aesthetics, durability, and speed of construction for the state.

Emerging Delivery and Design Methods

Design options for bridges may expand as ALDOT ventures into new delivery methods. For example, for the Interstate 10 (I-10) Mobile River Bridge, a planned six-lane structure that will be the state's second cable-stayed bridge, three prequalified teams now are putting together design, cost, and schedule proposals for the project, which is being financed through a public-private partnership. This approach offers the only way that the state can afford the project, which will cost between $800 million and $1 billion.

Design options for bridges may expand as ALDOT ventures into new delivery methods.

The I-10 Mobile River Bridge will also be the state's first design-build project, which became possible when legislation was passed to allow the use of that delivery method on projects costing more than $100 million. Clearly, many new concepts will emerge from this process, and ALDOT is watching closely to determine what benefits these approaches can provide for future projects.

Bridge Aesthetics

New design ideas will also aid in creating more aesthetically pleasing bridges, which has become a higher priority. The range of aesthetic options available with concrete has

broadened, with new decorative methods being produced and material costs declining. Decorative railings, abutments, and even lighting systems, such as the one for the Birmingham Central Business District project, are becoming more prominent.

A recent project emphasizing aesthetics is the replacement of the Moores Mill Road Bridge in Auburn, which spans the busy Interstate 85 corridor between Atlanta, Ga., and Montgomery, Ala. Faced with growing ADT, Auburn wanted a wider bridge, with a dedicated sidewalk and aesthetic treatments that suited the structure's high-visibility location. The new design features two 120-ft-long spans using 54-in.-deep modified prestressed concrete bulb-tee girders, a 74-ft-wide deck between gutters, and a 9-ft-wide sidewalk. Precast concrete pier columns expedited the work in the I-85 median. Formliners were used to simulate brick on the barrier rails, portions of the columns, abutment wings, and toe wall. They were stained to closely resemble the brick color used at nearby Auburn University. Decorative lighting and railing were installed to complement the simulated brick. The $3.79 million bridge opened in the spring of 2018.

Even along Alabama's coast, where durability is the primary consideration, aesthetic design is a growing priority. The recent bridge replacement on State Route 182 in Gulf Shores, Ala., shows how durability and aesthetic details can be combined. The new bridge is a 191-ft-long, three-span continuous structure that uses precast concrete arch girders with just mild reinforcement (no prestressing or post-tensioning). The bents are composed of spun precast concrete cylinder piles and precast concrete caps, which were chosen for their durability. The abutments feature square prestressed concrete piles with cast-in-place concrete caps and deck.

Formliners were used to simulate brick on the Moores Mill Road Bridge over Interstate 85 in Auburn, Ala. The "brick" was then stained to resemble the brick used for buildings at nearby Auburn University. Photo: ALDOT.

Decorative reliefs were cast into the outside fascia of the girders, and a varied height, custom aluminum "wave" shape was designed for the railing to complement the beach environment. The $1.71 million bridge opened in 2015.

Looking to the Future

More construction options will be added to the toolbox as the state adjusts to advanced techniques. For example, ALDOT will soon let its first project using Northeast Extreme Tee (NEXT) beams. They are a good solution for specific applications, and ALDOT expects to use them on other future projects as well. Florida I-beams (FIBs) are another option, with good potential for specific applications. ALDOT is closely watching the use of 100-ft-long FIB36 beams for a privately funded bridge to be built in Huntsville, Ala.

Also, owing to the varied and complex soil conditions in Alabama, micropiles are expected to be used more often in the future. They offer a good alternative when soil conditions create challenges and the Birmingham Central Business District project has demonstrated their advantages.

Alabama's future may include more design-build and public-private partnership projects, as those delivery methods potentially provide benefits and more innovative designs in some situations. Traditional approaches often offer the best solution, but the state is open to new options that will help create economical, efficient, and aesthetically pleasing designs.

William (Tim) Colquett is state bridge engineer with the Alabama Department of Transportation in Montgomery.

The top priority for the new State Route 182 Bridge over the channel at Little Lagoon in Gulf Shores, Ala., was durability, but designers also added aesthetic touches to play off the structure's oceanside setting. Photo: Scott Bridge Company.

To reflect the maritime location, decorative elements including dolphin shapes, were cast into the fascia and a custom aluminum "wave" shape for the railing was added to the State Route 182 Bridge over the channel at Little Lagoon in Gulf Shores, Ala. Photo: ALDOT.

