ASPIRE®
THE CONCRETE BRIDGE MAGAZINE
SUMMER 2020
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PHX Sky Train Stage 2 at Phoenix Sky Harbor International Airport

Phoenix, Arizona

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Glenwood Springs, Colorado

COPLAY-NORTHAMPTON BRIDGE
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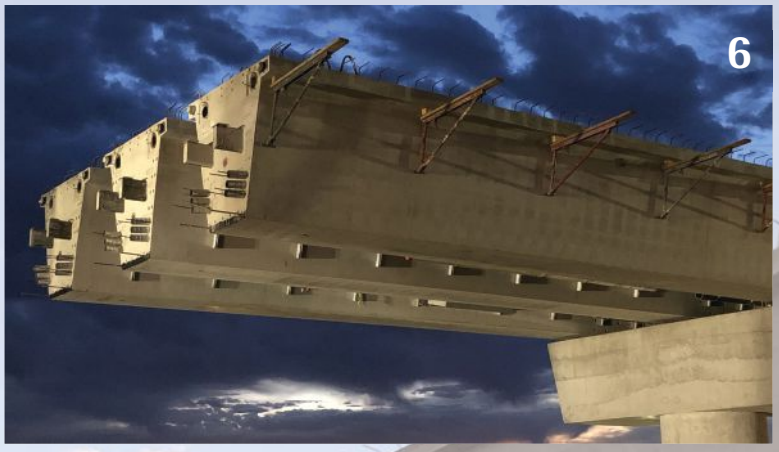

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EDITORIAL

Be Careful What You Ask

by William Nickas, Precast/Prestressed Concrete Institute, and Gregg Freeby, American Segmental Bridge Institute

Most recent readers of the editorials in this magazine clearly grasp the notion that the Florida International University (FIU) pedestrian bridge collapse struck us deeply. Like you, we believe our business and this profession of ours demand our best.

When we first read the comments of National Transportation Board (NTSB) vice chairman Bruce Landsberg, they stung (the full text of his letter was reprinted in the Spring 2020 issue of *ASPIRE*®). How is it that Landsberg's words touched such an emotional chord? He is not an engineer; his entire professional career centers on private and commercial aviation—how can he possibly understand? We had several long and thoughtful discussions about this. Seeking insight into Landsberg's perspective, we made a call to my former Citadel roommate, a recently retired military officer who spent most of his 34 years of service in the cockpit of military aircraft. In addition to being an aviator, he is also a civil engineer. Surely, he would be able to help us gain greater insight into the vice chairman's statement.

Be careful what you ask. On this call, the two of us were educated on mission analysis, course of action development, associated risk, aircrew selection, premission briefings, rehearsals, route adjustments, risk assessment, final briefing, and launch authority. The military's process for each mission is thorough and vetted, double-checked with a keen eye on risk and safety. Every step and sequence is studied. After the mission is flown, a complete after-action report and review occurs. These sessions are inclusive, detailed reviews in which all crew members' candid assessments and perspectives are sought. The result is a holistic and honest critique, outlining things to maintain and sustain as well as aspects that must be addressed and fixed. It's during these reviews that lessons learned are identified and shared among the community as a way to learn, grow, and develop.

As my former roommate put it, "We're carrying our most precious resource, our sons and daughters. Our job is to get them to and from their destinations safely, without taking unnecessary risk. I was tough on discipline. Undisciplined crews and units take unnecessary risk. Our business is inherently dangerous and risky; there's no reason to take unnecessary risk by being undisciplined. We're trying to mitigate risk, not increase it. Fly the aircraft to standard, and adhere to the checklist and procedures." His comments on discipline led us to pause and think.

Further in the conversation, my old friend expressed his view that the vice chairman's commentary was direct, accurate, and clearly stated. "It resonates because it's an accurate assessment. The reason leaders are in the positions they're in is to lead and make the hard call when required. You only get one chance to land the aircraft safely."

"Wow. Thanks for sharing." At this point, there was an almost uncomfortable silence on the call.

Bridge engineers and aviation experts are not so different after all. Isn't our job as engineers to provide safe infrastructure that gets our most precious resource, our sons and daughters, to and from their destinations safely, without taking unnecessary risk?

That's why when our community suffers a catastrophic event, its effects ripple through our profession. As professionals, we seek to understand the cause(s) or reasons for such tragedy. Our aim is not to criticize but to understand the issues and circumstances. Then, we adjust our processes to remove this threat from future builds. It's why we continue to revise codes, enhance designs, and prove concepts through calculation, modeling, and simulation. Our pledge continues to be about lives, safety, and the health and welfare of the general public, and all our efforts must be based on sound, disciplined engineering judgment.

In this issue, an article by Dr. William Lawson gives his views on engineering judgment as it relates to the FIU tragedy. Also, Dr. Donald Meinheit shares an implementation plan for the design requirements, testing, installation, and inspection of concrete anchors, which are all now coordinated to "get it right" as a result of the "Big Dig" anchor failures.

Also in this issue, you will find an article by Dr. Oguzhan Bayrak about structural behavior and redundancy, further issues identified in the NTSB report. In the article, he notes the cracking levels observed in various lab specimens are more than experimental results. They should be alerting us to what cracks observed in the field can tell us about in-service behavior.

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Erection of the PHX Sky Train over the existing Terminal 2 using three drop-in girder segments.. Photo: Modjeski and Masters Inc.

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If you have a suggestion for a project or topic to be considered for *ASPIRE*, please send an email to info@aspirebridge.org

Unlike the engineering profession, the aviation community actively shares lessons learned. They don't do this as a "gotcha," but as a way to move their profession forward and avoid repeating mistakes. They're about making the system better and developing a TTP (tactic, technique, or procedure) that enhances mission success and survivability.

Is it time to develop a national repository of engineering lessons learned? This could be a place where structural engineers can seek historical perspectives when a situation develops that might require a tough call.

Thank you, Mr. Shutt

For nearly three decades, Craig Shutt has been presenting clear and accurate information through PCI's various publications. Leading up to the institute's 50th anniversary, in 2004, Craig authored "PCI 50 Years: Visions Taking Shape." When PCI launched *ASPIRE* magazine, in 2007, Craig was on board as its first managing editor.

In recent years, Craig has written many of the project articles that have appeared in *ASPIRE* and *ASCENT*. Armed with a wealth of construction industry history and an impressive number of relationships within the precast concrete industry, Craig brought an understanding and craftsmanship to his writing that has been very much appreciated.

We thank Craig for all his contributions to the advancement of the industry and we wish him well as he embarks upon retirement.

ERRATA

Discussion of Sweep in Precast, Prestressed Concrete Bridge Girders–Part II

In "Sweep in Precast, Prestressed Concrete Bridge Girders—Part II" by Dr. Bruce W. Russell published in the Fall 2019 issue of *ASPIRE*® magazine (pp. 38–43), an error was found in the calculations for the girder braced using a king post. The author has revised the text on page 43 beginning with the second line as follows (deletions are ~~struck through~~; additions are shown in red):

> "... post. The king-post arrangement with four ~~fully-tensioned~~ strands tensioned to 30 ksi deflects the center of the girder ~~0.44~~ 0.89 in., which is a sizable amount of the original sweep used in this example. Moreover, largely because of the straightening effects, the factor of safety against cracking FS_θ is increased from 1.60 (without the king post) to ~~2.62~~ 2.35 with ~~8 fully-tensioned~~ eight 0.6-in.-diameter strands tensioned to 30 ksi. These computations ..."

The remainder of the paragraph is unchanged.

Values in **Table 5** are revised to reflect the recalculated stiffness of the braced-girder system and the revised tension in the strands of the king post. The corrected version of Table 5 is shown below.

The revised values do not alter the author's conclusion.

The article posted on the *ASPIRE* website will be revised to reflect these changes.

Table 5. Increasing Lateral Stiffness and Straightening the Girder with a One-Sided, External King Post at a 36 in. Standoff

No. of External Strands	A_{sx}, in.^{2}	Lateral Stiffness Factor	z_0, in.	FS	F_{px}, kip	Δ_x, in.	θ_i, rad	FS_θ
0	0	1.00	15.43	2.18	0.00	0.00	0.0323	1.60
1	0.217	1.01	15.26	2.21	0.60	0.22	0.0281	1.69
2	0.434	1.02	15.09	2.23	1.20	0.44	0.0239	1.79
4	0.868	1.05	14.76	2.28	2.39	0.89	0.0155	1.99
6	1.302	1.07	14.45	2.33	3.59	1.33	0.0070	2.19
8	1.736	1.10	14.16	2.38	4.79	1.77	-0.0014	2.35

CONTRIBUTING AUTHORS

Gregg A. Freeby is the executive director of the American Segmental Bridge Institute and president of the Infrastructure Advancement Institute.

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

William N. Nickas is PCI's managing director of transportation systems. He is a professional engineer with 36 years of experience dedicated to the design and construction of transportation-related facilities.

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.

Dr. John Schemmel is the Bruce and Gloria Ingram Endowed Chair in Engineering in the Ingram School of Engineering at Texas State University in San Marcos.

CONCRETE CALENDAR 2020–2021

The events and dates listed were accurate at the time of publication but may change as local guidelines for gatherings continue to evolve.

July 15, 2020
2021 Call for Papers
PCI Convention and National Bridge Conference
Submission deadline

July 31, 2020
2020 CRSI HONORS Program
Submission deadline

August 2–6, 2020
AASHTO Committee on Materials and Pavements Annual Meeting
Virtual subcommittee meetings

August 12–14, 2020
PTI Level 1 & 2 Multistrand and Grouted PT Specialist Workshop
Dallas, Tex.

August 15, 2020
PTI Level 1 & 2 Multistrand and Grouted PT Inspector Workshop
Dallas, Tex.

August 18, 2020
2021 PCI Design Awards
Submission deadline

August 26–28, 2020
PTI Level 1 & 2 Multistrand and Grouted PT Specialist Workshop
Gainesville, Fla.

August 29, 2020
PTI Level 1 & 2 Multistrand and Grouted PT Inspector Workshop
Gainesville, Fla.

August 31, 2020
2020 PCA Concrete Bridge Awards Competition
Submission deadline

September 8–26, 2020
PCI Committee Days
Virtual committee and council meetings

September 13–16, 2020
AREMA Annual Conference & Expo
Hilton Anatole
Dallas, Tex.

September 27–30, 2020
PTI 2020 Convention & Expo
Hilton Miami Downtown
Miami, Fla.

October 25–29, 2020
ACI Fall 2020 Convention
Raleigh Convention Center &
Raleigh Marriott
Raleigh, N.C.

October 28, 2020
ASBI 32nd Annual Convention and Committee Meetings
Webinar

October 28–30, 2020
DBIA Design-Build Conference & Expo
National Harbor, Md.

November 18-20, 2020
PTI Level 1 & 2 Multistrand and Grouted PT Specialist Workshop
Austin, Tex.

November 21, 2020
PTI Level 1 & 2 Multistrand and Grouted PT Inspector Workshop
Austin, Tex.

January 19–22, 2021
World of Concrete
Las Vegas Convention Center
Las Vegas, Nev.

January 24–28, 2021
100th Transportation Research Board Annual Meeting
Walter E. Washington
Convention Center
Washington, D.C.

February 23–27, 2021
PCI Convention with the Precast Show and National Bridge Convention
Ernest N. Morial Convention Center
New Orleans, La.

March 28–April 1, 2021
ACI Spring 2021 Convention
Hilton & Marriott Baltimore
Baltimore, Md.

April 18–21, 2021
PTI 2021 Convention & Expo
Westin Indianapolis
Indianapolis, Ind.

July 11-15, 2021
AASHTO Committee on Bridges and Structures Annual Meeting
Indianapolis, Ind.

PHX Sky Train Stage 2 at Phoenix Sky Harbor International Airport

by Andrew Mish, Modjeski and Masters Inc.

The Phoenix Sky Harbor International Airport (PHX) is poised to see a significant increase in the number of travelers using the facility in the coming years. According to the City of Phoenix Aviation Department, 2019 was a record year for PHX, with just shy of 46.3 million passengers traveling through the airport. This was a 5% increase over the nearly 44 million passengers using the PHX facility just two years prior, in 2017.

To handle the increasing volume of PHX travelers, the City of Phoenix has undertaken a nearly $2 billion capital investment program to expand and improve airport services. A major part of this program is the Stage 2 project to build a 2.2-mile extension of the Phoenix Sky Train from Terminal 3 to the Rental Car Center. This extension will provide a direct link for passengers between the airport and the Rental Car Center. The trains will depart every 3 to 5 minutes with a travel time of 8 minutes to and from the terminals, providing a significant improvement to the current shuttle bus system, which can require up to 30 minutes of wait and travel time.

Construction is ongoing; however, precast concrete element delivery is complete, with terminal construction and train testing to run through 2021. Upon the project's completion, scheduled in 2022, the extension will allow the automated train to run from the Rental Car Center east through the airport, with stops at all existing passenger terminals and a new passenger drop-off/pickup station, before terminating at the 44th Street Station, which connects to the Valley Metro Rail serving Phoenix and other local communities.

Stage 2 Extension Bridges

Stage 2 of the Phoenix Sky Train provides two guideways for automated trains. The project lengthens the existing tracks by approximately 2.2 miles with nearly 7000 ft on guideway bridges. The guideway bridges support the

Cantilevered ends of pier girder segments with lower corbels are ready for installation of post-tensioning. Photo: Hensel Phelps.

profile

PHX SKY TRAIN AT PHOENIX SKY HARBOR INTERNATIONAL AIRPORT, STAGE 2 / PHOENIX, ARIZONA

BRIDGE DESIGN ENGINEER: Gannett Fleming, Phoenix, Ariz.

PRECAST CONCRETE ENGINEER: Modjeski and Masters Inc., Littleton, Colo.

PRIME CONTRACTOR: Hensel Phelps, Phoenix, Ariz.

TEMPORARY WORKS SUBCONTRACTOR: Pulice Construction, Phoenix, Ariz.

PRECASTER: EnCon Arizona LLC, Phoenix, Ariz., dba Tpac—a PCI-certified producer

PRECAST CONCRETE FORM SUPPLIER: Helser Industries, Tualatin, Ore.

automated people mover system, which consists of Bombardier Innovia APM 200 vehicles. Live loads used for design were provided by the vehicle manufacturer and based on the vehicle configurations that can be used with the system. The extension also includes two depressed sections to carry trains underneath two future aircraft taxiway bridges.

The elevated guideway bridges include 80 spans of precast concrete U-girders, which vary in length from 55 to 198 ft. The substructure consists of 5- to 6-ft-deep concrete pier caps on single- or double-column piers founded on drilled shafts. The column diameters range from 5 to 8 ft. All substructure concrete was cast-in-place.

The two taxiway bridges are single-span structures, which vary in length from 33 to 35 ft and are composed of precast concrete voided rectangular box girders. The project also includes a facilities access road bridge constructed using precast concrete voided slab girders with a single span of 42 ft.

In total, the bridges constructed in Stage 2 used 446 individual precast concrete girders, including two hundred ninety-six 60-in.-deep precast concrete U-girders, twenty-four 78-in.-deep precast concrete U-girders, 118 precast concrete voided rectangular box girders, and eight precast concrete voided slabs. The 78-in.-deep U-girders were used in a continuous, post-tensioned (PT) bridge unit that was constructed to span over an existing and active airport terminal building and a future taxiway. Although the bridge alignment was curved at some locations, it was possible to use straight girder segments to construct the PT bridge unit. The terminal building, Terminal 2, was in use during construction, but it saw its last flight in February 2020 and is scheduled for demolition over the next year. This unit required special staged construction analysis and significant temporary works (for details, see the Concrete Bridge Technology article on page 34).

Project Delivery

To speed project delivery from design through construction, the owner decided to use a construction manager at risk (CMAR) method. With this method, the owner retains the design team to start the design process. Once significant progress has been made on the design, typically 30% to 60%, the owner initiates a second contract to hire a construction manager. The designer and construction manager collaborate as the design progresses into the final stages. The construction manager provides input on constructability and can offer insights to the designer that may add value to the project. The CMAR arrangement also allows for earlier starts to construction activities. Once the design is significantly complete, the construction manager provides a guaranteed maximum price to the owner to construct the project.

The precaster selected for the project engaged a design firm that had expertise with precast, PT concrete design and construction to provide precast concrete engineering services.

Project Team Collaboration

One of the key components contributing to the success of the project was the collaboration of the project team. The precaster selected for the project engaged a design firm that had expertise with precast, PT concrete design and construction to provide precast concrete engineering services. The designer, contractor, precaster, and precast concrete engineer met regularly to develop the concepts and procedures that would be incorporated into the final design and construction process.

All parties invested in the success of the project and shared ideas openly throughout the entire design process. When 60-in.-deep pretensioned concrete U-girders were selected as the typical girder system for the skyway bridges, the precaster shared the capabilities of its U-girder casting beds to ensure that the designs considered the girder section and tensioning capacity of the casting facility. All girders used straight strand patterns of 0.6-in.-diameter strands, with up to 51 strands per girder. The design used a combination of debonding and top strands to control top tensile stresses at transfer of the pretensioning force. The 60-in.-deep U-girders varied in length from 55 to 103 ft and were used on the pretensioned simple-span portions of the elevated guideway bridges. For the continuous PT bridge unit, the final design used 78-in.-deep U-girders to achieve the longer spans needed to span across the future taxiway and over the existing Terminal 2 building. Arriving at this cross section was a collaborative process among the precaster, precast concrete engineer, contractor, and designer. The precaster needed to modify its existing forms to cast the 78-in.-deep sections. Girder weights were an important consideration for handling at the precast plant, shipment to the jobsite, and crane capacity for erection. All parties worked together to achieve the final design solution.

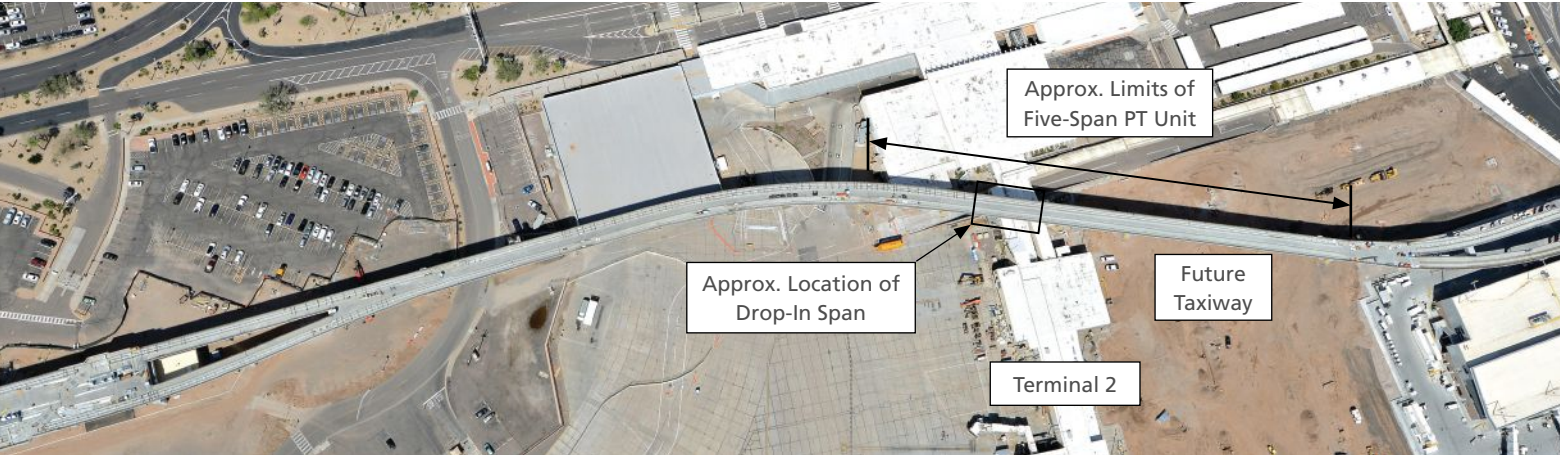
CITY OF PHOENIX AVIATION DEPARTMENT, OWNER

POST-TENSIONING SUPPLIER: DYWIDAG Systems International, Long Beach, Calif.

PROJECT DESCRIPTION: 2.2-mile-long extension of the automated train system, including 6988 ft on guideway bridges, to connect Terminal 3 of the airport to the Rental Car Center. The project included a 676-ft-long five-span, continuous post-tensioned superstructure unit with drop-in girder segments that crosses a future taxiway and a terminal building.

STRUCTURAL COMPONENTS: Two hundred ninety-six 60-in.-deep precast, pretensioned concrete U-girders, twenty-four 78-in.-deep precast, post-tensioned concrete U-girders, 118 precast concrete voided rectangular box girders, and eight precast concrete voided slabs; cast-in-place concrete composite deck slab; cast-in-place concrete pier caps on single- or double-column piers founded on drilled shafts

PHX SKY TRAIN TOTAL PROJECT BUDGET: $745 million



Aerial view of the completed PHX Sky Train guideway structure, including the five-span continuous PT unit, drop-in span, and other major features. Photo: Tpac.

Site Conditions and Construction Challenges

Constructing more than 2 miles of guideway track through an existing airport presented numerous challenges. Buried utilities had to be avoided, relocated, or carried by the new structures. Pier locations were constrained by underground utilities and clearance to existing building structures, and an access road running under the terminal walkway further restricted site access and limited crane placement, falsework locations, and girder delivery. Aircraft taxiways needed to be accommodated, and existing facility buildings had to remain operational. Most importantly, while passengers could see that construction was occurring, the owner did not want their traveling experience to be significantly affected by the project.

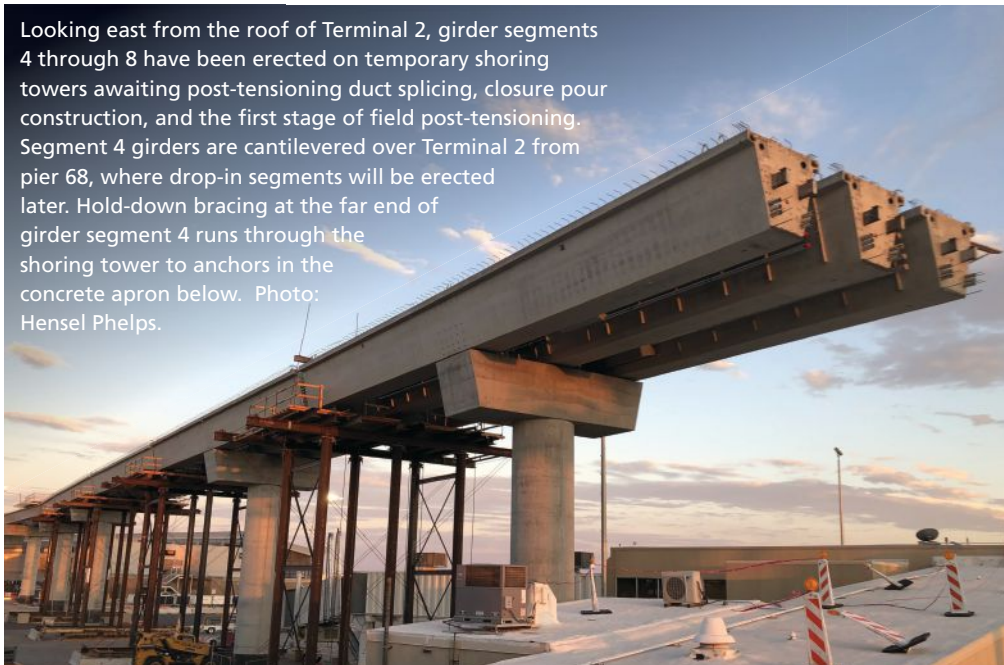
One of the more challenging design issues was the design and construction of a bridge unit over Terminal 2 and a future taxiway. Because the terminal building needed to remain in use, girder erection activities over the terminal building were restricted to the hours between 10:00 p.m. and 4:00 a.m.—from deplaning of the final flight in the terminal to morning access for the workers preparing for the first flights on the following day. Because of site constraints, falsework towers could not be located to support the girders spanning directly over the building. Special staged construction methods using embedded corbels were designed for this bridge unit to allow the girder segments over the terminal to be hung from the adjacent girder segments that cantilevered beyond the piers during this stage of construction. This meant that there was a compressed time window to complete the most complicated girder erection for the project.

Beyond the challenge of the existing conditions, the design of this bridge unit also had to span a future aircraft taxiway. The design for the taxiway required a 162-ft-wide, 40-ft-tall obstacle-free area to accommodate the aircraft clearance envelope. A span of 198 ft was required over the future taxiway, and a span of 163 ft was required to accommodate the crossing of the existing terminal building. These requirements exceeded the simple-span capabilities of the 60-in.-deep U-girders used for most of the other guideway bridges. Given the combined site constraints and longer spans required for the taxiway and terminal building in this bridge unit, the project team determined that a staged, PT structure was the ideal solution. To meet this challenge, the engineer of record led the design effort for a five-span precast, PT concrete superstructure unit at this location; this design used precast concrete U-girders to maintain a cohesive aesthetic with the rest of the elevated guideway. After an iterative design process with input from all team members, 78-in.-deep U-girders were chosen to accomplish the design objectives for this unit.

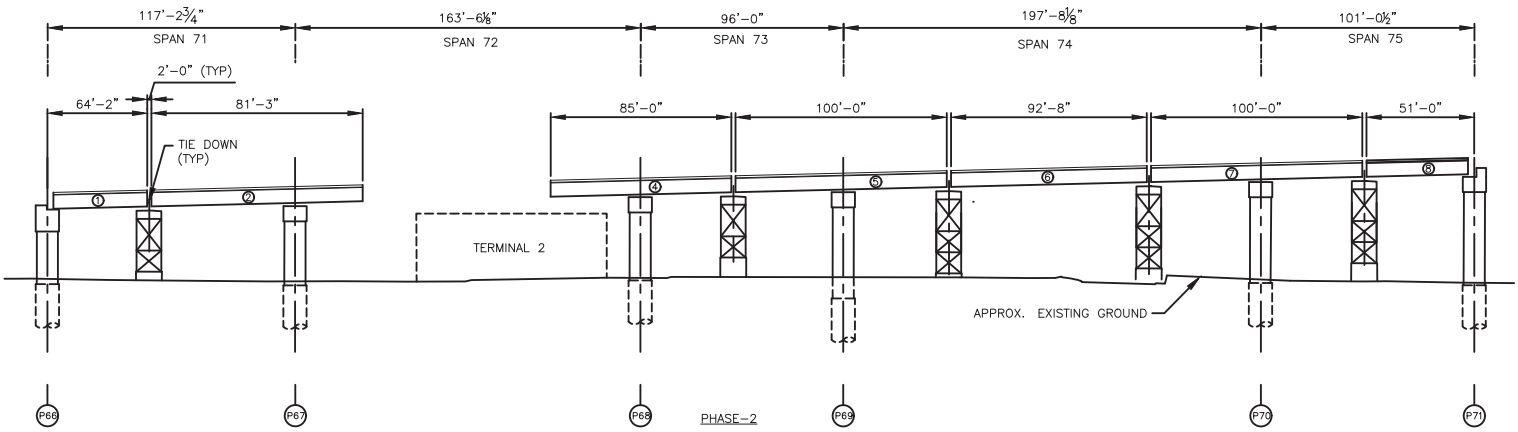
Precast Concrete Engineering and Staged Construction

For the challenging bridge unit at Terminal 2, the precaster, precast concrete engineer, and engineer of record for the project worked together to deliver a girder section that could be cast with only minor modifications to the precaster's existing U-girder forms. The final design specified 78-in.-deep post-tensioned U-girders; these were deeper than the precaster's existing forms and had thicker webs to accommodate the post-tensioning ducts while maintaining adequate clear cover to the web reinforcement. The formwork was designed to retrofit the precaster's existing U-girder forms, which helped minimize fabrication costs and reduce lead time for the formwork fabrication, delivery, and assembly.

The five-span PT unit is composed of three girder lines with eight precast



Looking east from the roof of Terminal 2, girder segments 4 through 8 have been erected on temporary shoring towers awaiting post-tensioning duct splicing, closure pour construction, and the first stage of field post-tensioning. Segment 4 girders are cantilevered over Terminal 2 from pier 68, where drop-in segments will be erected later. Hold-down bracing at the far end of girder segment 4 runs through the shoring tower to anchors in the concrete apron below. Photo: Hensel Phelps.



Original erection schematic from design plans for the five-span post-tensioned unit, with all girder segments erected except the drop-in segment, which was 87 ft 3 in. long. During construction, the shoring tower shown in span 71 was moved 20 ft closer to pier 67 and girder segment lengths were adjusted accordingly. This move caused erection stability concerns when placing segment 2 girders in their cantilevered position. Sufficient stability was provided with girder ballast and hold-down bracing at the temporary support. These stability measures were also necessary for segment 4 girders. Figure: Gannett Fleming.

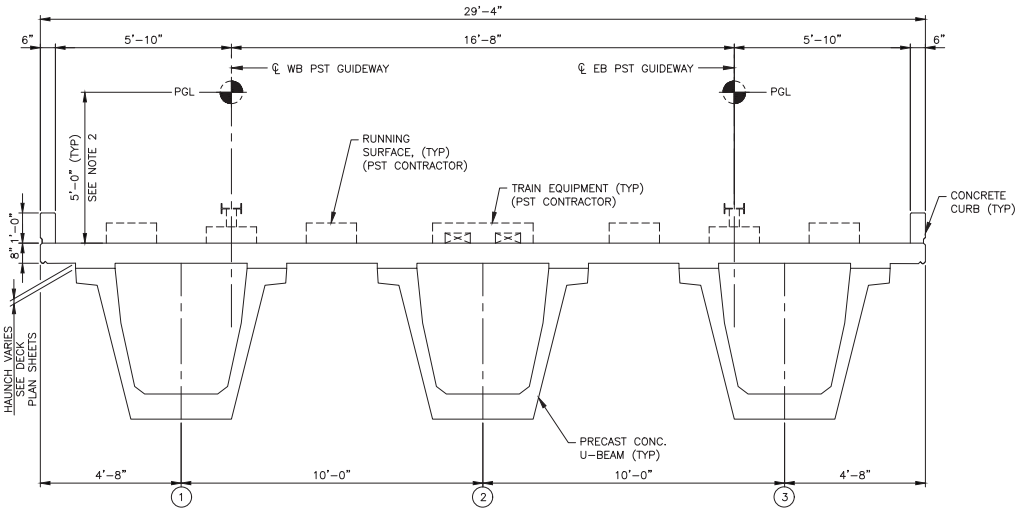
concrete girder segments in each girder line. The girder segments vary in length from 53 to 100 ft and weigh up to 210 kip. Because of the restrictive site conditions, the span over the terminal building was designed to be erected with drop-in girder segments supported using embedded hollow structural section (HSS) steel corbels. This required staged erection and post-tensioning. Girder segments 1 and 2 and girder segments 4 through 8 were erected on each side of the terminal building. Girder segments 2 and 4 spanned from falsework towers to piers with cantilevers beyond the piers. Girder segment 2 required temporary ballast weight and hold-down rods to provide a sufficient factor of safety against overturning of the segment prior to the casting of splices with girder segment 1. Following the erection of these girder segments, reinforcement was placed and ducts were spliced in the closure. Concrete was then placed between the erected segments. Pier diaphragms were also cast at this stage, with reinforcing bars placed through sleeves in the girder webs and threaded reinforcing bar couplers in the bottom slab to make an integral connection with the girders.

Next, two PT tendons, each consisting of twelve 0.6-in.-diameter strands, were tensioned. Because girder segments 2 and 4 cantilevered beyond the supporting piers, this post-tensioning—in combination with two top-flange PT tendons consisting of four 0.6-in.-diameter strands installed at the precast plant—provided the negative moment capacity to support the drop-in girder segments. These tendons were incorporated into the final design. After the drop-in girder segments were erected and the splices completed, six continuity PT tendons were tensioned and grouted in each girder line for the full-length of the unit. A composite deck slab was then cast on the girders to complete the superstructure.

Instead of using external strongbacks to support the drop-in girders over the terminal building, an embedded corbel system was designed and cast into the precast concrete girders. The corbels were composed of concrete-filled HSS tubes with welded bearing plates. Only two erection bolts per corbel were required to make the connection during erection. Given the brief erection window, this was very important because it eliminated the time that would have been needed to connect external strongback hardware to the girders. For more details on the corbel connection, see the Concrete Bridge Technology article on page xx.

This project is a testament to the flexibility and versatility of precast concrete construction. When design and construction teams collaborate, innovative solutions can be achieved safely and economically. PT concrete structures expand the bridge designer's toolbox and offer solutions to unique and complex bridge construction challenges.

Andrew Mish is project manager and office manager at Modjeski and Masters Inc. in Littleton, Colo.



Typical section for 60-in.-deep U-girder spans. The typical section for the spliced 78-in.-deep U-girder spans is similar. Figure: Gannett Fleming.

2021 PCI DESIGN AWARDS

CALL FOR ENTRIES

Entries open on **May 18, 2020**. Join us in our search for excellence and submit your projects electronically by **August 18, 2020.**

The newly improved PCI Design Awards program will showcase the winning projects in multiple ways:

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- **Full coverage in *PCI Journal*, *Ascent*, and *ASPIRE* magazines**
- **Opportunity to appear on the front cover and/or as a project feature of *Ascent***
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- **Exclusive project profile**
- **Exclusive website page**
- **Coverage in external national magazines**

The PCI Design Awards is not just looking for design excellence, but also for projects with outstanding use of precast concrete. PCI looks for projects that push the envelope and advance the precast concrete industry.

PCI Precast/Prestressed Concrete Institute

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Perspectives on Structural Behavior and Redundancy: Structural, Load Path, and Internal Redundancies

by Dr. Oguzhan Bayrak, University of Texas at Austin

Concrete bridges have long been a preferred solution for bridge owners due to their adaptability, versatility, durability, and reliability. An important attribute of concrete bridges relates to their redundancy, which can be defined as the ability of a concrete bridge at either its system level or the component level to develop alternate load paths at the strength limit state or under extreme loads. This article focuses on redundancy, with the intent of providing a succinct discussion about the effects of redundancy on structural behavior.

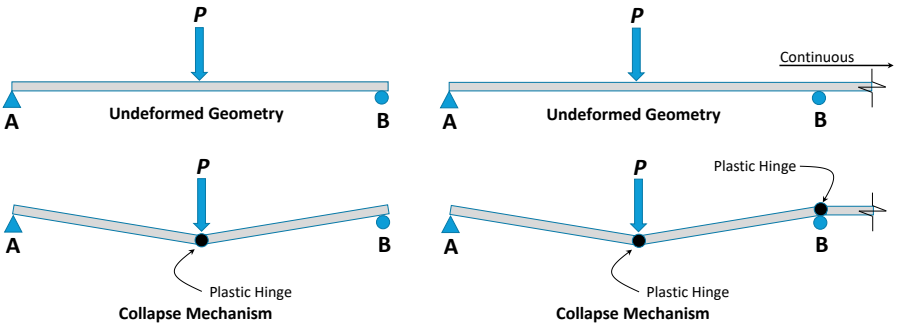
Structural Redundancy

At its most basic level, redundancy can be described by the level of indeterminacy present in a structure. The level of structural indeterminacy controls the behavior of a structure as it is gradually loaded to failure. To facilitate a discussion of collapse mechanisms as a function of redundancy, consider a reinforced concrete beam with ample shear capacity for which flexure is the controlling failure mode.

If a simply supported reinforced concrete beam supporting a concentrated load at its midspan is loaded to failure (**Fig. 1a**), the beam develops a plastic hinge at the section of maximum moment. At this point a plastic mechanism forms and the beam collapses. The beam fails in this manner because the beam is statically determinate.

If the same beam is continuous at support B (**Fig. 1b**), the beam is statically indeterminate—that is, there are more support reactions than the number of equilibrium equations that can be written to determine the support reactions. In this case, under a gradually increasing load *P*, the first plastic hinge will form either at midspan or at support B. The location where the first hinge occurs is a function of the magnitude of the applied moments at those sections, as well as the respective flexural capacities of the reinforced concrete beam at the maximum positive and negative bending moment sections. Regardless of its location, the beam does not collapse at the formation of the first plastic hinge. A second plastic hinge is needed to complete the collapse mechanism shown in Fig. 1b. The incremental load needed to form the second plastic hinge gives an opportunity for engineers, inspectors, and the general public to observe the structure in distress and take necessary actions.

Figure 1. The formation of plastic hinges in a determinate beam and an indeterminate beam. All Figures: Dr. Oguzhan Bayrak.



(a) Simply Supported Span

(b) Continuous at One End

Load Path Redundancy

To discuss load path redundancy, let us focus on a bridge with five girder lines (**Fig. 2**). In this case, assume that the bridge span under consideration is simply supported. That is to say, the reinforced concrete deck is supported on five simply supported pretensioned concrete girders that are supported on bearing pads and are free to rotate at the supports. In this case, the superstructure has load path redundancy, which is explained in the following discussion.

If one of the fascia girders shown in Fig. 2 were to be hit by a truck at location 1 and all prestressing strands were severed, we can assume that the fascia girder has failed because it has no positive moment resistance. If the reinforced concrete deck has top and bottom mat reinforcement in longitudinal and transverse directions at location 2 of Fig. 2, the load that can no longer be supported by the failed fascia girder can be transferred to the adjacent girder—with the slab serving as a cantilever to pick up the weight of the fascia girder and deck. The system of remaining girders can then transfer the load to the supporting bents at location 3 of Fig. 2. In this scenario, the superstructure has load path redundancy and can therefore support the deck. In other words, local failure in one of the supporting girders does not trigger disproportionate failure that would lead to the collapse of the total structure. In this context, the term "disproportionate failure" is used to signify total collapse of a

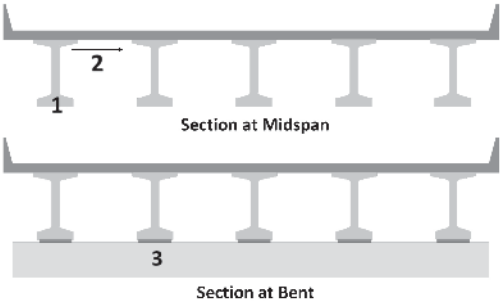


Figure 2. Load path redundancy in a pretensioned concrete girder bridge with a reinforced concrete deck.

structure that is out of proportion with the local damage experienced in one component.

It has also been observed that the bridge rail, originally designed for different loading conditions, may serve as a beam that stiffens the edge of the bridge deck, helping to carry the load to the supports. As an aside, when trucks collide with fascia girders, the superstructures often can be repaired and then kept in service. Such repairs are commonplace and routinely conducted nationwide.

Internal Redundancy

The third type of redundancy is internal redundancy of structural components. Reinforced and prestressed concrete components designed and detailed to comply with the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[1] possess a significant amount of internal stress redistribution capability (internal redundancy). To illustrate this concept, let us focus on a simply supported deep beam. Such elements are to be designed in accordance with the strut-and-tie modeling (STM) design provisions of the AASHTO LRFD specifications. Because STM is a lower-bound, plasticity-based method, the resulting designs are quite conservative. This conservatism can be attributed to the calibration of the efficiency factors[2] and the nature of all lower-bound plasticity methods.

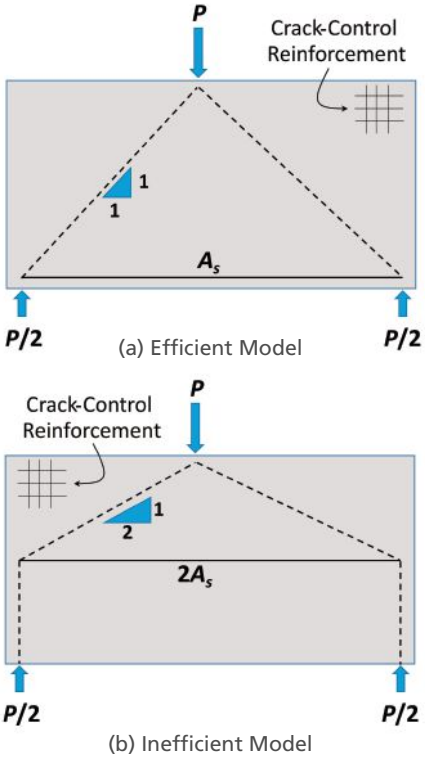
Experienced structural designers commonly make decisions to optimize their designs. **Figure 3** shows a classic deep beam example. Most bridge designers would use the solution presented in Fig. 3a. The 45-degree struts and logical placement of flexural reinforcement A_s (primary tie) result in an efficient design. With that stated, let us assume another designer chooses the model shown in Fig. 3b. If that model is used, the unusual location of the longitudinal reinforcement appears awkward at first pass. The strut inclination in this model is 26.5 degrees and, as such, doubles the force in the primary tie and results in twice as much reinforcement. Both of the structural components designed by STM will support the design loads safely. The use of crack-control reinforcement (0.3% reinforcement in each direction, as per the AASHTO LRFD specifications) will facilitate the internal redistribution of the stresses, and both models (a) and (b) envisioned by the designers will form at the strength limit state. In other words, the reinforcement detailing within the member will control the behavior of that member and hence control the development of a load transfer mechanism consistent with that reinforcement detailing. With that stated, the use of model (a) will result in the best service performance because this model more closely resembles internal compression and tension fields in an elastic model. The in-service performance is important, particularly as we aspire to design our bridges to have 70- to 100-year service lives.

To understand the internal redundancy of deep beams like the one shown in Fig. 3a, consider the test results from two deep beam tests conducted at Phil M. Ferguson Structural Engineering Laboratory at the University of Texas[2] (**Fig. 4**). The photographs in Fig. 4 focus on the portion of the beam between the load point and the left support. Support and loading plates are drawn to provide better visualization. The two test specimens depicted in Fig. 4 have different reinforcement details. The specimen shown on the left does not have any crack-control reinforcement, and the specimen shown on the right contains 0.3% crack-control reinforcement in both directions in compliance with the AASHTO LRFD specifications. All other details of these specimens were kept constant to facilitate a direct comparison. Figure 4 shows that the specimen with no crack control reinforcement developed fewer cracks as the loading was gradually increased to failure. The specimen that contained 0.3% crack-control reinforcement in the vertical and horizontal directions developed extensive flexural and shear cracking before it ultimately failed. Therefore, it can be concluded that crack-control reinforcement facilitated internal redistribution of stresses and associated cracking. As mentioned earlier, reinforced concrete elements designed to meet the requirements of the AASHTO LRFD specifications possess a fair amount of internal redundancy and the ability to form cracks to distribute and redistribute internal stresses prior to failure.

Conclusion

Concrete bridges have the advantage of possessing to varying degrees all three types of redundancy discussed in this article. Structural indeterminacy, load path redundancy, and internal redundancy serve as three layers of protection for concrete bridges. These redundancies lead to the presence of alternative load paths and give concrete bridges different lines of defense if they are subjected to extreme loading conditions during their service lives. As we move forward in the 21st century, bridge engineers must give due consideration to redundancy when introducing new structural systems, technologies, and construction methods.

Figure 3. Internal redundancy of a member accommodates two different strut-and-tie model designs.