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AASHTO LRFD Bridge Design Specifications: Service IV Load Combination

by Dr. Oguzhan Bayrak, University of Texas at Austin

Our *ASPIRE*® team recently received a question about when and why we use the provision for prestressed (pretensioned and post-tensioned concrete) substructure solutions with the load combination for the Service IV limit state, which is one of many limit states in the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*.[1] This article aims to shed light on that topic.

Before we address the Service IV load combination specifically, let us first consider the historical development of bridge design specifications. From the issuance of the first bridge design specifications in the late 1920s until early 1970s, bridges were designed by using allowable stress design (ASD) principles. In a nutshell, ASD is based on the premise that stresses created by loads acting on bridges and their components create "working" or allowable stress conditions that must be kept below a certain fraction of actual material strength.

After nearly half a century of successful use, ASD was set aside, and a new direction was taken based on the more rigorous notion of the factor of safety against failure of the section. This was intended to give a more uniform factor of safety against failure, which was not possible with ASD.

For load factor design (LFD), the strength (resistance) of the section was compared to service loads that were individually increased by load factors that were greater than 1.0. Prior to the comparison of strength to demand of the applied loading, the resistance was reduced by a resistance factor that was typically less than 1.0. The magnitudes of the load and resistance factors were based on judgment, considering uncertainty and variability of the loads and section properties.

The transition to load and resistance factor design (LRFD) for concrete simply involved the formal calibration of the load and resistance factors used in LFD design by using statistical methods and field data on loads, material properties, and other aspects. Subsequently, variability in material resistances, the accuracy with which structural capacities can be estimated, and the consequences of different modes of failure were all brought into the LRFD design process.

The first edition of the AASHTO LRFD specifications was published in 1994. Since 1994, the AASHTO LRFD specifications have been revised or updated as our knowledge about the performance of concrete bridges has evolved. Our decisions while developing new techniques, revising older methods, and calibrating load and resistance factors have been informed by data generated in new experimental programs, the advent of increased computing power, the associated development of advanced analysis techniques, and the field performance of concrete bridges.

In the eighth edition of the AASHTO LRFD specifications, which was published in 2017, loads and load combinations are covered in Section 3.[1] As stated in that section, we have five strength limit states, two extreme event limit states, and four service limit states, in addition to the two fatigue and fracture limit states. The Service IV limit state relates to controlling tension in prestressed concrete columns, with the ultimate goal of preventing or minimizing the chances of cracking in these substructure elements to improve durability and maintain a higher flexural stiffness.

Load combinations and load factors from Table 3.4.1-1 in the AASHTO LRFD specifications. Figure: *AASHTO LRFD Bridge Design Specifications*, 8th edition.

Load Combination Limit State	*DC DD DW EH EV ES EL PS CR SH*	*LL IM CE BR PL LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	Use One of These at a Time				
										EQ	*BL*	*IC*	*CT*	*CV*
Strength I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength III	γ_p	—	1.00	1.00	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—	—
Strength V	γ_p	1.35	1.00	1.00	1.00	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Extreme Event I	1.00	γ_{EQ}	1.00	—	—	1.00	—	—	—	1.00	—	—	—	—
Extreme Event II	1.00	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00
Service I	1.00	1.00	1.00	1.00	1.00	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—	—
Service III	1.00	γ_{LL}	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service IV	1.00	—	1.00	1.00	—	1.00	1.00/1.20	—	1.00	—	—	—	—	—
Fatigue I—*LL, IM & CE* only	—	1.75	—	—	—	—	—	—	—	—	—	—	—	—
Fatigue II—*LL, IM & CE* only	—	0.80	—	—	—	—	—	—	—	—	—	—	—	—

The use of segmentally constructed columns and precast, prestressed concrete columns is currently proliferating as project stakeholders aim to accelerate bridge construction and leverage the greater quality control associated with plant-fabricated concrete elements. Also, post-tensioned cantilever and straddle caps are growing in use as we renew the interstate system. These substructure elements should be designed for many load combinations, including the Service IV load combination. In some cases, the load combination of Service IV may control the design of these substructure elements. The commentary for Article 3.4.1 of the AASHTO LRFD specifications explains the evolution of the Service IV load combinations.

To better understand the historical evolution of the Service IV load combinations, we must appreciate the fact that previous editions of the AASHTO LRFD specifications were based on fastest-mile wind speed. In other words, the wind effects were averaged over different lengths of time over a distance. This approach has been revised and modernized over the years. The current AASHTO LRFD specifications are based on 3-second gust wind speed, which means that the wind speed is averaged over 3 seconds.