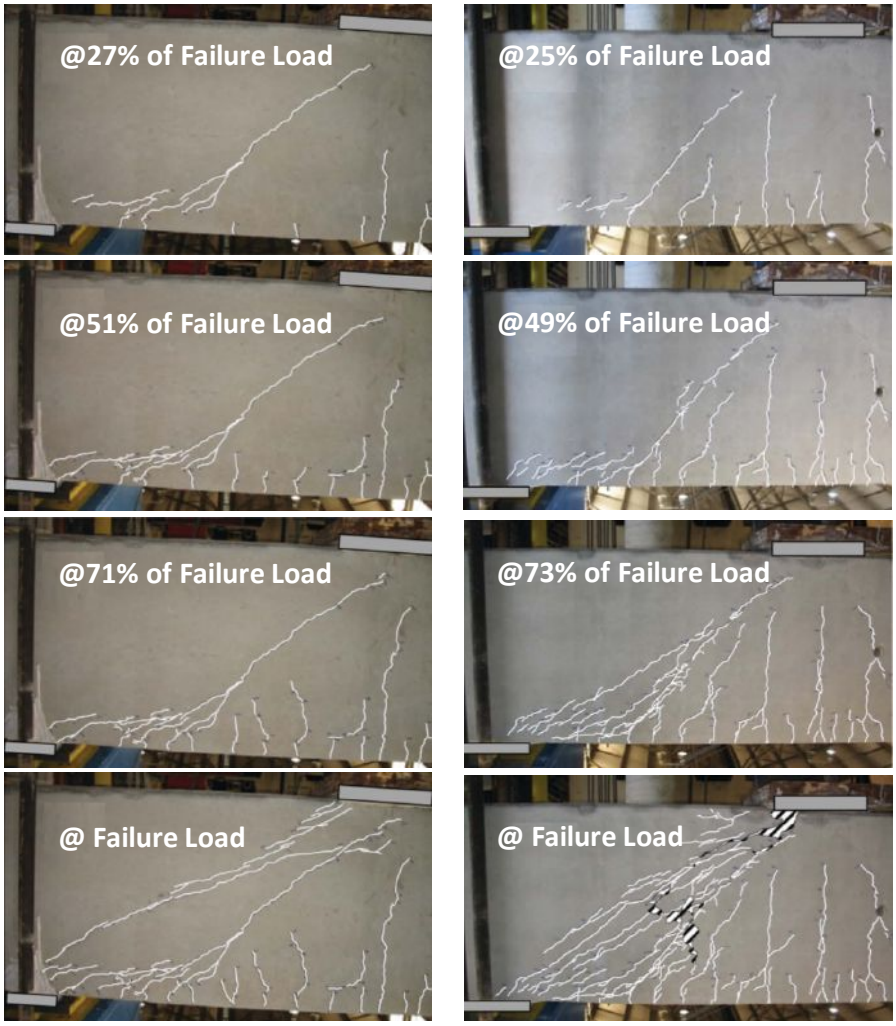


Figure 4. Redistribution of internal stress (internal redundancy) is illustrated by the different cracking behaviors of two deep beams with different reinforcement. The specimen on the left does not have any crack-control reinforcement, whereas the specimen on the right contains 0.3% crack-control reinforcement in both directions.

References

1. American Association of State Highway and Transportation Officials (AASHTO). 2017. *AASHTO LRFD Bridge Design Specifications*, 8th ed. Washington, DC: AASHTO.
2. Birrcher, D.B., R.G. Tuchscherer, M.R. Huizinga, O. Bayrak, S.L. Wood, and J.O. Jirsa. 2009. *Strength and Serviceability Design of Reinforced Concrete Deep Beams*. Technical Report 0-5253-1. Austin: Center for Transportation Research, Bureau of Engineering Research, University of Texas at Austin.

Dr. Oguzhan Bayrak is a professor at the University of Texas at Austin.

A Time for Transformational Change in Civil Engineering Education

by Dr. John Schemmel, Texas State University

In the context of civil engineering, the term "infrastructure" refers to the structures and systems that are vital to the economy, health, and prosperity of a population center. Whether publicly or privately owned, infrastructure assets tend to be physically large and have a high initial cost, with variable long-term maintenance costs. Bridges, energy grids, water and wastewater treatment plants, hospitals, and educational institutions are just a few examples of infrastructure assets. Traditionally, civil infrastructure assets have been designed to meet the functional needs of the owner and provide safe operation by the general public. Facility maintenance often occurs only after a repair need is identified and implemented. For example, most bridges are inspected on a two-year cycle. Unless there is prominent, easily identified damage to a bridge, maintenance will only take place following the routine inspection of the bridge.

Recently, terms such as "sustainability" and "resilience" have been used to describe the longer-term performance of an asset. As populations grow and natural resources are depleted, engineers must now think beyond initial design and construction and imagine how an asset will be monitored, managed, and maintained during its life span. In addition, engineers must attend to the continued safe use of existing infrastructure assets. Advances across a broad range of interdisciplinary technologies are now enabling engineers to monitor the condition and performance of assets in real time, making maintenance proactive instead of merely reactive.

Advances in Asset Monitoring

Although the use of technology to monitor and maintain infrastructure assets is rapidly advancing, it is still in its infancy. However, each time a new monitoring device, power supply, data storage system, analytic method, or asset management tool is released, we inch closer toward widespread technology implementation.

The ability to remotely monitor a structure, component, or process over time and then use the observational data to manage the asset will profoundly impact scientific and engineering professions. It is envisioned that technology will ultimately advance to the point of autonomous life-cycle management, such that every infrastructure asset will become a living structure with self-monitoring and repair capabilities. The feedback loop from the use of these technologies will contribute to fundamental changes in the way assets are designed, constructed, repaired, maintained, and managed. These advancements will produce opportunities for transformative change and innovation in the ways that scientists and engineers are educated, the research that is conducted, and the commercialization of new technologies. Eventually, a tipping point will be reached, and asset monitoring and maintenance will become standard operating procedure.

Technology-Enhanced Infrastructure

Today, terms such as "smart cities," "smart infrastructure," and "structural health monitoring" are commonly used to refer to the use of technology to monitor and maintain infrastructure. However, these terms do not encompass the full array of technologies that can contribute to the improved life-cycle performance of an asset. Moreover, the term "smart" implies the technology has an autonomous reaction to an event, which is not always the case with all infrastructure monitoring systems. The term "technology-enhanced infrastructure" (TEI) may be a more appropriate descriptor because TEI encompasses all technology—smart or otherwise—that contributes to the management and maintenance of infrastructure. With TEI (**Fig. 1**), event-detection devices, data-harvesting and transmission systems, cybersecurity, big data analytics, and predictive models are used to improve the tools employed in the maintenance of assets such as highway bridges. It is envisioned that this paradigm shift will stretch current thinking about infrastructure and the role that technology can play to enhance the safety, longevity, and economic value of any asset.

As consulting firms, construction companies, departments of transportation, owners, and others begin to understand and appreciate the benefits of implementing TEI concepts, each stakeholder will do so with increasing regularity. One challenge these groups will face, particularly in the next decade or so, is where to find college graduates with the necessary skills to evaluate and implement TEI products and services. A classic civil engineering program does not have the flexibility to add TEI-related courses to its mainstream curriculum. Programs may initially have to add such courses as electives, if electives are an option in their curricula. Consequently, civil engineering education is on the cusp of a truly transformational change. New academic programs will need to include TEI concepts throughout their proposed curricula. The Accreditation Board for Engineering and Technology (ABET) and the American Society of Civil Engineers will need to recognize the importance of TEI principles and therefore modify their accreditation requirements and program criteria, respectively. Existing academic programs will need to examine their curricula to determine how they can include TEI concepts within their current framework.

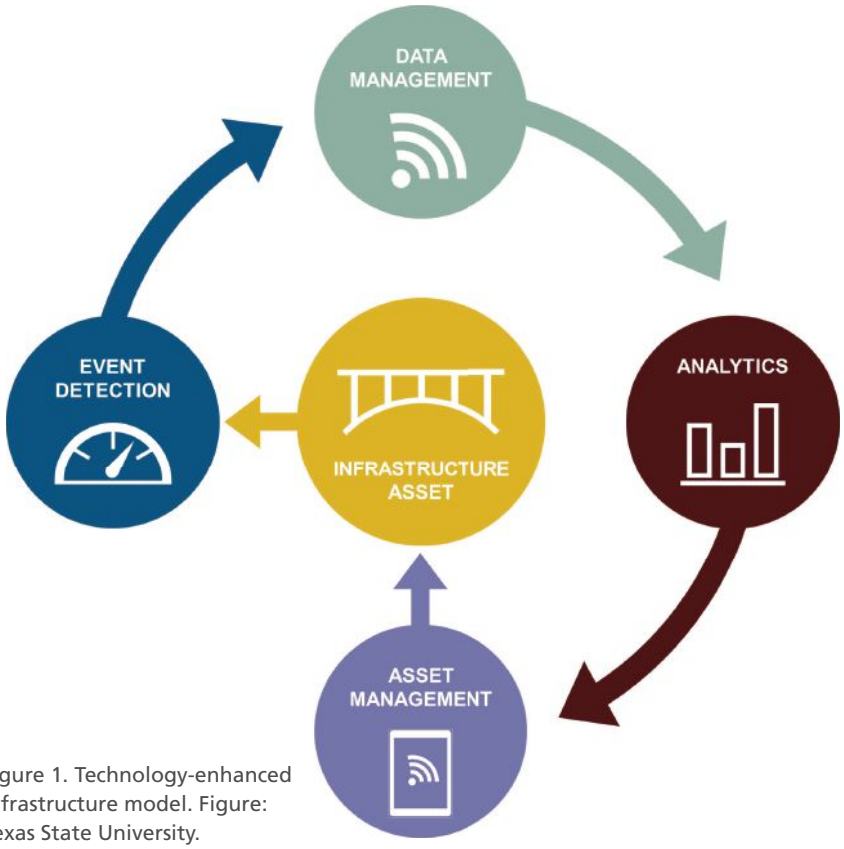


Figure 1. Technology-enhanced infrastructure model. Figure: Texas State University.

Integrating TEI in Civil Engineering Education

One example of a novel approach to engineering education is the new civil engineering program in the Ingram School of Engineering at Texas State University. A required five-course sequence developed in collaboration with the computer science and geography departments introduces students to technology and planning concepts from a civil engineering perspective. All traditional classes include at least one aspect of event detection (sensors), data management, analytics, or asset management related to the core content of the course. For example, embedded strength sensors are covered in the reinforced concrete design course, and water quality data analysis is included in the environmental engineering class. Senior design projects will include monitoring plans along with drawings and specifications. Current ABET requirements are met, and students learn the benefits of TEI through hands-on monitoring exercises and coursework.

It will take considerable effort for the more than 250 ABET-accredited civil engineering degree programs to include TEI courses or concentrations or shift the programs' collective focus to TEI. Regardless of the difficulties and challenges associated with moving the civil engineering undergraduate education in the direction of TEI concepts, this effort must begin today. Otherwise, the civil engineering profession will be behind the curve with respect to reaping the benefits modern technology can provide in terms of monitoring and managing the nation's infrastructure.

Acknowledgment

This article was written with input from Evan Humphries, Texas State University.

EDITOR'S NOTE

Electrically isolated tendon (EIT) systems are examples of the TEI concept discussed in this article. EIT technology was installed in the Coplay-Northampton Bridge, which is discussed in the Project article on page 20 of this issue of ASPIRE®*. The system, which was installed at the request of the Federal Highway Administration to demonstrate EIT technology, will allow long-term monitoring of the condition of several post-tensioning tendons, as discussed in a Concrete Bridge Technology article in the Spring 2019 issue of* ASPIRE.

The Coplay-Northampton Bridge, which has sensors installed on several post-tensioning tendons to monitor the condition of those tendons, is an example of a technology-enhanced infrastructure asset. Photos: Dr. Clay Naito.

PROJECT

Grand Avenue Bridge
Glenwood Springs, Colorado

by Clint Krajnik, RS&H Inc.

Overhangs with tapered coffers illuminated by architectural lighting. All Photos and Figures: RS&H Inc.

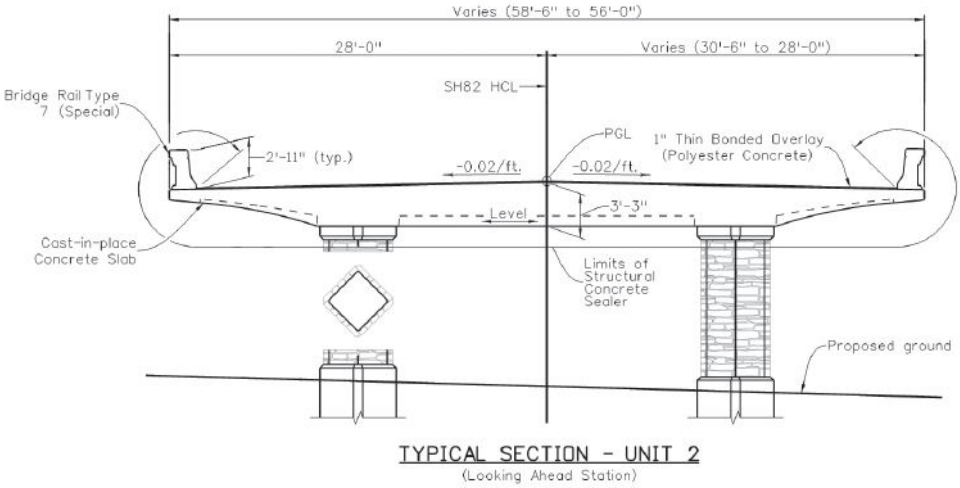
A half century after its construction in 1953, the nine-span steel plate girder Grand Avenue Bridge in Glenwood Springs, Colo., was functionally obsolete, with travel lanes just over 9 ft wide. The Colorado Department of Transportation (CDOT) decided the structure had to be replaced and sought a durable, aesthetically pleasing design for the new viaduct.

Aesthetic and Functional Concrete Design

The new viaduct has two units. Unit 1 of the new viaduct carries Grand Avenue (State Highway 82) over Interstate 70, the Colorado River, and the Union Pacific Railroad on a curved steel superstructure. Unit 2, known as the downtown unit, is an architecturally enhanced cast-in-place concrete slab bridge that descends into the city's historic central business district, serving as the aesthetic focal point of the project.

For the 197-ft-long downtown unit, which has three spans (60, 77, and 60 ft) and includes the spans over 7th Street and a planned pedestrian plaza, the project team selected a 3-ft-thick cast-in-place concrete slab bridge. This design provides a thin structure to improve overhead clearance for the plaza below and opened possibilities for creative concrete forming of the slab soffit to enhance the user experience. A concrete slab superstructure was also preferred for its sound-deadening attributes, which would mitigate noise from traffic on the bridge reaching the plaza area beneath it. As another noise-abatement measure, a solid parapet was chosen for the bridge rail.

Typical section of the cast-in-place conventionally reinforced concrete slab bridge. Coffer recesses on the bottom of the slab were up to 9 in. deep.



Cast-in-Place Concrete for an Accelerated Schedule?

Aesthetic opportunities aside, the decision to use cast-in-place concrete at first seemed counterintuitive. The construction schedule for the last two spans of unit 1 and the entire downtown unit was limited to a 95-day window during which Grand Avenue would be closed as the bridge was replaced. This tight timeline might seem to rule out cast-in-place concrete, which is not considered a speedy

profile

GRAND AVENUE BRIDGE / GLENWOOD SPRINGS, COLORADO

BRIDGE DESIGN ENGINEER: RS&H Inc., Denver, Colo.

PRIME CONSULTANT: Jacobs Engineering, Denver, Colo.

OTHER CONSULTANTS: Bridgescape LLC, Columbia, Md.

PRIME CONTRACTOR: Granite/RLW Joint Venture, Glenwood Springs, Colo.

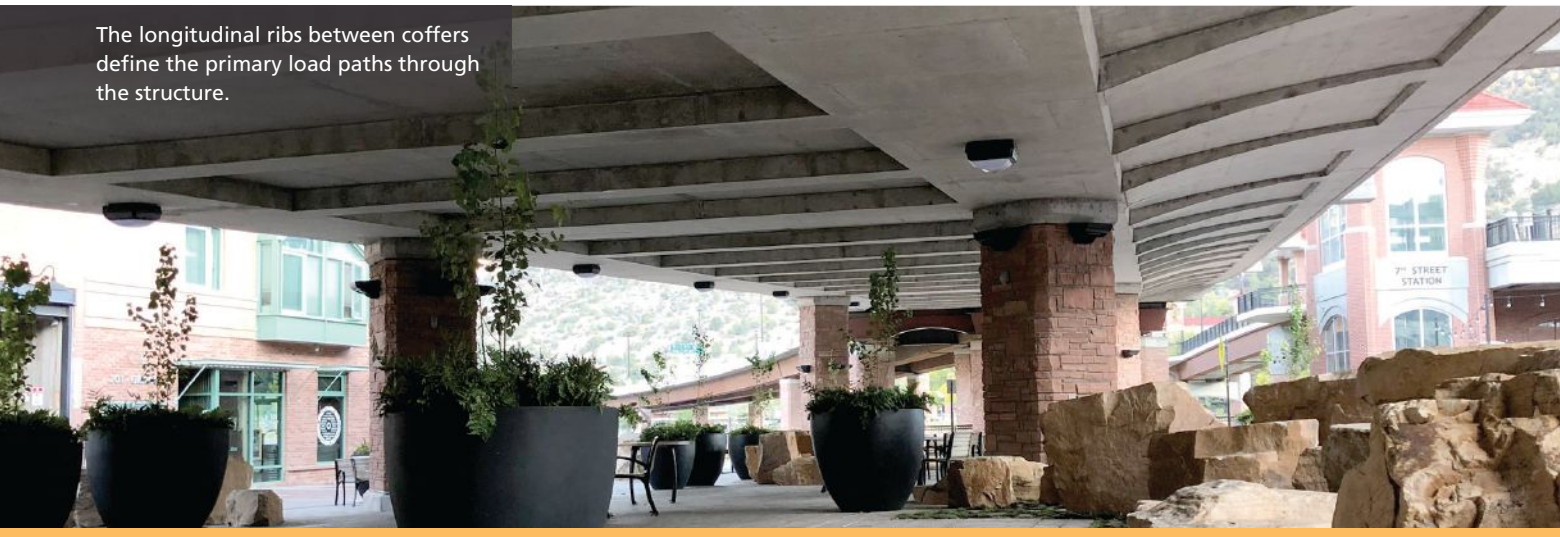
Epoxy-coated reinforcement was used in the slab. The reinforcement projecting into the slab from an integral column is visible.

Shading on the tapered overhangs produces a visually thin superstructure. The stone finish on the columns matches the color of the mountains in the area.

construction method. However, cast-in-place concrete construction does not require the use of large cranes, and that was an advantage on this project where construction equipment had to fit between adjacent downtown buildings. Also, because 7th Street was permitted to be closed during construction, the contractor was able to use economical scaffolding-style falsework to construct the unit.

To help accommodate the accelerated closure schedule, several of the bridge's spread-footing and drilled-caisson foundations were strategically located under or outside of the existing bridge footprint, allowing their installation prior to the closure. The slab for the downtown unit was cast in one continuous concrete placement of roughly 940 yd^3. A single shift crew worked 15 hours and placed 65 to 70 yd^3 of concrete per hour to construct the slab. During the closure, the contractor increased the crew size to work in shifts 24 hours per day, 7 days per week to increase production rates. CDOT provided an incentive for the contractor to finish early and a disincentive for finishing late. According to Roland Wagner, CDOT's project manager, "The public was supportive of a shorter-duration full closure instead of the long-term hassle of a phased approach, so we went with the full closure and did what we could to minimize impacts." Construction on the downtown unit started in August 2017, and the structure was open to traffic 10 days ahead of schedule in November 2017. The project was completed in June 2018.

Post-tensioning of the slab bridge was considered to achieve an even thinner structure, providing additional overhead clearance while gaining the serviceability



The longitudinal ribs between coffers define the primary load paths through the structure.

COLORADO DEPARTMENT OF TRANSPORTATION, OWNER

BRIDGE DESCRIPTION: Unit 2: Three-span, 197-ft-long, cast-in-place conventionally reinforced concrete slab bridge with epoxy-coated reinforcement

STRUCTURAL COMPONENTS: Unit 2: Cast-in-place concrete coffered slab with varying-thickness overhangs supported on cast-in-place concrete columns, spread footings, and drilled shafts

BRIDGE CONSTRUCTION COST: \$2.8 million (\$255/ft^2 of bridge deck) for Unit 2

AWARD: 2018 Rocky Mountain Chapter ACI Excellence in Concrete Award, Infrastructure Category

Scaffolding-style falsework was used for construction of the new bridge in close proximity to existing buildings. This style of falsework was also a factor that made the cast-in-place concrete slab economical. The use of simple scaffolding and large crews helped accelerate construction.

benefits of prestressed concrete. However, the extra steps of tendon installation, tensioning, grouting, and installing concrete cover caps would have increased the duration of the Grand Avenue closure, which was already pushing the boundary of public acceptance.

Aesthetic Innovations

Given the planned pedestrian plaza area under the bridge, Frederick Gottemoeller, the project's bridge architect, suggested two key improvements over a typical slab bridge. First, typical blunt slab edges were replaced by graceful, 10-ft-wide tapered and arched overhangs. "The taper allows more natural light to enter the space under the bridge, and the arched profile complements the historic character of Glenwood Springs," Gottemoeller said. "The taper also makes it impossible for an observer to judge how thick the slab actually is. It looks like it is only as thick as its edge. That allows the downtown unit to fit better with the delicate 19th-century architecture only a few feet away."

Further paying homage to the traditional aesthetic, a coffered bottom soffit was proposed. The "coffers" consist of large, 9-in-deep rectangular recesses in both the flat middle portion of the slab and within the tapered overhangs. "The coffers are effective at imposing a pedestrian scale to what would otherwise be a wide, smooth expanse of concrete," Gottemoeller noted. A pattern of rectangular impressions was added to the back of the concrete barrier to complement the coffers. Another aesthetic enhancement was the elimination of pier drop caps throughout the bridge. Instead, the structure has integral pier caps that reduce the visual mass at the piers and allow unobstructed views from under the bridge toward the city's historic architecture, the Colorado River, and the beautiful mountain scenery. The nearly 60-ft-wide bridge used only two widely spaced columns per pier, which further opened the pedestrian plaza. The square columns of the piers were rotated 45 degrees relative to the bridge, producing a "diamond" orientation to enhance the aesthetics of the plaza. Additionally, a rose-colored, ashlar-pattern cut-stone veneer was applied between the formed concrete capitals and pedestals of the columns, abutment, and wingwalls.

Atypical Slab Bridge Behavior

Concrete slab bridges with mild reinforcement are among the easiest structure types to design. Equations for live load distribution can be combined with a simple beam-model analysis and force effects are easily calculated. Typically, the structural capacity is also easy to calculate using the 1-ft-wide strip assumption of a solid section. However, the architectural enhancements on the Grand Avenue Bridge project complicated the slab's behavior.

"Because of the ribs created by the deep coffers, the presence of wide tapering overhangs, and the discretely spaced columns that frame into the superstructure, we knew early on that this wasn't going to be a typical slab design," said Jack Garrison, RS&H's structural design engineer for the downtown unit. To capture the true behavior, the bridge was analyzed using a three-dimensional plate model in LARSA 4D software. "We used varying shell thicknesses to model the changes in section depth at the coffers and through the tapered overhang, with the discrete columns modeled as beam elements," Garrison said.

With the refined analysis approach, Garrison learned that force effects from both dead and live loads funneled quickly to the two main longitudinal ribs that lined up with the columns, thus minimizing the force effects in the integral cap between columns. "The results showed us where the reinforcing steel was really needed, and the longitudinal ribs, especially near the columns, required the most concentrated reinforcement, while the integral pier caps required only a nominal amount of steel," he explained.

Durability Mitigation

Durability of the reinforced concrete structure was enhanced by using epoxy-coated reinforcement and a polyester concrete overlay, both of which are proven mitigation measures against the corrosive magnesium chloride used on Colorado's roads during the winter. In lieu of a colored concrete coating, exposed concrete surfaces were left in their raw natural state, a preference shared by both the architect and the local stakeholder groups, to create a more authentic aesthetic within the historic downtown setting. The contractor used enhanced procedures, forming techniques, and systems to achieve the best quality natural concrete finish. For protection, a flat-finish clear silane concrete sealer was applied to all exposed concrete surfaces.

Conclusion

In November 2017, after the contractor finished construction 10 days early, more than 3000 people attended a bridge walk-on and opening ceremony just before the structure was opened to traffic. "The public response was overwhelmingly positive," said Wagner. "The downtown unit is now a popular gathering spot for the historic downtown area, with movie nights under the bridge and live concerts also using the new venue."

Clint Krajnik is the Denver Bridge Group leader with RS&H Inc. in Denver, Colo.

Free Webinars

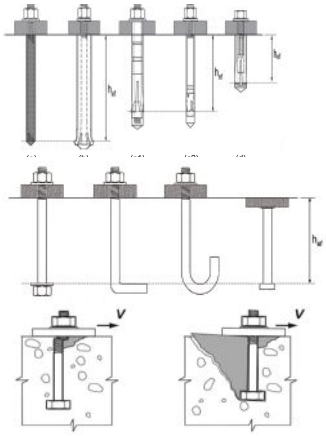
PCI
Precast/Prestressed Concrete Institute

Implementing the New AASHTO LRFD Article 5.13 on Anchoring to Concrete

To register for these webinars, go to pci.org/PCI/Education/Webinars.

Purpose of the five webinars: Accelerate implementation of the new LRFD section for concrete anchor design and minimize overlap of state highway agencies' implementation efforts through the use of a "deployment kit" for nationwide implementation.

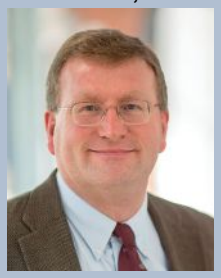
Article 5.13, Anchors, of the reorganized section 5 of the AASHTO LRFD *Bridge Design* Specifications, as found in the eighth and ninth editions, has adopted by reference chapter 17 of ACI 318-14 for the procedures to design, detail, and install anchors, with amendments as appropriate for application to highway bridges. Thus, wholesale reproduction of the voluminous ACI provisions is avoided. Professor Ron Cook of the University of Florida, the primary author of ACI 318-14 chapter 17, assisted with the development of the adopted LRFD article 5.13.



A team of experts assembled by PCI,

Don Meinheit,

Ron Cook, and



Neal Anderson,

will deliver five webinars to the bridge community that address how an agency may elect to implement the findings of National Cooperative Highway Research Program (NCHRP) Reports 639 and 757, which were codified.

Schedule of the five-part webinar series hosted by PCI:

Webinar	Topics	Date	Time (Eastern)
Anchor 1: Scope	Scope and objectives, LRFD loads, and anchorage behavior	Tuesday, July 7, 2020	2:00 p.m. – 4:00 p.m.
Anchor 2: Design	Code anchor design	Thursday, July 9, 2020	2:00 p.m. – 4:00 p.m.
Anchor 3: Product qualifications	Product qualification and ESR information	Tuesday, July 14, 2020	2:00 p.m. – 4:00 p.m.
Anchor 4: Selection and procurement	Calculations, selection, and procurement	Thursday, July 16, 2020	3:00 p.m. – 5:00 p.m.
Anchor 5: Inspection and compliance	Construction/inspection and compliance inspection and testing	Tuesday, July 21, 2020	3:00 p.m. – 5:00 p.m.

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Coplay-Northampton Bridge: Innovative Rehabilitation of a Historic Pennsylvania Structure

by Thomas J. McNavage Jr., AECOM

This project involves the rehabilitation of the existing seven-span viaduct structure carrying State Route 7404 (Chestnut Street) over the Lehigh River, Norfolk Southern Railroad, Bridge Street, and the Ironton Rail Trail in the boroughs of Coplay, Lehigh County, and Northampton, Northampton County, Pa. Almost nine decades after its construction in 1930, the historic structure required rehabilitation due to significant deterioration, inadequate load-carrying capacity, and public safety concerns.

At the time of its closure, the existing bridge carried approximately 11,000 vehicles per day. It also had two sidewalks for pedestrian traffic, but one sidewalk was already closed due to significant deterioration of the concrete railing. Rampant spalling of the concrete encasement of the steel girder–floorbeam spans necessitated the installation of a canopy to protect users of the Ironton Rail Trail, which crosses under the structure. Deteriorated components of the concrete arch spans led to a reduced load-carrying capacity that required load posting of the existing bridge. Additionally, a collapsed drainage culvert on the east side prompted sinkhole concerns, which would need to be addressed in the design of the rehabilitated structure. The rehabilitated structure was designed to take advantage of the newest available technology to produce a nearly maintenance-free structure type.

Aerial photo of the original Coplay-Northampton Bridge (constructed circa 1930). The existing bridge was eligible for listing in the National Register of Historic Places because it included three structure types typical of the era. Photo: Ann Bartholomew, Delaware and Lehigh Canals. Easton, Pa.: Center for Canal History and Technology.

Site-Related Challenges

The rehabilitation project faced substantial challenges related to site constraints and the historic significance of the original structure. The approaches of the 1116-ft-long existing bridge were located adjacent to relatively

profile

COPLAY-NORTHAMPTON BRIDGE / BOROUGHS OF COPLAY AND NORTHAMPTON, PENNSYLVANIA

BRIDGE DESIGN ENGINEER: AECOM, Conshohocken, Pa.

GEOTECHNICAL ENGINEER: Dawood Engineering, Inc., Harrisburg, Pa.

PRIME CONTRACTOR: Trumbull Corporation, Pittsburgh, Pa.

PRECASTER: Northeast Prestressed Products, Cressona, Pa.—a PCI-certified producer

POST-TENSIONING CONTRACTOR: DYWIDAG-Systems International, Bolingbrook, Ill.

closely spaced buildings. Retained fill approaches of 206 ft and 94 ft on the west and east ends, respectively, connected the existing bridge to the intersections of Chestnut Street and Front Street in Coplay and 9th Street and Main Street in Northampton. The existing structure was eligible for listing in the National Register of Historic Places because it was composed of three distinct structure types in use at the time of construction: a 350-ft-long three-span steel girder–floorbeam system on the western approach, three concrete arch spans in the 548-ft-long main unit over the river, and a single-span 218-ft-long steel truss on the eastern approach.

Combining the Old with the New

The project involved an elaborate superstructure replacement and is considered to be a structure rehabilitation because it retained significant portions of the existing substructure units. The existing steel girder-floorbeam spans were replaced with prestressed concrete PA bulb-tees made continuous for live load. The existing single-span steel truss was replaced with two spans of prestressed concrete PA bulb-tees also made continuous for live load over the new reinforced concrete pier founded on micropiles. The arch spans were replaced with fully continuous spliced prestressed concrete PA bulb-tee units—the first use of this structure type in Pennsylvania. This technology was chosen for use at this location because of the desire to retain the existing pier footings for the river spans and the recent approval of spliced prestressed concrete girder standards by the Pennsylvania Department of Transportation (PennDOT).

The design team believed it would be advantageous to reuse the existing, massive (21 by 64 ft in plan, and nearly 30 ft deep) concrete footings of the river piers if they were structurally sound. Concrete testing of the pier footings was performed, and the concrete was found to be in excellent condition, with a compressive strength of 10 ksi and minimal chloride contamination. Retaining the existing pier footings saved considerable time and expense.

Testing indicated that the existing piers could be incorporated into the rehabilitated structure. Dowels were installed to tie the new pier to the existing footings. Retaining the existing pier footings saved considerable time and expense. Photo: AECOM.

The river spans were 181, 181, and 186 ft long, from west to east. Initially, these span lengths were considered too long for the use of prestressed concrete beams, so a steel superstructure was proposed. However, standards for spliced precast, prestressed concrete beams had been recently approved by PennDOT, prompting the design team to evaluate that structure type as an alternate. Ultimately, the team concluded that precast concrete would be economically competitive.

Shipping considerations such as truck-turning radius and vertical clearance limited the girder segments to 125 ft 0 in. in length and a maximum depth of 115 in. Seven beam segments were originally proposed for each fully continuous beam, with a total of five beams ranging in depth from 79 in. to 115 in. over the piers. During construction, the number of segments was reduced to five after alternate shipping routes were identified. Beam segments were delivered to the project

Pier segment delivery and erection. Beam segments were delivered to the project using trailers with steerable dollies and erected from a temporary causeway constructed in the Lehigh River. Photo: AECOM.

LEHIGH COUNTY, OWNER

BRIDGE DESCRIPTION: Eight-span, 1116-ft-long bridge with widths between 44 ft 8¼ in. and 50 ft 8¼ in. The bridge is composed of a three-span, 350-ft-long approach unit with prestressed concrete bulb-tee beams made continuous for live load; a three-span, 548-ft-long, fully continuous post-tensioned, spliced prestressed concrete bulb-tee beam main unit; and a two-span, 218-ft-long approach unit with prestressed concrete bulb-tee girders made continuous for live load. Multicolumn bent piers were constructed on reused foundations.

STRUCTURAL COMPONENTS: For the main unit: spliced prestressed concrete modified (increased web thickness) PA bulb-tee beams post-tensioned for full continuity. The unit was constructed with five girder lines with five segments each, including haunched pier sections; four post-tensioning tendons for each girder line, with one tendon per girder using electrically isolated tendon (EIT) components. For the approach spans on both ends of the bridge: conventional prestressed concrete PA bulb-tee beams made continuous for live load; piers doweled into existing footings with two existing pier foundations stabilized using limited-mobility grouting; one additional reinforced concrete pier constructed and founded on micropiles. All spans have an 8-in.-thick reinforced concrete deck slab.

BRIDGE CONSTRUCTION COST: $25.8 million (total project).

Girder splice with strongback support. The 1-ft-wide gap between girder segments for post-tensioning duct splicing was later filled with 9000-psi cast-in-place concrete, which was placed simultaneously with the diaphragms. Photo: AECOM.

using trailers with steerable dollies and, in some instances, drivers had to back the trailers nearly one mile to deliver the segments to the site. The beam segments were erected from a temporary causeway constructed in the Lehigh River, which washed out repeatedly in heavy rains. The contractor ultimately decided to construct a concrete footing on the causeway to provide a temporary support that was less likely to be compromised as hurricane season approached.

Once in place and temporarily supported, the beams were made continuous by the placement of a 1-ft-wide, 9000-psi, cast-in-place concrete closure at each splice. Upon completion of the concrete closures, four post-tensioning tendons, each tensioned to 660 kips and consisting of fifteen 0.6-in.-diameter seven-wire strands, were used to tie the segments together. With the beams fully continuous, the temporary supports were then removed and the 8-in.-thick deck was constructed in the conventional manner.

During construction, the Federal Highway Administration (FHWA) elected to make the Coplay-Northampton Bridge a demonstration project with respect to the use of electrically isolated tendon (EIT) systems. PennDOT and both Lehigh and Northampton counties agreed to allow the use of EIT technology on the bridge. EIT technology uses special anchorage hardware for the post-tensioning tendons and links the reinforcement to form an electrically continuous loop through the entire beam. After the tendons were tensioned and grouted, electrical resistance was measured at the beam end using a multimeter. If the resistance measured was above a calculated level, the tendon was considered fully encapsulated in grout and thus fully protected from corrosion. One tendon per beam used the EIT technology, and the short-term tests indicated that they were fully encapsulated. Lehigh University will be performing long-term EIT monitoring of the spliced girders (see the related Concrete Bridge Technology article in the Spring 2019 issue of *ASPIRE*® for more details).

The contractor was required to provide a mock-up of a single 540-ft-long tendon duct to demonstrate the ability to completely encapsulate the strands in the tendon with grout from end to end. Clear plastic duct was used at critical points along the profile for inspection purposes. Photo: AECOM.

Aesthetics, Accessibility, and Traffic Flow

In cross section, the rehabilitated bridge accommodates two lanes of traffic and varies in width from 44 ft 8¼ in. to 50 ft 8¼ in. to accommodate a turn lane at the east end. The bridge also has an 8-ft-wide sidewalk with a vertical wall barrier (1 ft 0 in. wide, 3 ft 6 in. high) on the south side and a safety-shape concrete barrier (1 ft 8¼ in. wide, 3 ft 6 in. high) on the north side. The structure is supported on full-height cantilevered reinforced concrete abutments, which use the existing abutment footings for support, and seven reinforced concrete multicolumn bent piers. Approximately 110 lineal ft of protective fence are mounted on top of the south bridge barrier in the span over the Norfolk Southern Railroad.

AESTHETICS **COMMENTARY**

by Frederick Gottemoeller

Replacing a 90-year-old historic landmark is always a challenge, particularly when the old bridge includes three different structural systems, and none of the three lends itself to emulation by modern structural systems. If the goal is to reflect some aspect of the old bridge in the design of the new bridge, then the challenge becomes, "Which of the old systems do we respond to?"

Thankfully, the widening acceptance of spliced precast concrete girder technology provided an answer for this structure. It allowed the precast concrete girders for the three longest spans over the river to be haunched at the piers. Those girders thus recall the haunched steel girder spans of the original bridge. This decision also adds visual interest to the bridge. The haunched girders make evident where the forces in the bridge are the greatest and give observers an idea of how the bridge is working.

The original haunched steel girder spans also provided the inspiration for the bridge's new piers, which emulate the features of the old piers. That provides observers another recollection of the old bridge. Finally, replicating the towpath apron along the former canal gives future users of the Delaware and Lehigh Trail another feature that they can relate to the old bridge. It is easy to understand why local officials are so pleased with the results of this project.

The existing concrete towpath apron attached to the face of the pier adjacent to the former Lehigh Canal was retained and rehabilitated to its original condition. A trail will be constructed under the bridge. Photo: AECOM.

Decorative lighting poles are mounted on the barriers at blister locations, generally at the substructure units and along the retaining walls at each corner. The light poles and luminaires were selected to match those of the nearby Pine Street Bridge.

In addition to decorative lighting, other architectural elements were included in the design of the bridge to mitigate the effects of the rehabilitation and reflect the historic significance of the existing structure and its location. The piers were designed as multicolumn bents, keeping the same style as the existing approach span piers. All piers incorporated a horizontal, incised pattern to replicate the one used on the existing bridge. The existing concrete towpath apron attached to the face of the pier immediately adjacent to the former Lehigh Canal—which is intended to be part of the future Delaware and Lehigh Heritage Trail network—was retained and rehabilitated to its original condition. An interpretive panel describing the Lehigh Canal, which has been filled in at this site, and its historic significance to the region was also installed at this location.

Extending the Service Life

The rehabilitated structure is designed to achieve PennDOT's goal of 100-year service life for bridges. Use of deck joints was minimized to the fullest extent possible, and the selection of prestressed concrete spliced girders will reduce maintenance needs and extend service life compared to a steel structure. Epoxy-coated and stainless steel reinforcing bars were used in areas with a high potential for corrosion. Additionally, the remaining portions of the existing substructure units received a coating of epoxy resin as a permanent sealant.

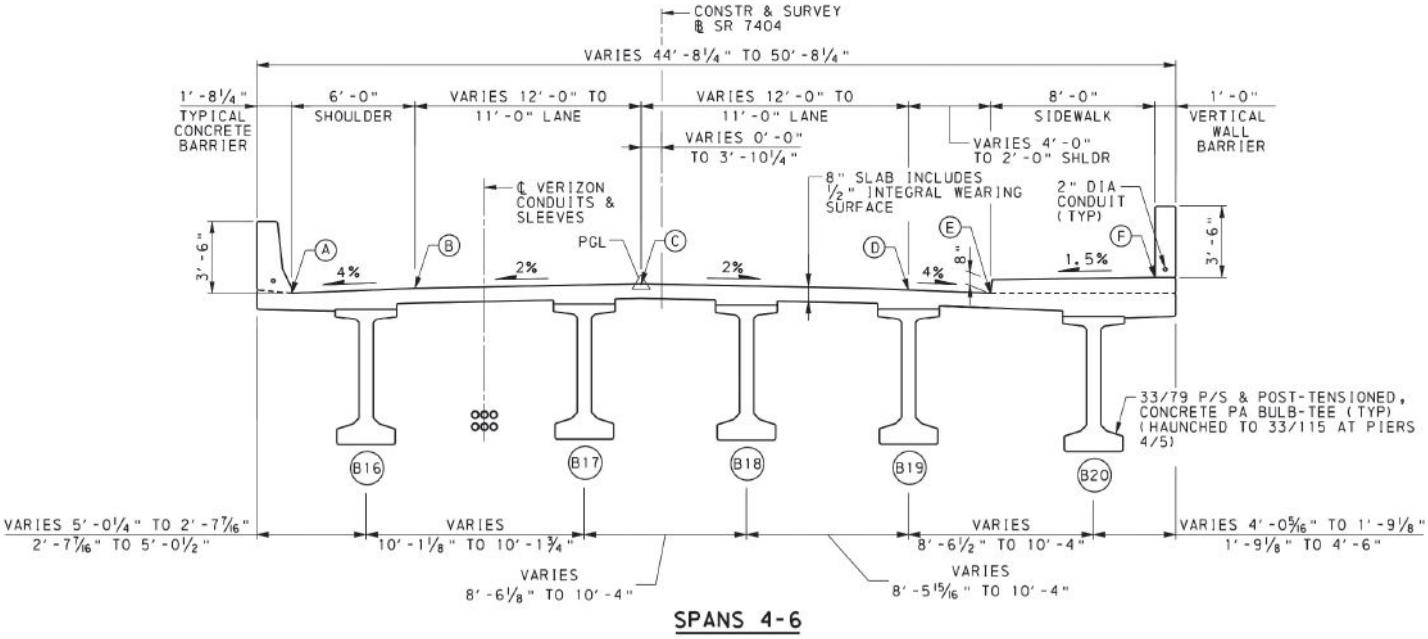
Conclusion

The rehabilitated bridge—which is dedicated to Brigadier General Anna Mae Hays (1920–2018), a former resident of the Coplay-Northampton area, and the first woman in the U.S. Armed Forces to be promoted to a General Officer Rank—was opened to traffic on December 19, 2019. Construction is scheduled to be completed on June 30, 2020. Lehigh County officials have indicated that they are extremely pleased with the final structure and the cooperation exhibited by all involved parties to renew a vital transportation link in the region.

View of the completed bridge looking west. The horizontal, incised pattern on exterior pier columns mimics the details of the original bridge. Remaining portions of the existing piers are coated with epoxy resin. Photo: AECOM.

Thomas J. McNavage Jr. is a project engineer with AECOM in Conshohocken, Pa., and the engineer of record for the Coplay-Northampton Bridge.

Typical section of main unit that used modified (thickened web) PA bulb-tee beams spliced and post-tensioned for full continuity—the first use of this structure type in Pennsylvania and the first use of electrically isolated tendons in the United States. Typical sections of the approach spans are similar, but the bulb-tee beams were not spliced. Figure: AECOM.



Sweep in Precast, Prestressed Concrete Bridge Girders—Part III

by Dr. Bruce W. Russell, Oklahoma State University

This article is the third and final article in a series focusing on sweep, or lateral bowing, in prestressed concrete bridge girders. The first article focused on potential causes of sweep and provided recommendations for actions that can be taken at the prestressing plant to mitigate its effects (see the Spring 2019 issue of *ASPIRE*®). The second article concentrated on the lifting, transportation, and erection of girders with sweep (see the Fall 2019 issue of *ASPIRE*). This article focuses on effects that girders with sweep will have on long-term stresses, strength, and other performance criteria. Also, the effects of attempting to partially straighten a girder to bring it within tolerance are examined. By the time you reach the article's conclusion, you may agree with me that sweep in prestressed concrete girders that is within, or in some cases beyond, the published tolerance may not have significant adverse effects on the long-term performance of a bridge.