The use of a large number of loads and load combinations is commonly facilitated by design software. Using computers for the analysis and design unquestionably gives us a chance to improve the precision and levels of reliability in our structural designs to a degree we could not have dreamed of during the major infrastructure expansion in the United States after the conclusion of World War II. Establishing uniform levels of safety has many design and cost benefits and reduces environmental impact by removing unnecessary conservatism (that is, overdesign) where appropriate. With these important goals stated, we must remain alert in our designs to ensure that all important or controlling load applications are considered. Limiting tensile stresses in prestressed concrete (both cast-in-place and precast) substructure elements (piles, caps, footings, and columns that are pretensioned or post-tensioned) while in a fabrication plant and during transportation, erection, construction, and service is an important goal of design as we aspire to see our concrete bridges serve our communities for at least 100 years.

Service IV load combination was used to design the precast concrete segmental substructures for the U.S. Route 183 elevated viaduct in Austin, Tex. Photo: Dr. Oguzhan Bayrak.

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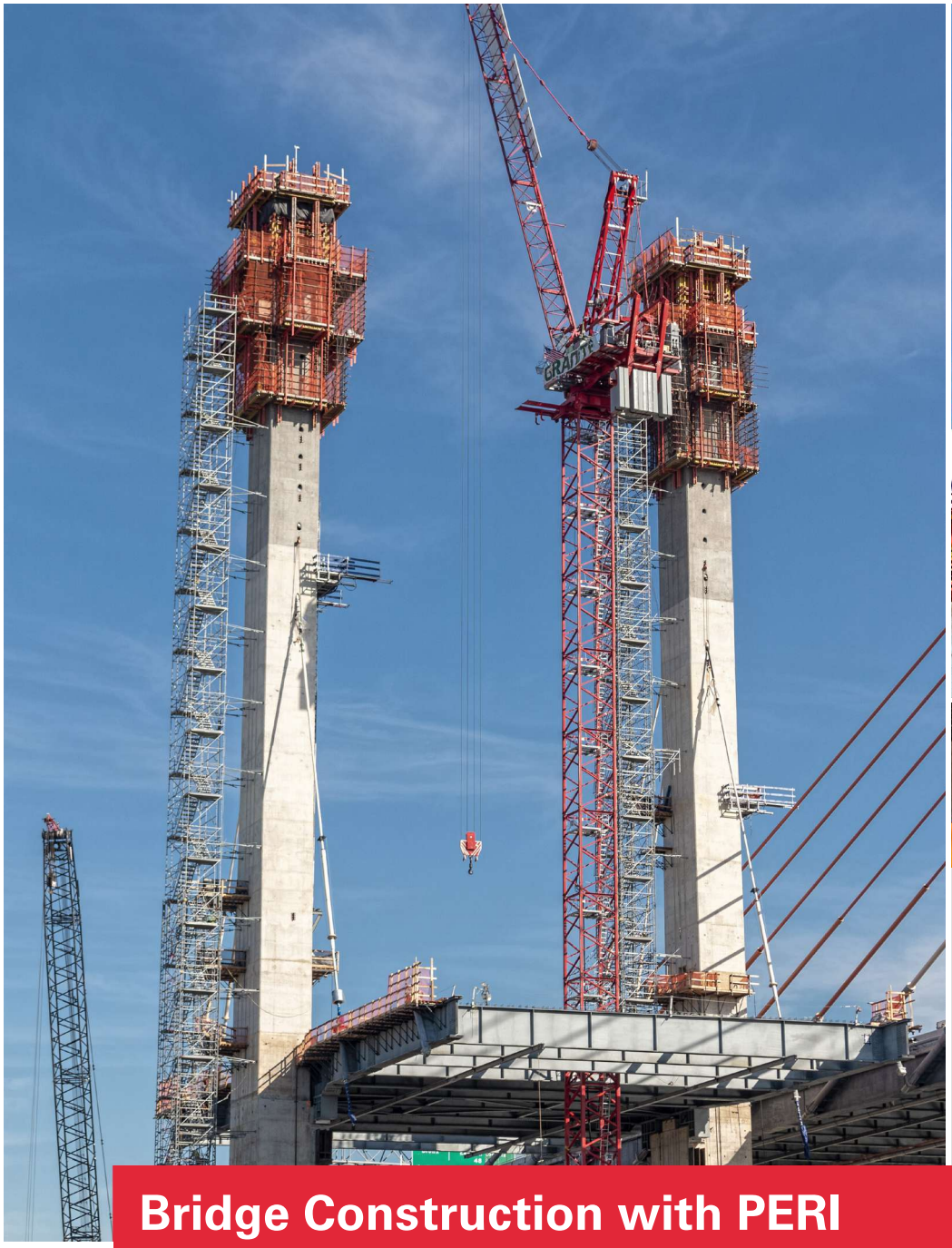

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