Table 1. Girder Design Details for Example

Girder Design:	Bruce W. Russell, PhD, PE	
Length:	130.5 ft	
Type of Girder:	PCI-AASHTO BT-72	
A =	767	in.^{2}
h =	72.0	in.
c_b =	36.6	in.
c_t =	35.4	in.
I_{xx} =	545,113	in.^{4}
I_{yy} =	37,543	in.^{4}
J = torsional inertia =	6673	in.^{4}
w_b =	0.799	kip/ft
f'_c =	10.0	ksi
E_c =	5760	ksi
Width of bottom flange	26	in.
Width of top flange	42	in.
N =	38	0.6-in.-diameter Gr. 270 ASTM A416, low-relaxation strands
Strand eccentricity =	29.02	in.
Δ_{camber} =	2.72	in.↑ estimated at time of handling and includes girder self-weight

Design Example Illustrating Sweep's Effects

To provide an example for discussion, a PCI–American Association of State Highway and Transportation Officials (AASHTO) bulb-tee BT-72 girder with a span length L of 130 ft 6 in. and a girder spacing of 7 ft 2 in. is used. **Table 1** presents the girder cross-section and material properties as well as other design information. For purposes of discussion in this article, one of the girders is assumed to exhibit a midspan sweep f of 1.631 in., which is equal to the PCI recommended sweep tolerance f_{tol} of ⅛ in. per 10 ft of length.[1] Other cases are considered where the sweep varies from zero to twice the PCI tolerance.

In this article, four cases are considered:

- The effects of sweep after girder erection but prior to deck slab placement
- The effects of sweep during deck slab placement

Figure 1. Schematic plan and elevation of girder with sweep showing vertical reactions, torsional restraint at bearings from bracing or other means, and flexural moment at midspan for self-weight only. Note: For illustration purposes sweep is exaggerated. All figures: Dr. Bruce Russell.

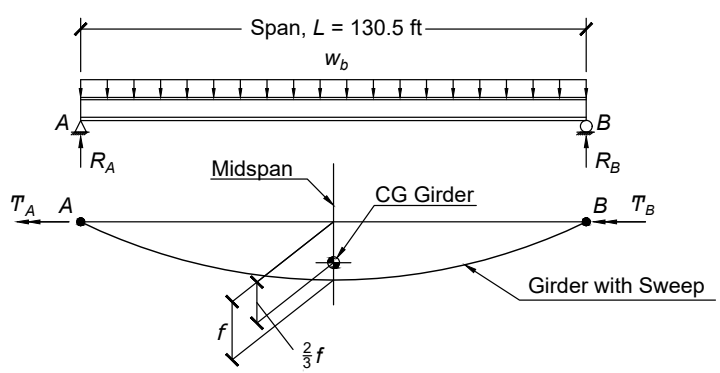


FIGURE 1a. SCHEMATIC ELEVATION AND PLAN VIEWS

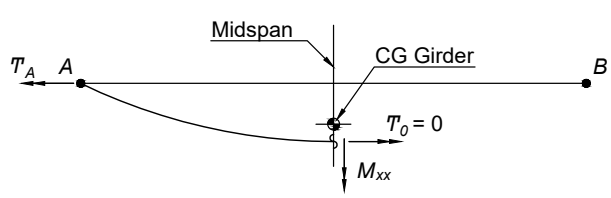


FIGURE 1b. MECHANICS AT MIDSPAN, PLAN VIEW

Table 2. Effects of Sweep on Girder After Erection, Prior to Deck Slab Placement

Sweep f	Sweep at Midspan f, in.	CG of Girder from Centerline of Bearings $\frac{2}{3}f$, in.	Torsional Restraint Required at Bearing T_A, kip-ft	Equivalent Eccentricity at Bearing $e_T = T_A / R_A$, in.	Stable? (Yes or No)	Torsional Deformation at Midspan ϕ, rad	Maximum Torsional Stress τ, ksi
0.0	0.00	0.00	0.00	0.00	Yes	0.0000	0.000
0.5 f_{tol}	0.82	0.54	2.36	0.54	Yes	0.0009	0.017
1.0 f_{tol}	1.63	1.09	4.72	1.09	Yes	0.0018	0.034
1.5 f_{tol}	2.45	1.63	7.09	1.63	Yes	0.0027	0.052
2.0 f_{tol}	3.26	2.18	9.45	2.18	Yes	0.0036	0.069

Notes:
1. Calculations are based on a BT-72 girder with length L = 130.5 ft. Vertical reaction at bearing R_A = 52.13 kip for all cases.
2. Sweep tolerance f_{tol} = 1.631 in. is computed from a PCI standard practice[1] where sweep tolerance is $\frac{1}{8}$ in. per 10 ft of length.

- The effects of sweep in the completed composite bridge superstructure during service (primarily live loads)
- The effects of reducing sweep in the girder after erection but prior to deck slab placement

Effects of Sweep After Girder Erection, Prior to Deck Slab Placement

For illustration purposes, this discussion focuses on the forces, stresses, and deformations in the example girder with sweep that are caused by self-weight. **Figure 1a** shows a schematic plan view of a girder with sweep as it rests on its bearings. The girder spans 130.5 ft from center to center of the bearings with a sweep f equal to 1.631 in. Note that the center of mass (CG) of the girder is located at $\frac{2}{3}f$ from a line connecting the centerlines of the bearings, which are assumed to be centered under the girder ends. The eccentricity of the girder's CG creates an imbalance of the girder's self-weight w_b, which must be resisted by torsional restraint T_A and T_B at the bearings. The restraint may be provided by the bearings alone or additionally by end bracing, which is commonly installed as soon as the girders are erected. The torsional restraint required at each support is computed by the following equation:

$$T_A = T_B = \frac{w_b L}{2} \times \frac{2f}{3} = \frac{w_b L}{3} \times f$$
$$= \frac{(0.799 \text{ kip/ft})(130.5 \text{ ft})}{3} \times \frac{(1.631 \text{ in.})}{12 \text{ in./ft}} = 4.72 \text{ kip-ft}$$

Table 2 reports the calculation results for varying amounts of sweep.

At midspan, the torsional moment T_0 is zero, both horizontal (transverse) and vertical shears are zero, and the out-of-plane bending moment M_{yy} (not shown) is also taken as zero (**Fig. 1b**). Note that regardless of sweep, the vertical reaction at the supports is constant, $R_A = R_B = 52.13$ kip. The bending moment at midspan about the girder's strong axis M_{xx} also remains constant:

$$M_{xx} = \frac{w_b L^2}{8} = \frac{(0.799 \text{ kip/ft})(130.5 \text{ ft})^2}{8}$$
$$= 1700.9 \text{ kip-ft}$$

Girder sweep causes both torsional deformation, or twist, and shear stresses. Maximum torsional rotation θ occurs at midspan, whereas the maximum torsional shear stress τ occurs at the ends of the girder. Table 2 lists the values for rotations and stresses as a function of sweep. Calculation of these values, which is not shown here, uses the St. Venant's torsional inertia J given in Table 1. The value of J for this example was based on thin-walled theory and is conservative. Computation of torsional rotations and accompanying stresses is typically not performed in bridge design except for curved beams.

For the example case, with sweep equal to the sweep tolerance, the angular deformation at midspan is 0.0018 rad (about 0.10 degree). This angular deformation at midspan results in a lateral deflection of the top fiber of about $\frac{1}{16}$ in., which would be nearly imperceptible. The maximum torsional shear stress for this same case is 34 psi, which would appear to be a small enough increment to not cause principal tensile stress cracking or raise any serviceability concerns. Even with sweep equal to twice the tolerance in the example girder, torsional stress is only about 70 psi. Overall, the numbers in this simplified example demonstrate that cracking or other serviceability problems due to the effects of self-weight and sweep alone are unlikely.

Table 2 also shows the result of the computation of whether sweep in the example girder is likely to cause the girder to overturn on its bearings. The eccentricity of the equivalent couple from the torsional effects is easily calculated by dividing the torque T_A by the vertical reaction R_A. In this approach, the girder can be considered stable for cases where the eccentricity of the torsional moment is located within the middle third of the bearing, which ensures that the full width of the bearing remains in compression. The width of the BT-72 bottom flange is 26 in. A typical bearing pad width for this beam would be 22.5 in., which accounts for the ¾ in. bottom chamfers and a 1 in. offset on each side of the girder; therefore, the middle third extends to $\frac{22.5 \text{ in.}}{6} = 3.75$ in. on each side of the centerline of bearing. If the girder sweep f equals 1.631 in., the calculation for the eccentricity of the torsional moment is:

$$e_T = \frac{T_A}{R_A} = \frac{(4.72 \text{ kip-ft})(12 \text{in./ft})}{52.13 \text{ kip}}$$
$$= 1.09 \text{ in.} \leq 3.75 \text{ in.} \checkmark \text{ OK}$$

Even for the scenario in which sweep is twice the tolerance, Table 2 shows that the girder is stable when resting on its supports. Even if the girder has no sweep, bracing each end of each girder with blocking or other devices is recommended to prevent tipping and overturning of the girder in the event of wind or other lateral loads that may be applied to the girder during construction. Temporary bracing represents good practice, even if the structural computations suggest that the girder is stable. Depending on the length of the girder, midspan or multiple-point bracing may be necessary during construction. Guidance is provided in the PCI *Recommended Practice for*

Figure 2. Schematic plan and elevation of girder with sweep showing vertical reactions, torsional restraint at bearings from bracing or other means, and flexural moment at midspan for the weight of fresh concrete at completion of deck slab placement. Note: For illustration purposes sweep is exaggerated.

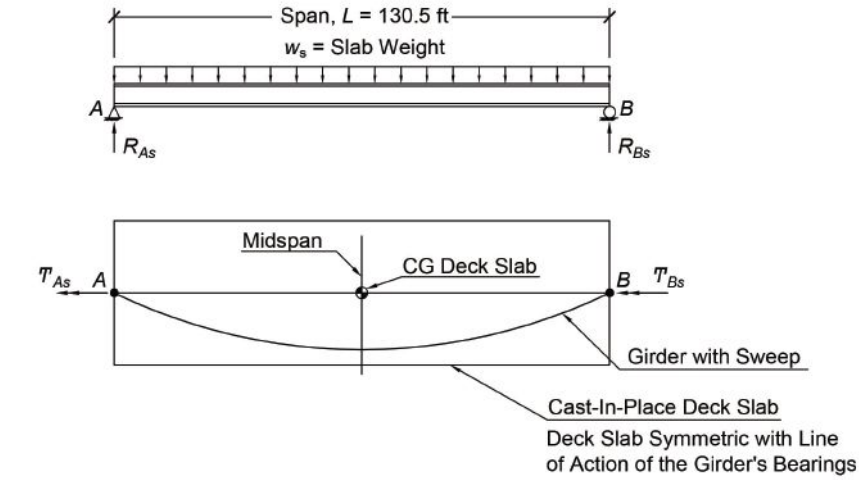


FIGURE 2a. SCHEMATIC ELEVATION AND PLAN VIEWS

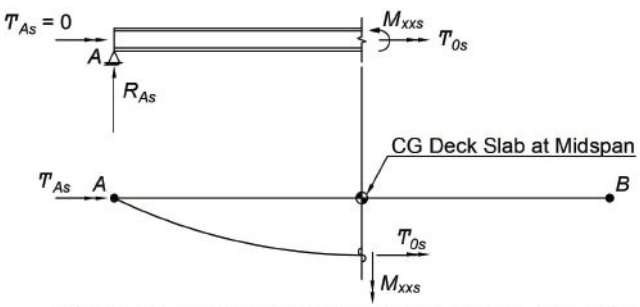


FIGURE 2b. MECHANICS AT MIDSPAN-ELEVATION AND PLAN VIEWS

Lateral Stability of Precast, Prestressed Concrete Bridge Girders.[2]

Effects of Girder Sweep During Deck Slab Placement

This part of the discussion focuses on the stresses and deformations that occur within the example bridge girder with sweep as it supports the deck slab's fresh concrete weight. Construction loads, including workers, formwork, and other equipment, especially on exterior girders, should be included in a detailed analysis, but they are ignored here for simplicity of discussion. Note that stresses and deformations caused by self-weight on a girder with sweep exist prior to deck slab construction. It is assumed that girders are sufficiently braced at the time of deck slab placement.

The results of the following approximate analysis may be surprising to some. As shown in **Fig. 2**, which presents both plan and elevation views of the bridge girder with sweep supporting the weight of fresh concrete for a cast-in-place deck slab, the weight of the fresh concrete deck slab on the bridge girder with sweep does not produce torsional stresses or deformations at girder ends, if the deck is placed the full length of the girder in a single placement. (Phased construction or partial deck placement is a different situation and must receive due consideration.) For this analysis, the 8-in.-thick deck slab is assumed to be symmetrically placed about the line of action between the girder's bearings.

Figure 2a shows the arc of the centerline of the girder with sweep and the line of action between the girder's bearings. The CG of the deck slab is located directly over the line of action; therefore, there is no torsional moment at the girder ends. However, the girder will experience torsion at midspan due to the eccentricity of the deck load.

Figure 2b provides both elevation and plan views of the girder with sweep from the support to midspan. To calculate the deflection and stresses for the fresh weight of the deck slab concrete for a girder with sweep, we use the same calculations used for a straight girder (without sweep). Accordingly, there are no out-of-plane moments, no out-of-plane shears, and no torsion at the girder ends caused by the self-weight of the deck slab. It is important to analyze the effect of out-of-tolerance sweep on exterior girders. Overhanging formwork or a concrete finishing machine may place loads on an exterior girder that could possibly exceed in-service conditions.

Effects of Girder Sweep in the Completed Composite Bridge Superstructure

For this discussion, only the live loads are considered because dead loads applied to the composite section tend to be small compared to live loads. In this example, the service live load moment on an interior girder is 1621.6 kip-ft.

Note that for an interior girder, the live load moment and shear are also applied on the line of action between the bearings, and the horizontal eccentricity of the girder sweep does not produce torsional effects in the composite cross section. The analysis for the composite cross section is similar to that shown in Fig. 2 for addressing the effects the fresh concrete weight of the slab.

The primary effects caused by girder sweep when considering an individual girder with composite deck slab section for traditional longitudinal bending stress analysis are that the composite cross section becomes asymmetric near midspan, and the principal axes are not precisely vertical and horizontal. As the principal axes rotate, this also creates bending moments about both of the principal axes. However, the applied live load bending moment (1621.6 kip-ft)—which is produced by the vector addition of the two moments about the principal axes—does not change. For the case considered with sweep equal to the sweep tolerance, the rotation of the principal axes is about −0.024 rad (less than −2 degrees), which produces additional lateral moment and stress of 38.54 kip-ft and 19 psi, respectively. Therefore, the total bending stress due to live load only at the bottom of the prestressed concrete

girder is 0.942 ksi (tension), which is about 2% greater than the 0.923 ksi bending stress in the cross section without sweep. This analysis demonstrates that the effects of girder sweep on the completed system for this bridge are very small.

The analysis just discussed assumes that the isolated girder and deck slab section can deflect laterally and rotate to generate bending moments in the lateral direction. When a single girder and its composite deck slab are considered, the lateral deformation remains very small—on the order 0.065 in. at midspan for the example case. However, one must recognize that any lateral deformation from the composite section of a single girder with sweep will be resisted by the whole system of girders, presumably without sweep, that are connected by the composite deck slab. Therefore, the effects on the girder when considered in the composite cross section will be further reduced from what was previously computed, and there will be virtually no lateral deformations, and therefore no stresses, in the superstructure system related to the girder with sweep.

Having considered the effect of sweep on the girder in the composite section, we will now turn to consider the effects of girder sweep on the deck slab design, which will be shown to be small. Using the example bridge, if it is assumed that the span of the deck slab increases from 65 in. (7 ft 2 in. spacing minus one-fourth of the flange width on each side) to 66.6 in. due to sweep at midspan equal to the tolerance, bending stresses in the slab due to a single point load will increase by about 2.5%.

In this simplified example, the effects of girder sweep on the stresses, serviceability, and strength in the completed bridge have been shown to be small when the girder is considered to be independent from the rest of the bridge. However, these minor effects due to asymmetry are restrained by the rest of the bridge because the composite deck ties together all girders, preventing independent girder rotation. In other words, the effects of a girder with sweep on the completed bridge are small enough that the owner should be reasonably assured that girder sweep within tolerance does not cause a significant change of the stresses and deformations due to live load in the completed structure.

What about exterior girders with sweep? Exterior girders are not generally balanced in the slab loads that they resist, so the unbalanced load from sweep would be evaluated using standard procedures.

Effects of Reducing Sweep After Girder Erection

The preceding sections show that sweep within tolerance, and in some cases even greater than tolerance, has a small effect on stresses and deformations in the prestressed concrete girder and the completed bridge. Therefore, after a girder is erected, it is generally not necessary, nor desirable, to attempt to reduce a sweep that is within, or possibly even greater than, the recommended tolerance.

The owner or engineer may wish to consider reducing the amount of sweep in a girder if the sweep exceeds tolerance. A girder with sweep should be "straightened" only to the extent that the sweep is reduced to within tolerance. For example, a girder that has a sweep of twice the sweep tolerance $2f_{tol}$ could be straightened to reduce the sweep to f_{tol}. The contractor, in consultation with an engineer and the owner, may consider straightening an out-of-tolerance girder because of concerns about deck formwork fit-up or final acceptance of the finished structure.

The first step should be to analyze the girder after erection and in the finished structure, similar to the analyses in the preceding sections, to determine whether out-of-tolerance sweep will have detrimental effects on the bridge or if the effects are small enough to be acceptable to the owner and engineer. If the analyses indicate that straightening seems advisable, a straightening plan can be developed and an analysis performed that considers the entire system that will be engaged as the force is applied to push or pull the girder with sweep into tolerance. The contractor will require engineering services to safely carry out any type of straightening scheme, as straightening a girder requires significant engineering, planning, and bracing of all affected bridge elements. Most straightening schemes will produce lateral bending and torsion in all girders involved. Although it may be possible to design a straightening method that does not twist the girders, applying such a method will further complicate the straightening operation and require more hardware, effort, and engineering expertise.

There are many possible schemes for straightening girders, and the means and methods of straightening are beyond the scope of this article. However, for the sake of this discussion, we will explore how one girder can be straightened by pushing or pulling from two adjacent girders by using diagonal bracing elements connected to the adjacent girders near their supports. **Figure 3** illustrates one possible concept for straightening a single girder using two adjacent girders for bracing. Of course, all three girders will experience lateral forces, deformations, and possibly torsion. The forces required to straighten one girder are significant, and an engineer must design or review the detailing required for attachments and hardware, as well as the effect of the straightening activity on the adjacent girders.

When straightening a girder, one method is to apply a lateral point load at midspan. The horizontal force required to "straighten" the girder can be calculated using a basic, classic mechanics formula. If our example girder has a midspan sweep of $2f_{tol}$ equal to 3.26 in., then the horizontal force F_H required to straighten the girder to within the sweep tolerance of 1.63 in., with the change in sweep taken as Δf, is:

$$F_H = \frac{48EI_{yy}}{L^3} \times \Delta f$$

$$= \frac{48(5760\text{ ksi})(37{,}543\text{ in.}^4)}{(130.5\text{ ft})^3(1728\text{ in.}^3/\text{ft}^3)}$$

$$\times(3.62\text{ in.} - 1.63\text{ in.}) = 4.42\text{ kip}$$

This calculation assumes the force is applied at the centroid of the girder cross section. It also assumes that permanent end diaphragms have not yet been placed so the ends of the girder are free to rotate in the horizontal plane but are braced against overturning.

Table 3 lists F_H for reducing varying amounts of sweep. As expected, F_H increases in proportion to the amount of "straightening" that is desired. Table 3 also shows the out-of-plane

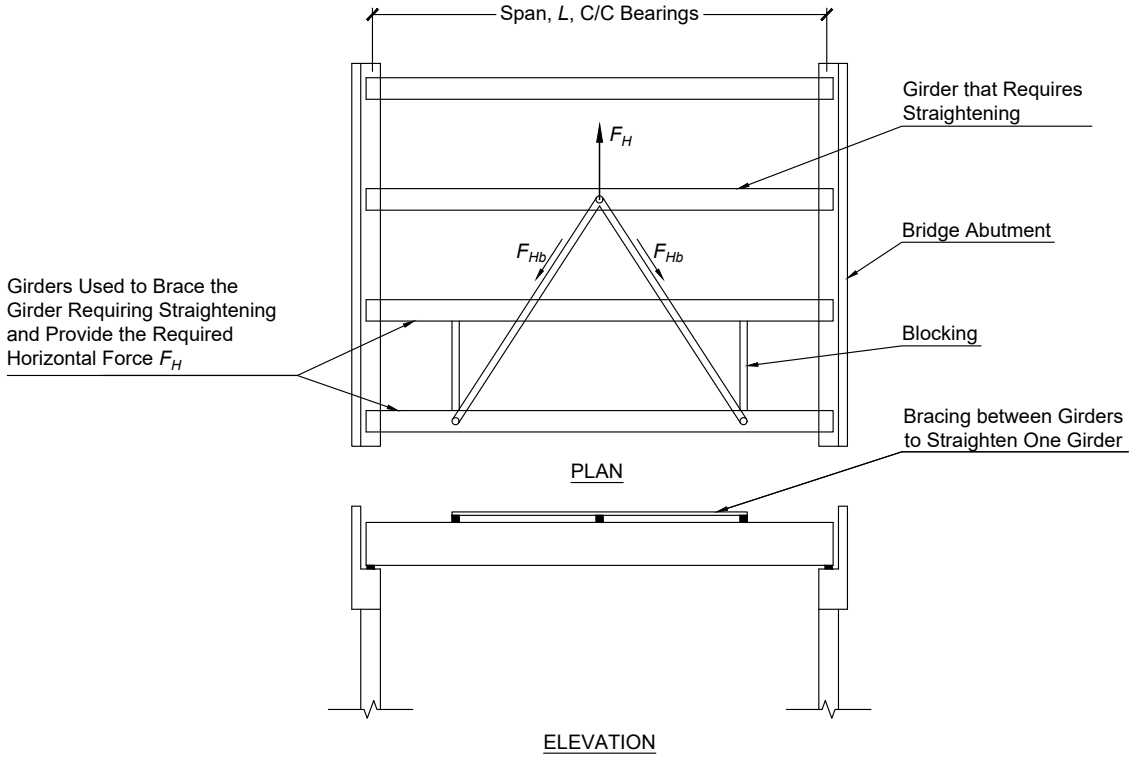


Figure 3. Schematic for possible scheme to straighten one girder in a four-girder span. Lateral girder bracing is required for stability at supports. Note: For illustration purposes girder spacing is exaggerated with respect to the girder span.

bending moment and the related bending stress that result from the straightening force being applied at the centroid of the girder cross section. These bending stresses about the weak axis are computed at the extreme top corner of the BT-72 top flange, which is 42 in. wide, and are in addition to stresses that already exist in the girder. For the example case, the lateral bending moment is 143.8 kip-ft and the additional stress at the corner of the top flange is 0.966 ksi.

$$f_t = \frac{M_{yy}c}{I_{yy}} = \frac{(143.8 \text{ kip-ft})(21 \text{ in.})(12 \text{ in./ft})}{(37{,}543 \text{ in.}^4)}$$

$$= 0.966 \text{ ksi}$$

Table 3 compares this additional stress to the modulus of rupture (MOR) f_r for the girder concrete. The MOR is given by the formula $f_r = 0.24\sqrt{f'_c} = 0.24\sqrt{10 \text{ ksi}}$ = 0.759 ksi. In the example case, the additional lateral bending stress at the outer edge of the top flange would be 0.966 ksi, which would exceed the MOR if there were no preexisting compressive stress in the top flange. These calculations show that the effort to straighten the girder could result in flexural cracking at the edge of the top flange if the precompression at this location is less than the difference between the two stresses, or 0.207 ksi. Table 3 reports the additional bending stresses at the extreme fiber as a result of various amounts of straightening. Total compressive stresses must also be considered.

One should note that if straightening is attempted after the end diaphragms have been placed, and rotation of the girder in the horizontal plane is prevented at the end diaphragms, the force required to straighten the same amount of sweep will increase fourfold, which in this case will be nearly 18 kip. The resulting moment, and therefore the added stresses in the girder, will double. For these reasons, cracking of the girder is more likely if straightening attempts are made after end diaphragms are placed.

When F_H is applied at the top of the girder, torsion and torsional stresses will be imposed on the bridge girders. As shown in Table 3, if the sweep correction is 1.63 in. and F_H is applied to the top flange, the imposed torsional moment will be 13.23 kip-ft and shear stresses will be in the range of 100 psi; these results are greater than the torsional restraining moments and stresses created by the

Table 3. Effects of Reducing Sweep by Applying Lateral Force F_H to Girder at Midspan

Amount of Sweep Reduction, Δf	Amount of Sweep Reduction at Midspan Δf, in.	Horizontal Force Required at Midspan F_H, kip	Moment about Weak Axis M_{yy}, kip-ft	Change in Stress at Extreme Top Fiber f_t, ksi	Change in Stress Greater than MOR ($0.24\sqrt{f'_c}$)? (Yes or No)	Torsional Restraint Required at Bearing T_A, kip-ft	Maximum Torsional Stress τ, ksi
0.0	0.00	0.00	0.00	0.000	No	0.00	0.000
0.5 f_{tol}	0.82	2.20	71.92	0.483	No	6.61	0.048
1.0 f_{tol}	1.63	4.41	143.84	0.966	Yes	13.23	0.096
1.5 f_{tol}	2.45	6.61	215.76	1.448	Yes	19.84	0.145
2.0 f_{tol}	3.26	8.82	287.69	1.931	Yes	26.45	0.193

Notes:
1. Calculations are based on a BT-72 girder with length L = 130.5 ft.
2. Sweep tolerance f_{tol} = 1.631 in. is computed from a PCI standard practice[1] where sweep tolerance is 1/8 in. per 10 ft of length.
3. Calculations assume that the horizontal force F_H is applied at the centroid of the girder cross section.
4. MOR = $0.24\sqrt{f'_c} = 0.24\sqrt{10 \text{ ksi}} = 0.759$ ksi

girder's self-weight with an initial sweep of $2f_{tol}$. Although the horizontal forces could be applied to both the top and bottom of the girders to avoid torsion, that is a jobsite decision and beyond the scope of this article.

All things considered, the computations for this example girder indicate that attempts to straighten a girder with sweep will require significant engineering analyses, planning, and the application of a force that may cause cracking in the girder being straightened. The straightening scheme would also affect the girders that are being used in the bracing system; these effects would need to be considered but have not been explored in this analysis. The owner and the engineer may wish to consider accepting a girder with out-of-tolerance sweep, particularly when, as demonstrated by the earlier discussion of this example, long-term implications for the bridge performance may not be significant. Changes to the slab formwork system may need to be considered to accommodate a girder with sweep beyond tolerance, but it is anticipated that these changes would be simpler than undertaking any straightening measures.

Summarizing the Effects of Sweep in Bridge Construction and in Service

Torsional moments, stresses, and deformations are produced for girders with sweep by the girder self-weight when it is set on the bearings. However, in the example we are discussing, the effects are relatively small, and one may therefore reasonably assume that the owner, engineer, and contractor can ignore the effects of girder sweep. The computations outlined in the discussion and summarized in the tables also clearly show that the effects of sweep do not have large impacts on the structure or the structural mechanics of the bridge during deck slab placement or after the deck slab concrete hardens and the girders become composite with the concrete bridge deck slab.

Sweep of any amount does create the need for torsional restraint at the bearings, but that restraint occurs naturally when considering the self-weight of the girder resting on its bearings. More restraint can be provided by installing bracing at girder ends.

Computations in this article demonstrate the inherent stability of girders with and without sweep. Even so, the conditions for stability must be examined for each individual case; the methods of computation provided in this article may be applied. It is the conclusion of this author that girders with sweep within tolerance can be safety erected on their bearings without any special consideration for girder sweep beyond the temporary bracing that is typically recommended at each end of each girder regardless of sweep.

This article also discusses the forces, stresses, and deformations produced when attempts to "straighten" a girder with sweep are made. Girder straightening is not recommended for girders with sweep that is within tolerance, and it may not be necessary for girders with even greater sweep. When girder straightening is desired to bring a girder's sweep within tolerance, horizontal forces are required to push or pull a girder into alignment, with bracing required against one or more adjacent girders. Three things are apparent from the computations: First, there is significant risk that the straightening attempts could cause cracking. Second, a specialty engineer must design or review the methods and details necessary for straightening to help ensure the practicality and safety of the process. Third, the straightening attempts are likely to produce torsion, torsional stresses, and lateral bending stresses in the girders that may be large enough to crack the girder(s).

It is the conclusion of this author that girders with sweep within tolerance can be safety erected on their bearings without any special consideration for girder sweep beyond the temporary bracing that is typically recommended at each end of each girder regardless of sweep.

This author has concluded that efforts to straighten a girder with sweep are most likely counterproductive to the long-term serviceability of the bridge girder and the bridge itself, and therefore recommends acceptance of girders with reasonable amounts of sweep.

Concluding Remarks

In conclusion of this three-part series, I would like to make a few additional remarks.

The causes of girder sweep are myriad and may reflect other problems or issues within the fabrication of the precast, prestressed concrete bridge girder. In those cases, the tolerance for sweep in prestressed concrete girders may play a role in the overall quality control and quality assurance programs for the production of bridge girders. These issues are discussed in Part I of this series. Additionally, the storage, handling, and transportation of prestressed concrete girders may be further complicated by girders that exhibit sweep, but the problems are manageable. These topics were covered in Parts I and II. Finally, and perhaps most importantly, the analysis described in this final article shows that constructing a bridge with a girder that exhibits a reasonable amount of sweep has little or no impact on the serviceability or strength of the bridge. Therefore, this author's conclusion is that the owner and engineer should consider allowing girders that exhibit reasonable sweep to be incorporated into a bridge. In other words, if a bridge girder with sweep beyond tolerance has been analyzed to ensure that all criteria for stability, strength, and serviceability are met, it can be transported and erected. I believe it is both expedient and reasonable for the owner and engineer to accept that girder, make no attempt to straighten it, and continue with construction.

EDITOR'S NOTE

There is an Errata for Part II published in the Fall 2019 ASPIRE included on page 3 of this issue.

References

1. Precast/Prestressed Concrete Institute (PCI). 1999. *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products*, 4th ed. (MNL-116-99). Chicago, IL: PCI.
2. PCI. 2016. *Recommended Practice for Lateral Stability of Precast, Prestressed Concrete Bridge Girders.* (CB-02-16). Chicago, IL: PCI.

Making UHPC Practical for Implementation in Precast, Pretensioned Concrete Elements

by Dr. John Lawler and Elizabeth Wagner, Wiss, Janney, Elstner Associates Inc., and Dr. Maher Tadros, e.construct USA LLC

Ultra-high-performance concrete (UHPC) is a class of concrete that relies on a highly refined microstructure and fiber reinforcement to achieve superior performance characteristics, including high compressive strength, postcracking tensile strength and ductility, and exceptionally long-term durability in aggressive environments. The constituent materials of UHPC typically include cement; silica fume; sand; a fine-grained supplemental material such as limestone powder, fly ash, ground silica, or slag cement; and a potent high-range water-reducing admixture (HRWRA) that supports production at water-binder ratios of 0.2 or less. In recognition of the great potential of UHPC to improve the performance of concrete structures, particularly in precast concrete bridge and building applications, the Precast/Prestressed Concrete Institute (PCI) has funded a research project, "Implementation of Ultra-High-Performance Concrete in Long-Span Precast Pretensioned Elements for Concrete Buildings and Bridges," to make UHPC implementation practical for North American precasters. This article presents guidance produced to date by this project.

Advantages and Challenges Associated with UHPC

Because of its exceptional tensile performance, which is sufficiently robust to be relied on for design purposes, and its enhanced durability, UHPC has the potential to greatly extend the capabilities of concrete construction. For example, by using optimized designs member weights can be reduced by 50% compared to conventional concrete without reducing capacities. Also, even with severe marine or deicing salt exposures, service lives of 200 years or more can be reasonably anticipated.

Despite this highly desirable performance, applications of UHPC in the United States have been limited, and designers and owners have been reluctant to accept and use this material. The high cost of proprietary UHPC materials, technical challenges associated with production, and a shortage of national design guidelines have all contributed to limited implementation.

Phase I Outcomes of PCI-Funded UHPC Research

The precast concrete industry is particularly well suited to support implementation of UHPC because of the greater materials handling, concrete production, and construction quality control possible within a plant setting. There is also the potential for efficient production of optimized sections that make effective use of the relatively high-cost UHPC material. Therefore, the PCI-funded research project seeks to foster more widespread implementation of UHPC in precast concrete applications through the publication of guidelines for developing and producing cost-effective UHPC mixtures, a guide specification for UHPC materials qualification and acceptance, and structural design guidelines. The overall project and design guideline development are being led by e.construct USA of Omaha, Neb., and the materials aspects of the project are led by Wiss, Janney, Elstner Associates of Northbrook, Ill.

A first goal of this project is to develop methods for implementing cost-effective UHPC mixtures made with locally available materials and existing production facilities at precast concrete plants throughout the United States. A second goal is to develop design methods and novel designs for optimized long-span structural members for bridges and buildings. The first phase of this ongoing PCI project was recently completed, and a Phase I report[1] about the significant progress made toward achieving these goals was published in February 2020. The report includes draft guidelines for production, a draft materials guide specification, and proposed guidelines for design with examples. Each of these parts of the report are discussed in the following sections.

Guidelines for Production

The draft guidelines for production present an overview of UHPC production specific to long-span precast, pretensioned concrete structural elements. Topics include recommendations for raw materials selection; development of UHPC mixture designs; batching, handling, and placement of materials; and methods for evaluating the performance of UHPC materials for mixture qualification and quality assurance.

The draft production guidelines were validated through UHPC production trials in precast plants in the United States and Canada. UHPC mixtures were developed for five precasters using the efficient mix-development approach presented in the guidelines, which is based on proportioning materials to achieve an optimized packing of particles. The mixtures developed for this project were largely based on materials already used in production at each plant, with the exception that the typical concrete sands were replaced with finer "masonry" sands available from the precasters' local concrete aggregate suppliers. Some precasters also chose to incorporate HRWRAs that differed from their current typical products.

The mixture development process included particle-size analysis of the constituent materials, theoretical determination of optimum proportions based on simplified

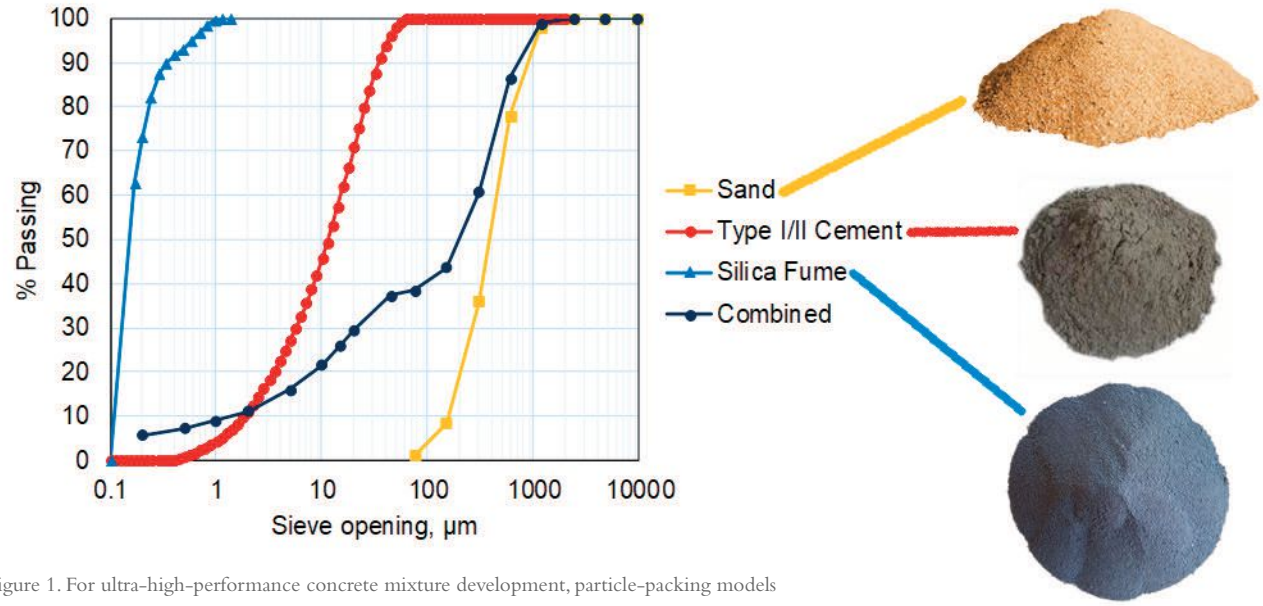


Figure 1. For ultra-high-performance concrete mixture development, particle-packing models considering the particle-size distribution of the constituent materials are used to determine optimized combinations of raw materials. Figure: Wiss, Janney, Elstner Associates Inc.

particle-packing models (**Fig. 1**), small-scale laboratory batches to minimize water content and admixture dosage, and finally, scaled-up testing using full-sized batches (**Fig. 2**) to verify that the UHPC material performance met expectations. **Figure 3** shows a four-point bending test conducted according to ASTM C1609[2] to determine flexural strength, which was employed as the most practical approach for evaluating tensile performance. UHPC has a minimum required tensile strength (see the Concrete Bridge Technology article in the Winter 2020 issue of *ASPIRE*®).

The production and handling guidelines were subsequently validated through production trials at each precast plant. Structural elements of various configurations were produced and used to refine concrete handling, placement, and finishing methods. **Figure 4** shows production at one participating precaster.

Materials Guide Specification

The draft materials guide specification is intended to be used by engineers and owners as a basis for the preparation of materials specifications for building and transportation structures that align with the structural design of precast, pretensioned concrete elements. This draft guide specification addresses the materials and production of structural precast, prestressed UHPC elements and provides requirements for materials qualification (submittals), raw materials, mixture proportioning and documentation, qualification and routine acceptance testing, and production, including storage and handling, mixing, transport and placement, finishing, curing, inspection, and testing.

Proposed Guidelines for Design

The proposed design guidelines provide a rational basis for design parameters such as flexure, vertical shear, interface shear, and punching shear, while considering other factors that will influence member performance, including time-dependent effects, bond, and end-zone bursting.

Figure 2. Mixing ultra-high-performance concrete in an existing horizontal twin-shaft mixer at a precaster's batch plant. From left, addition of water after blending of dry material, achievement of self-consolidating consistency, and addition of brass-coated steel fibers. Photos: Wiss, Janney, Elstner Associates Inc.

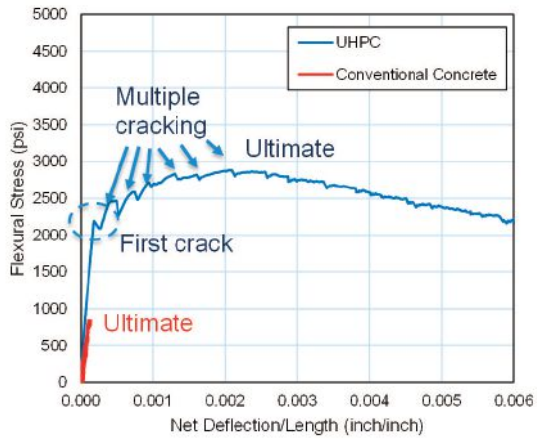


Figure 3. Verification of tensile properties of ultra-high-performance concrete through flexural testing. Photo and Figure: Wiss, Janney, Elstner Associates Inc.

Figure 4. Construction of 60-ft-long voided box beam using an ultra-high-performance concrete mixture developed using the approach outlined in the draft guidelines for production included in the Phase I report of the PCI-funded study. Photo: Wiss, Janney, Elstner Associates Inc.

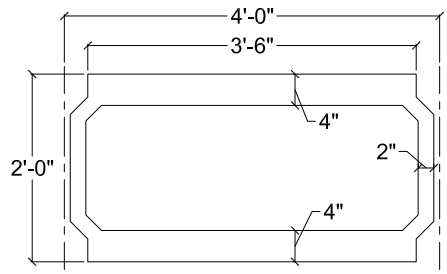


Figure 5. Optimized concrete hollow box beam cross section developed as a part of the PCI-funded study to take advantage of the tensile performance of ultra-high-performance concrete. Figure: e.construct USA LLC.

Chapters 4 through 10 of the Phase I report provide the basis of the design guidelines, which are then demonstrated in design examples. The report also covers product development concepts.

As part of Phase II of the project, several full-scale bridge and building elements have been produced and tested based on the designs developed in Phase I. **Figure 5** shows an example of an optimized box beam cross section intended for bridge construction. The results of the testing to date are promising, as the products have supported loads as much as double the required design loads. Phase II of the PCI project is ongoing and will include additional structural testing of components and full-sized members. This testing program is intended to verify, refine, and confirm the draft design guidelines presented in the Phase I report.

Conclusion

UHPC has the potential to significantly advance the capabilities of precast concrete for bridge and building construction because of the durability and the structurally reliable tensile strength and ductility of the material. It is hoped that the guidance now available for materials development, when combined with the structural design guidelines currently being finalized, will enable practical implementation of UHPC nationwide so that the potential of this material can begin to be realized.

References

1. e.construct USA. 2020. "Implementation of Ultra-High-Performance Concrete in Long-Span Precast Pretensioned Elements for Concrete Buildings and Bridges: Phase I Report." Precast/Prestressed Concrete Institute. https://www.pci.org/PCI_Docs/Members_Only/Research%20Reports/Material/IMPLEM.pdf.
2. ASTM International. 2019. *ASTM C1609/C1609M-19a: Standard Test Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam With Third-Point Loading)*. West Conshohocken, PA: ASTM International.

Acknowledgments

The authors would like to recognize and thank the other members of the project team that provided essential assistance in the completion of Phase I and continue to work on the ongoing activities of this project: the University of Nebraska-Lincoln, North Carolina State University, and Ohio State University, as well as the participating precasters: Standard Concrete Products Inc., Tampa, Fla.; Coreslab Structures (Omaha) Inc., Omaha, Neb.; Metromont Corp., Greenville, S.C.; Tindall Corp., Spartanburg, S.C.; Concrete Technology Corporation, Tacoma, Wash.; and FACCA Inc., Windsor, Ontario.

Dr. John Lawler is a principal and Elizabeth Wagner is an associate III, both with Wiss, Janney, Elstner Associates Inc. in Northbrook, Ill. Dr. Maher Tadros is the founder and managing partner of e.construct USA LLC in Omaha, Neb.

Using Embedded Corbels for the PHX Sky Train Project

by Colin Van Kampen, Modjeski and Masters Inc.

Pier girder segment after arriving on site. The corbels projecting from the end diaphragm with bearing plates on top are lower in the section to support the corbels for the drop-in segment. Post-tensioning anchors are located near the top of the diaphragm for first-stage post-tensioning tendons. Smaller, plant-installed post-tensioning tendons are located at the top outer edges of the section. Photo: Modjeski and Masters Inc.

Drop-in girder segment (girder segment 3) arriving on site. The corbels projecting from the end diaphragm with bearing plates on bottom are higher in the section so they can rest on the corbels in the pier segments. Shear keys and embedded couplers for installing mild reinforcement for the splice are visible on the end of the girder. Photo: Modjeski and Masters Inc.

The PHX Sky Train Stage 2 Extension under construction at Phoenix Sky Harbor International Airport in Arizona will be a 2.2-mile extension of the automated train system from Terminal 3 west to the Rental Car Center (for additional details, see the Project article on p. 6). The construction schedule's critical path required continuous-span construction within the airport footprint while minimizing the project's impact on passengers and airport operations. Because minimal gate closures were permitted for precast concrete erection, the project needed a fast and effective construction method for the five-span post-tensioned (PT) unit. Further complicating construction of the PT unit was the existing terminal beneath span two of the unit, which had to remain open throughout bridge construction. The use of embedded corbels helped facilitate this challenging project and limited the impact of construction on airport operations.

Benefits of Embedded Corbels

In most instances, erection of spliced PT concrete structures involves constructing temporary supports to support the segments at splice locations. However, use of those typical means of support was complicated or prohibited within the apron of the active airport. The 676-ft-long, five-span spliced unit was composed of eight 78-in.-deep U-girder segments of varying lengths in three parallel girder lines, each with seven splices. Five of the splices were at locations that could accommodate traditional temporary supports and would not impact airport operations. The other two splice locations presented extraordinary obstacles to the construction of the unit's superstructure.

The critical stage of the PT unit's construction did not occur within its longest span, the 198 ft 2⅜ in. taxiway span. The taxiway span was easily accessible and far enough from existing airport infrastructure that it presented no unusual challenges and was supported using traditional temporary supports. Instead, the critical span was the 163 ft 6⅛ in. span over Terminal 2. Within this critical span, girder lines passed over the passenger security checkpoint and within close proximity to an aircraft jetway. An additional complication was a service road ramp; its presence meant the construction team had no access to level ground on which to build traditional temporary works.

Embedded corbels combined with staged PT construction proved the optimal solution and allowed clearance of the terminal without temporary works. The use of embedded corbels also eliminated any need for additional terminal closures to dismantle temporary works or remove strongbacks. Additionally, the embedded corbels, located on the drop-in segment and the previously erected cantilevered segments, were completely encapsulated within closure pours, which maintained the aesthetics of the superstructure.

Construction Sequence

Initially, the superstructure was designed using a single stage of PT construction. When the construction method was

Overhead view of the construction site before drop-in segments were erected over Terminal 2. Splices and pier diaphragms are visible. Photo: Google.

changed to use embedded corbels to "float" the critical segment, several additional stages of construction and an additional stage of post-tensioning were required. Construction phasing for the complete five-span PT unit was as follows:

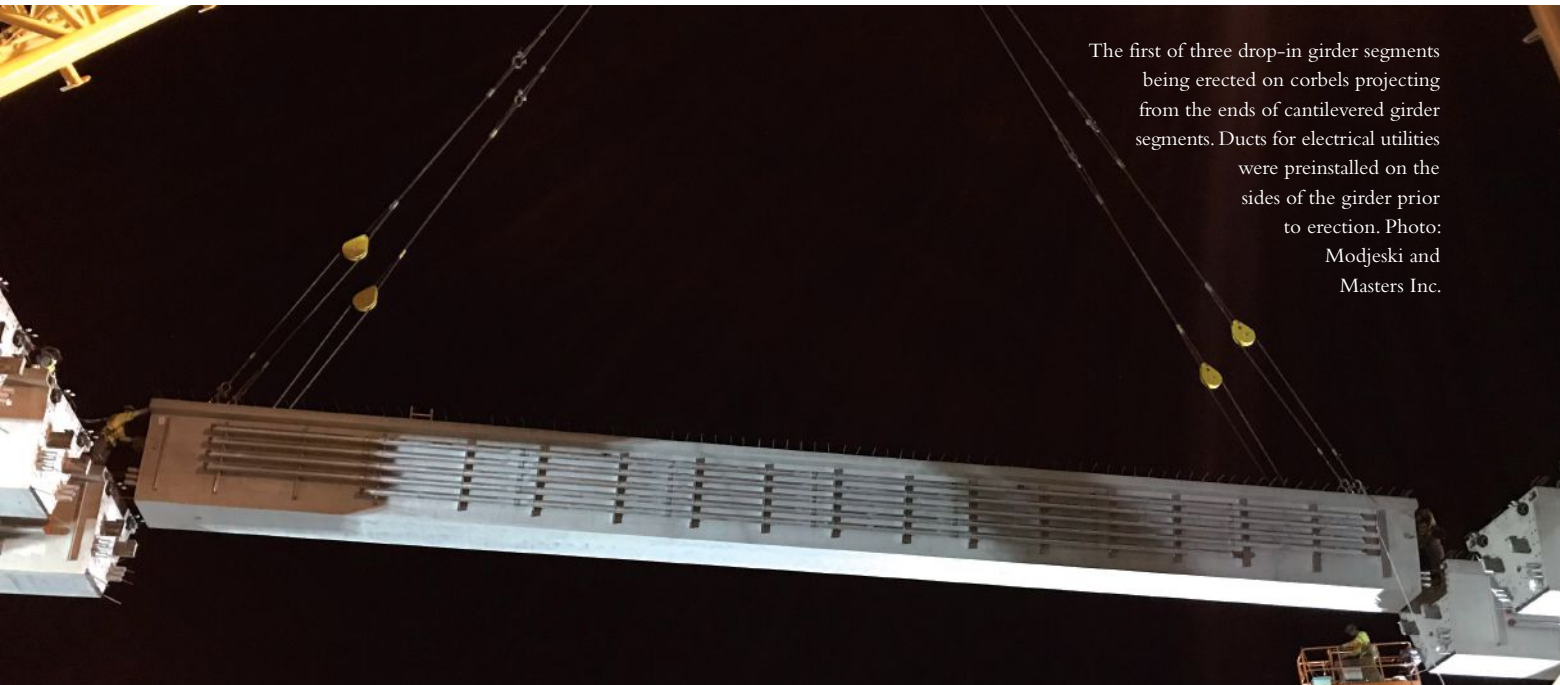
- Stage 1: Girder segments 1, 2, 4, 5, 6, 7, and 8 were erected on piers and falsework towers.
- Stage 2: PT ducts were spliced, and closure pours at splices and pier diaphragms were cast.
- Stage 3: Two PT tendons in each girder line were tensioned and grouted to form two subunits: girder segments 1 and 2 formed subunit 1, and girder segments 4 through 8 made up subunit 2.
- Stage 4: The drop-in segments (girder segment 3) were erected onto embedded corbels projecting from cantilevered ends of girder segments 2 and 4, and corbel connection bolts were installed.
- Stage 5: PT ducts were spliced, and closure pours at the remaining two splices were cast.
- Stage 6: Six PT tendons in each girder line that ran the full length of the unit were tensioned and grouted, completing the 676-ft-long continuous PT unit.

Embedded Corbel Design

The embedded corbels were designed to resist the dead load, construction live load, and wind load to which the girders would be subjected during stages 4 and 5. For each connection, casting the closure pour around the corbel connection provided additional stability and capacity to support temporary and permanent loads, although no allowance for design live load was included in the corbel design.

The *PCI Design Handbook*[1] has comprehensive information on the design of embedded steel corbels for concrete structures. This design methodology was adopted on this project to check the capacity of the steel corbels as well as the concrete capacity of the cross section. For these design checks, the lower resistance factors were adopted from the *PCI Design Handbook* while the higher load factors of American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[2] were used for load combinations and load factors. This resulted in a conservative design. The shear block check and design of the additional reinforcement were also based on AASHTO LRFD specifications, as well as the allowable concrete compression stresses.

The resulting corbel consisted of a 3-ft 9-in.-long hollow structural section (HSS) tube 12 by 4 by ⅝ in. with ⅜ by 8 by 6 in. bearing plates containing either slots or holes for the ⅞-in.-diameter A325 bolted connection. The HSS tube was pierced with no. 5 U-shaped reinforcing bars and filled solid with concrete having a compressive strength of 7000 psi. Fully assembled and filled with concrete, each corbel weighed approximately 365 lb. Additional



The first of three drop-in girder segments being erected on corbels projecting from the ends of cantilevered girder segments. Ducts for electrical utilities were preinstalled on the sides of the girder prior to erection. Photo: Modjeski and Masters Inc.

Embedded corbels projecting from drop-in girder segments, here shown at right, resting on corbels projecting from cantilevered ends of girder segments on left. Photo: Hensel Phelps.

reinforcement consisted of no. 4 closed-loop stirrups that surrounded the embedded portion of the HSS tube.

Incorporation of Embedded Corbels and Precast Concrete

Using a U-girder section for this project provided great benefits, including ample space for incorporating corbels into the ends of the precast concrete girders. While embedded corbels have been used in precast, prestressed concrete bulb-tee girders in many previous instances with great success, their incorporation can require additional forming at girder ends. The availability of space within the U-girder cross section allowed corbels to be incorporated without additional forming and also allowed a wider spacing of the corbels, resulting in better elevation control and lateral load resistance.

To create the end block that the corbels required, the interior form was simply held back from the girder end, creating a solid section capable of housing the corbel and associated reinforcement. This was a twofold solution because the staged PT sequence also required PT anchorages to be placed in these end blocks.

Projection of the corbel from the end face of the girder did require modification of the end form. Strict quality control and placement tolerances were adopted during casting to control corbel projection length and location, as well as both lateral and horizontal projection angles.

The addition of the solid end section posed a challenge to girder stability during erection. The additional weight of the solid ends at the end of the cantilevered portion of the pier girder segments caused additional overturning that had to be properly addressed to ensure adequate safety factors for stability during construction. The solution incorporated temporary ballast weight and hold-downs at the pier girder segments' temporary end supports.

Conclusion

For the PHX Sky Train Extension, the combination of staged PT spliced concrete girder technology and embedded corbels provided a streamlined solution to challenges associated with the restrictive site conditions. With the use of these technologies, the critical construction path activities could continue uninterrupted without substantially disrupting the operations of a major airport. Using embedded corbels in the ends of the girder segments allowed three 187,000-lb drop-in concrete girder segments to "float" over existing airport infrastructure, supported at the splice locations by the previouly erected girder segments. This eliminated the need for both external support hardware and secondary crane mobilizations to dismantle temporary works near active gate facilities, thus simplifying construction while also minimizing terminal closures during construction. This complex embedded corbel construction strategy was vital to the successful completion of the PHX Sky Train Extension superstructure.

References

1. Precast/Prestressed Concrete Institute (PCI). 2010. *PCI Design Handbook*, 7th ed. Chicago, IL: PCI.
2. American Association of State Highway and Transportation Officials (AASHTO). 2017. *AASHTO LRFD Bridge Design Specifications*, 8th ed. Washington, DC: AASHTO.

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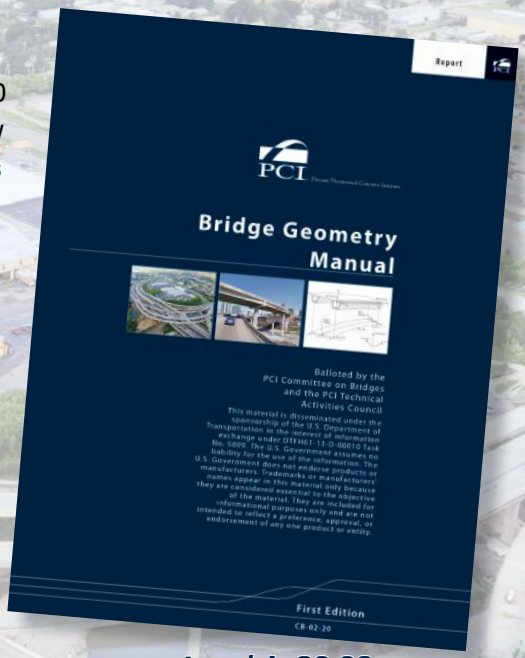
Bridge Geometry Manual

FREE PDF (CB-02-20)

The *Bridge Geometry Manual* has been developed as a resource for bridge engineers and CAD technicians. In nine chapters, the manual presents the basics of roadway geometry and many of the calculations required to define the geometry and associated dimensions of bridges. This manual and course materials are not linked to any software tool. The first five chapters are dedicated to the fundamental tools used to establish bridge geometry and the resulting dimensions of bridges. The vector-based approach to locating the north and east coordinates of a point defined by a horizontal alignment is then used to define the geometry of bridges. This manual includes the bridge geometry developed for straight bridges and curved bridges. The geometry of curved bridges using both straight, chorded girders and curved girders is presented.



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Guide Document for the Design of Curved, Spliced Precast Concrete U-Beam Bridges

FREE PDF (CB-03-20)

The *Guide Document for the Design of Curved, Spliced Precast Concrete U-Beam Bridges* has been developed as a resource for bridge engineers. In nine chapters, the guide documents the advancement of this bridge technology. This technology, which originated and progressed initially in Colorado over approximately 20 years, has evolved through the collaboration of designers, contractors, and owners. Much of the current technology is in its second or third generation. Agencies and builders have shown interest in replication of this bridge technology in several areas of the United States. However, there are certain areas of practice that have not been quantified. This has made it difficult for owners and the design community to fully embrace the technical solutions needed to design, construct, deliver, and maintain curved, spliced U-beam bridge systems. This document addresses those practices.

www.pci.org/cb-03-20

PCI Precast/Prestressed Concrete Institute

When Concrete Isn't Concrete: Preserving the Dry Canyon Bridge

by Rebecca Burrow, Oregon Department of Transportation

In 2017, nearly a century after it was constructed, the Dry Canyon Bridge on the scenic Historic Columbia River Highway underwent its first major rehabilitation, which included repairs to low-strength concrete, realkalization of the superstructure, application of a cementitious render, and an experimental deck overlay. With the completion of these repairs, it is expected that the bridge will have another 100 years of service life under the Oregon sun.

Concrete Mysteries

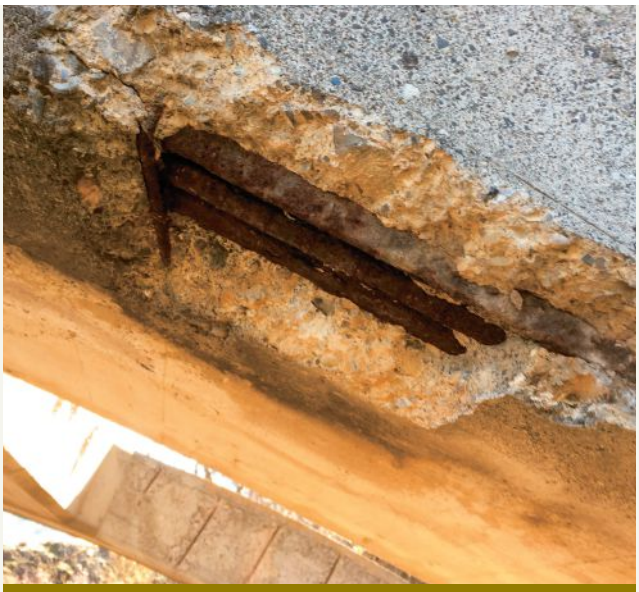
Constructed over a deep canyon in the basalt walls of the Columbia River Gorge, the Dry Canyon Bridge consists of a single-span 75-ft-long reinforced concrete deck arch with two hidden slab spans making up the approaches. Because bid prices for the original 1920–1921 construction project were high, the engineers chose to use state maintenance workers for construction. Their lack of bridge construction experience led to some quality issues but also resulted in a wealth of records beyond the usual design documents. Of historical interest, these records include details on the construction camp, such as food orders. More importantly for engineers, they include critical documentation of the construction materials and material tests.

The bridge has remained on a low-volume road, is in a dry climate, and has required little use of deicing salts. These beneficial conditions significantly delayed deterioration until around 2015, when the bridge inspector started noticing active spalling of the concrete.

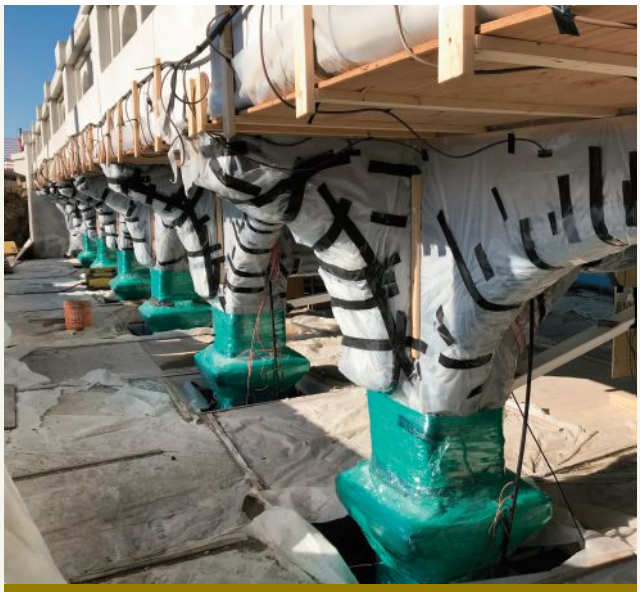
Initially, a simple bridge preservation project was planned. However, during preliminary design on the preservation project, closer examination of the arch revealed that the deterioration was much more significant than originally thought, and it could not be explained by the standard mechanisms of concrete breakdown, chloride intrusion, and freeze-thaw cycles. Following that discovery, an examination of the original records showed that the concrete used to construct the bridge was abnormal, and tests of the original concrete indicated that it had a compressive strength well below normal. Instead of the specified 2200-psi compressive strength, tests came back as low as 730 psi.

The below-normal strength was likely due to a bad batch of cement with added lime, but no additional testing was done at the time of construction. Instead, the engineer, Conde B.

The Dry Canyon Bridge was constructed over the winter of 1920–1921, before highway construction had reached this remote location in the Columbia River Gorge. The timber for the falsework and formwork was brought in by mule train. Photo: Oregon Department of Transportation Bridge Engineering Section.



Although concrete spalling and corrosion of the reinforcement were widespread on the concrete arch, the reinforcement maintained its integrity, leading to the decision to rehabilitate rather than replace the bridge. Photo: Rebecca Burrow.



The realkalization system involved wrapping the bridge with insulation matting to store the electrolyte, folding titanium mesh over that, and containing the system with a plastic covering. Photo: Rebecca Burrow.

McCullough, verified that the arch would be able to carry the expected loading with this lower-strength concrete, and construction continued.

Although concerning, this low strength did not explain the sudden spalling nearly 100 years after construction. To solve that mystery, a concrete core was sent for petrographic examination. The results showed that the concrete had undergone carbonation at least 2 in. deep, and later tests found that carbonation in some locations was more than 3 in. deep. Carbonation of concrete is a chemical reaction whereby the calcium hydroxide ($Ca(OH)_2$) in the concrete reacts with atmospheric carbon dioxide to form the more brittle calcium carbonate ($CaCO_3$). In reinforced concrete, this means that embedded steel is no longer protected from corrosion by a high-pH environment.

Though the process of carbonation occurs in all concrete, it is typically slow enough that other deterioration mechanisms happen first. However, in the case of the Dry Canyon Bridge, the low-strength concrete was more porous, allowing the reaction to occur deeper into the concrete. Once the reaction reached the reinforcing steel, which typically had 2 in. of cover, the steel began to corrode, resulting in widespread spalling. There are three long-term solutions to this problem: (a) replace the bridge; (b) remove and replace all of the carbonated concrete; or (c) realkalization.

Spalling of the concrete was widespread on the arch, but the underlying reinforcement still had sufficient integrity despite surface corrosion. This was a factor in the decision to rehabilitate rather than replace the bridge

Realkalization

Realkalization is an electrochemical treatment where an electric field is created between the reinforcing bars and an anode mesh that is temporarily applied on the surface of the concrete. The anode is surrounded by an alkaline electrolyte, typically sodium or potassium carbonate, which is pulled into the concrete surface by the electrical current. Over the course of a week, the process reverses the carbonation, re-passivates the steel, and densifies the concrete, slowing the carbonation process for the future.

A life-cycle cost analysis determined that realkalization was the preferred alternative for the Dry Canyon Bridge. As a first step in the process, the already spalled concrete had to be patched. Given the extremely soft concrete, the decision was made to use masonry mortar, Type S, as the patch material for compatibility. Next, the realkalization system was applied, which involved wrapping the bridge with insulation matting to store the electrolyte, folding titanium mesh over that, and containing the system with a plastic covering. Gravity flow was used to supply the

CONDE B. MCCULLOUGH

In 1919, Conde Balcom McCullough (1887–1946) became the state bridge engineer for Oregon, where he was responsible for the design of many beautiful and innovative bridges. He is best known for his large coastal bridges, but many of his smaller bridges, such as the Dry Canyon Arch, are also noteworthy for their economical yet aesthetically pleasing designs. Prior to joining the highway department, McCullough was the head of the engineering department at the Oregon Agricultural College (now Oregon State University); once he left, he recruited many students from there. These former students often became the resident engineers at his construction projects.

After being coated with a cementitious render and silane sealer, the rehabilitated bridge looks almost brand new. Photo: Rebecca Burrow.

electrolyte to the entire bridge. Because of the historic and scenic requirements of the site, the bridge was also coated with a cementitious render and silane sealer following the realkalization to restore its appearance. Overall, the work on the bridge took three months and never closed the roadway for longer than 20 minutes.

Another Surprise

After the realkalization work, the bridge had one more surprise in store for the project team. The intended scope of work had included removing the asphalt pavement, installing a waterproof membrane, and repaving. When the asphalt was removed, it became clear that some previous paving project had addressed grade issues at the site by grinding concrete off the bridge deck. The supposedly 12-in.-thick concrete approach slabs were actually less than 10 in. thick. Left in that condition, the bridge would have had to be restricted to small vehicles.

Instead, the decision was made to add a structural overlay to the bridge. Given the width of the structure and the length of the detour, it was not considered feasible to use Oregon's traditional microsilica overlay, which would have required closing the roadway for up to seven days. As an alternative, the contractor proposed the use of a latex-modified concrete, which had not been used previously in Oregon. This method allowed the entire overlay to be constructed in a single day. Despite some shrinkage cracks, the bridge is now able to carry normal legal loads with no restrictions.

Conclusion

With the overlay completed and the site cleaned up, the rehabilitation project was complete. What had seemed to be a simple preservation project had turned into much more, with new methods and materials helping to successfully restore the bridge. The result is a beautiful bridge that will complement its scenic location for generations to come.

Rebecca Burrow is a bridge standards engineer with the Oregon Department of Transportation in Salem.

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

IN THIS ISSUE

https://www.skyharbor.com/About/Development/PHXSkyTrainstage2
This is a link to the Phoenix Sky Harbor Airport website for the PHX Sky Train Stage 2 project, which is the subject of Project and Concrete Bridge Technology articles on pages xx and 34.

https://ascelibrary.org/doi/abs/10.1061/%28ASCE%29BE.1943-5592.0001551
This is a link to purchase *Construction and Field Evaluation of Electrically Isolated Tendons in a Prestressed Concrete Spliced Girder Bridge*. The system was installed in the Coplay-Northampton Bridge, featured in a Project article on page 20.

https://www.ocapa.net/assets/EIC_2019_folder/2019_EIC_Projects_PDFs/US30%20%20Mosier%20Dry%20Canyon%20%20Chenoweth%20Creek%20Bridges.pdf
This is a link to a presentation on the rehabilitation of the Dry Canyon Bridge, which is featured in a Concrete Bridge Preservation article on page 38.

https://www.wesavestructures.info
The Concrete Preservation Alliance website provides information on realkalization, which is discussed in the Concrete Bridge Preservation article on page 38.

https://www.fhwa.dot.gov/hfl/projects/ut_grs-ibs_i84_saltlakecity.pdf
This is a link to *Utah Demonstration Project: Geosynthetic Reinforced Soil Integrated Bridge System on I-84 near Salt Lake City—Draft Report*. The project was the first use of a geosynthetic reinforced soil-integrated bridge system (GRS-IBS) on a U.S. interstate. See the State article on page 42.

https://www.fhwa.dot.gov/publications/research/infrastructure/structures/bridge/17080/17080.pdf
This is a link to the FHWA report HRT-17-080 *Design and Construction Guidelines for Geosynthetic Reinforced Soil Abutments and Integrated Bridge Systems*. GRS-IBS is discussed in the State article on page 42.

https://www.youtube.com/watch?v=N3j8ATiqcAs&index=1&list=PL5_sm9g9d4T3ymcxjfVEdYnd2-Hj-Fozl&t=0s
This is a link to *Replaceable Grouted External Post-Tensioned Tendons*, a video by the Federal Highway Administration. The topic is the subject of the FHWA article on page 46.

https://store.transportation.org/item/collectiondetail/202
This is a link to information on the new 9th edition *AASHTO LRFD Bridge Design Specifications*. The revised column tie spacing requirement is discussed in the LRFD article on page 53.

STATE

Utah

by Cheryl Hersh Simmons, Utah Department of Transportation

The Utah Department of Transportation (UDOT) is committed to safety and optimizing mobility with an emphasis on quality, innovation, and collaboration. This commitment leads Utah to pursue and adopt advanced technologies, improved processes, alternative contracting methods, and other innovations that improve the design and construction of bridges.

Utah's bridge inventory consists of 3001 structures, including 1905 state-owned bridges and culverts. More than two-thirds of the bridges have concrete superstructures. To manage this inventory, UDOT is using accelerated bridge construction (ABC) techniques, advanced technologies and materials, and, most recently, digital delivery for exchange of data and information to improve efficiencies and meet project goals. Furthermore, open dialogue with the industry enables UDOT to document lessons learned, improve details, and refine project schedules. Four projects showcase how using these strategies with concrete structures have played a role in UDOT's vision to "Keep Utah Moving."

Pioneer Crossing over Interstate 15

The Pioneer Crossing project includes a new six-mile east-west connector from American Fork Main Street through Lehi to Redwood Road in Saratoga Springs and features a new diverging diamond interchange (DDI) with twin two-span prestressed concrete girder structures.

As part of the 2010 design-build project to replace the existing Interstate 15 (I-15) diamond interchange with the DDI, the contractor and designer chose to use self-propelled modular transporters (SPMTs) to reduce on-site construction time and traffic impacts and improve work-zone safety, as well as material quality and durability. The four spans of the twin two-span structures were constructed on temporary falsework supported on large concrete spread footings in adjacent staging areas less than ¼ mile from the bridge site. Each span had a 53-degree skew, with 94.5-in.-deep prestressed concrete girders spaced at 7.75 ft and an 8.5-in.-thick cast-in-place concrete deck. The 191-ft-long, 69-ft-wide single-span units weighed 2300 tons. At the time they were constructed, they were the longest and heaviest multigirder spans moved with SPMTs in the United States.

It took four nights to move the four spans. Two lines of SPMTs supported each span at its ends, and special tower-stand jacks raised each span off the temporary supports and lowered it onto the new abutments. For each structure, the two spans were positioned and then connected with a closure pour over the interior support. The decks were designed with additional reinforcement to account for the temporary tensile stresses during the move. To help minimize temporary stresses and accommodate live load continuity, the end diaphragms and the last 10 ft at each end of the deck were cast after the spans were moved.

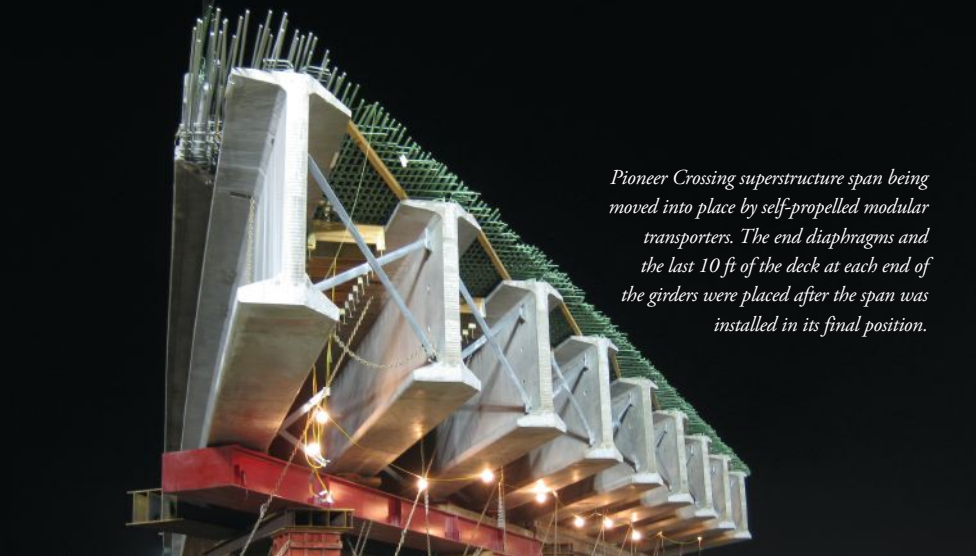
Aerial view of the Pioneer Crossing project and the diverging diamond interchange with twin two-span prestressed concrete girder structures. The spans were constructed nearby and moved into place using self-propelled modular transporters. All Photos and Figures: Utah Department of Transportation.

Interstate 84 over Echo Frontage Road

The 2013 design-bid-build project to replace twin bridges on Interstate 84 (I-84) over Echo Frontage Road in Summit County (50 miles east of Salt Lake City) involved multiple innovative construction techniques. Prefabricated bridge elements and systems along with a lateral bridge slide and a geosynthetic reinforced soil-integrated bridge system (GRS-IBS) for the abutments were chosen as the optimal solution to meet project goals. This was the first GRS-IBS project in Utah and the first GRS-IBS project on an interstate in the United States.

Seismic activity is a concern in Utah, and designers carefully accounted for seismic design factors. The project's peak horizontal ground acceleration design value was 0.25g. To assess the seismic design of the bearing capacity, results from the National Cooperative Highway Research Program (NCHRP) Report 556 were consulted.[1] The NCHRP project conducted full-scale shake-table tests at a loading level of 1.0g; these tests indicated that the GRS-IBS bridges would be able to withstand high-magnitude earthquakes.

Construction proceeded in three phases. First, the new eastbound bridge was constructed in the median of the existing I-84 alignment and temporarily used for I-84 westbound traffic while the I-84 westbound bridge was replaced. Next,



Pioneer Crossing superstructure span being moved into place by self-propelled modular transporters. The end diaphragms and the last 10 ft of the deck at each end of the girders were placed after the span was installed in its final position.

traffic was returned to the new westbound bridge, and the bridge in the median was used for eastbound traffic while the existing eastbound bridge was demolished and reconstructed using GRS-IBS. Finally, during a single overnight closure (27 hours) the median superstructure was transversely slid to its permanent geosynthetic reinforced soil (GRS) abutments. The roadway approaches were then completed with asphalt overlay and required tie-ins.

Construction of each GRS layer involved three major steps: placing a row of modular block facing units with fiberglass pins for vertical alignment; placing and compacting a layer of granular fill; and laying a sheet of geosynthetic reinforcement. After the first GRS layer was constructed over a leveling pad, additional layers were added until the required abutment height was reached. A cast-in-place footing (also referred to as a bearing sill) was then formed, placed, and cured directly on the GRS abutments. Each bearing sill was 2 ft 6 in. high and 4 ft 4 in. wide.

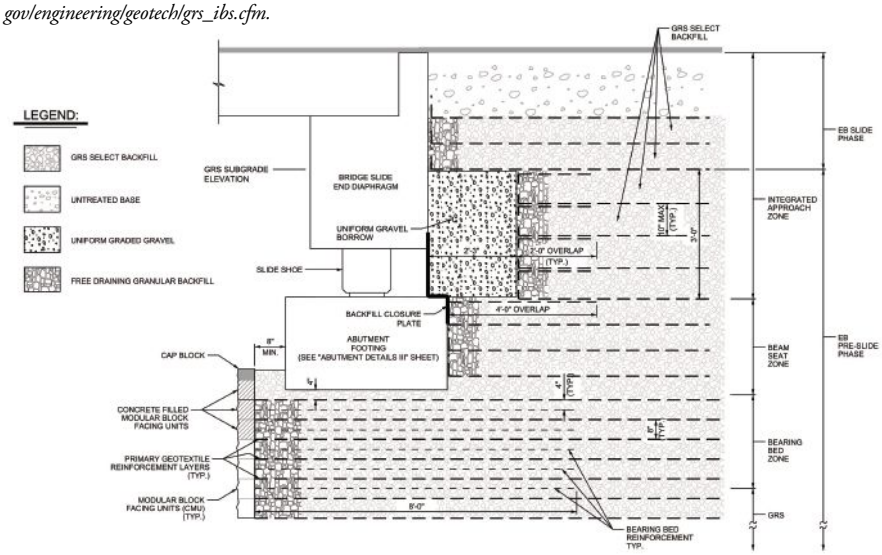
To facilitate sliding the bridge, the median and eastbound abutment designs were modified to include a diaphragm that would slide over the bearing sill. For additional support a clean, compacted gravel zone—1 ft 4 in. wide and wrapped in geotextile fabric—was provided next to the bearing sill.

The superstructure/deck system consisted of a precast concrete bridge slide end diaphragm and 58-ft-span voided-slab girders with no approach slabs. This system provided a lighter load for the GRS abutments and made the superstructure easier to slide, allowing the slide tolerances to be relaxed. Stainless steel mild reinforcement was used in the pretensioned concrete beams, parapet, abutments, and end diaphragm.

As a design contingency, slide shoes were prescribed for use during the slide process. Slide shoes clad with stainless steel at the bottom, with access between the shoes, provided a cost-effective option for lifting the bridge and accessing the bearings if there were a need to level surfaces.

The ABC techniques used in this project substantially reduced its impact on highway users. Using traditional methods, the roadway

Beam seat and integrated approach detail for the geosynthetic reinforced soil-integrated bridge system (GRS-IBS) was used for the abutments of the Interstate 84 over Echo Frontage Road structure. A full section of this GRS-IBS abutment can be found on the ASPIRE® *website: www.aspirebridge.org. See also the FHWA website: www.fhwa.dot.gov/engineering/geotech/grs_ibs.cfm.*



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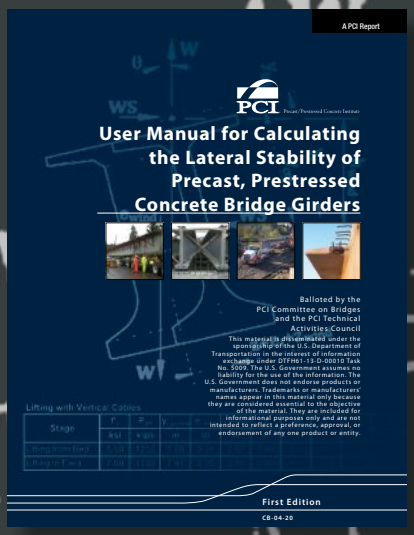


User Manual for Calculating the Lateral Stability of Precast, Prestressed Concrete Bridge Girders FREE PDF (CB-04-20)

This document, *User Manual for Calculating the Lateral Stability of Precast, Prestressed Concrete Bridge Girders*, PCI Publication CB-04-20, provides context and instructions for the use of the 2019 version of the Microsoft Excel workbook to analyze lateral stability of precast, prestressed concrete bridge products. The free distribution of this publication includes a simple method to record contact information for the persons who receive the workbook program so that they can be notified of updates or revisions when necessary. There is no cost for downloading the program.

This product works directly with the PCI document entitled *Recommended Practice for Lateral Stability of Precast, Prestressed Concrete Bridge Girders*, PCI publication CB-02-16, which is referenced in the *AASHTO LRFD Bridge Design Specifications*. To promote broader use of the example template, PCI developed a concatenated Microsoft Excel spreadsheet program where users may customize inputs for specific girder products.

www.pci.org/cb-04-20

PCI Precast/Prestressed Concrete Institute

Placement of a voided-slab girder on the precast concrete diaphragm for the Interstate 84 Bridge over Echo Frontage Road. All reinforcement in the bridge is stainless steel.

Neoprene bearing pads on the bearing sill of the Interstate 84 Bridge over Echo Frontage Road before the slide. The reinforced layers of fill in the GRS-IBS abutment are visible on the right.

would have been closed for 194 days, whereas the GRS method required 125 days of construction and closed the roadway for just 34 hours (7 hours for the removal of overhead sign structures and 27 hours during the bridge slide).

Interstate 15 Northbound/ 10600 South Interchange

The goals for the I-15 Northbound/10600 South Interchange project that was constructed in 2017 were to improve safety and reduce congestion on one of Utah's busiest interchanges by eliminating the need for vehicles to quickly cross multiple lanes. UDOT challenged the designer and contractor through the construction manager/general contractor (CM/GC) process to find a way to add a grade-separated crossing under 10600 South within a 16-day road closure. Ultimately, the team developed a three-sided box tunnel (123 ft 5 in. long, 22 ft high, and 37 ft 4 in. inside width) with a lateral slide solution—the first of its kind in Utah. This innovation allowed the contractor to build the three-sided structure outside of existing traffic. Without using a lateral slide, traditional construction would have required three phases, which would have likely taken 9 months and closed traffic lanes.

Precast concrete footing section being set for the Interstate 15 Northbound/10600 South Interchange three-sided box structure. In the background is the box structure where it was staged before the slide. After the slide, the section between the parallel precast concrete footings was cast in place to complete the bottom slab of the structure's foundation.

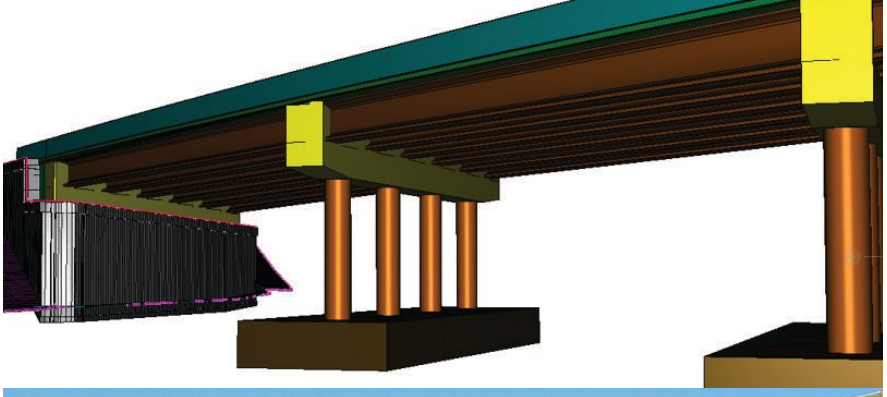
Soils in the Salt Lake valley are highly compressible, so most structures are constructed with deep foundations. Concerned about vibrations associated with driving piles near an existing sewer line, the project team investigated using spread footings. However, the soils could not provide sufficient strength to resist the pressure developed by strip footings under the three-sided structure walls, so the team developed a full-bottom slab solution to distribute the loads. The greatest design efficiency was the 15 by 10 by 4 ft precast concrete footing sections that provided a solid and level surface for the slide system. After the slide, the full-bottom slab was completed by placing concrete in the midsection between the parallel footings.

During the 16-day closure, crews worked over 10,000 man-hours to remove existing concrete pavement, excavate 9200 yd^3 of soil (much of which was reused as fill around the underpass), set 16 precast concrete footings weighing 56,000 pounds each, slide the structure into place, backfill, and construct approach slabs and the portland cement concrete pavement, while also ensuring sufficient cure time prior to opening. In three hours, crews slid the 3 million–pound structure 150 ft, where it connected with a 60 ft section of the underpass on the north side of 10600 South. This project successfully demonstrated the application of lateral slides to structure types other than typical highway bridges to minimize impacts to traffic, businesses, and adjacent communities.

Tooele Interchange at Lake Point Bridge

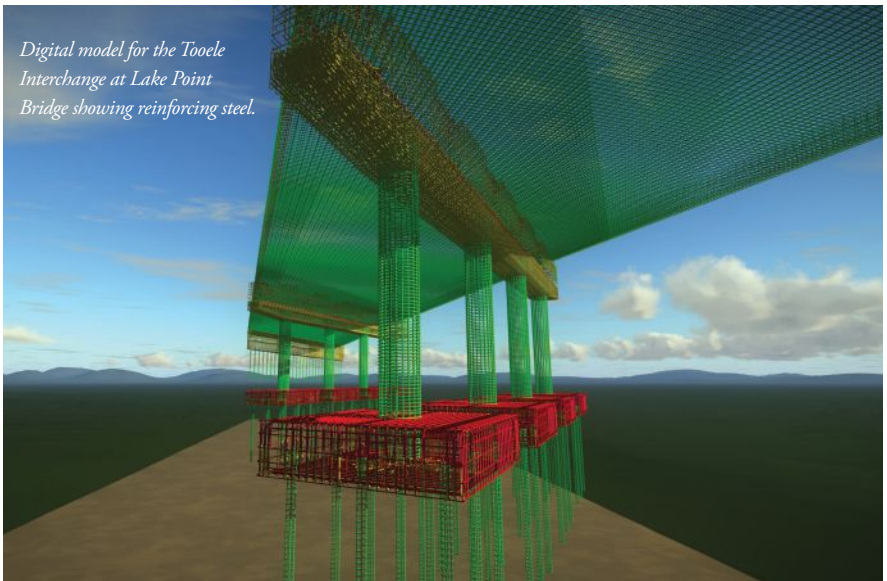
UDOT has a digital delivery program with four goals: produce more optimal designs; improve information transfer; obtain and manage better data to improve decision-making; and improve efficiency. The aim is to have a single source of information for all entities from design to construction and, ultimately, downstream for asset management and planning.

In 2019, UDOT chose a pilot project involving three bridge replacements at two locations along Interstate 80—Blackrock and State Route 36 (SR 36)—to use three-dimensional (3-D) model-based information exchanges for bridges. The 3-D model would be the legal document for construction with no plan sheets, and UDOT selected the CM/GC process to identify and resolve issues with the building information modeling (BIM)–based information exchanges.



Views comparing the Tooele Interchange at Lake Point Bridge model and the structure during construction.

Weekly model review and coordination meetings were held for the Tooele Interchange at Lake Point Bridge. The meetings reviewed updates to the three-dimensional model and construction activities.



Digital model for the Tooele Interchange at Lake Point Bridge showing reinforcing steel.

The consultant, designer, and contractor would use the project to help the UDOT Structures Division develop processes, standards, and procedures for BIM in project delivery.

The Tooele Interchange at Lake Point (SR 36) Bridge was set on a new alignment adjacent to the existing bridge. The new three-span superstructure, which consisted of six prestressed concrete girders with a cast-in-place concrete deck, had a complex geometry with both a vertical curve and a superelevation transition.

During construction, the contractor, UDOT, inspectors, designers, subcontractors, and suppliers met for weekly model coordination meetings to review updates to the model as well as current and upcoming activities. The meetings served as an effective way to collaborate. Initially, getting software loaded onto tablets and training personnel to extract data from the model for construction activities were hurdles for the contractor. The weekly meetings helped facilitate and mitigate such issues. Some efficiencies were found working through the BIM project delivery, such as collaborating with the contractor and their reinforcing steel supplier using data transfer to develop approved shop drawings for fabrication. Additional efficiencies are anticipated as more field tools are developed and as designers and contractors become more comfortable with new methods of data transfer and exchange.

Conclusion

For UDOT, collaboration with industry partners has been a cornerstone in the successful implementation of alternative construction processes and delivery methods. Going forward, concrete superstructures and prefabricated bridge elements will continue to play a key role in both accelerating projects and improving efficiencies, and digital delivery promises to facilitate information exchange among designers, contractors, and fabricators.

Reference

1. Wu, J.T.H., K.Z.Z. Lee, S.B. Helwany, and K. Ketchart. 2006. *Design and Construction Guidelines for Geosynthetic-Reinforced Soil Bridge Abutments with a Flexible Facing.* National Cooperative Highway Research Program (NCHRP) Report 556. Washington, DC: Transportation Research Board. http://www.trb.org/Main/Public/Blurbs/157301.aspx.

Cheryl Hersh Simmons is the chief structural engineer for the Utah Department of Transportation in Salt Lake City.

Adaptability Through Replaceable Post-Tensioning

by Reggie Holt, Federal Highway Administration

There is great interest within the concrete bridge community regarding promising technologies that provide bridge owners the ability to address unanticipated issues with their in-service bridges. Post-tensioning (PT) technologies have shown great promise in being able to address such unanticipated in-service issues as increased dead load, increased live load, poor in-service PT tendon performance, unforeseen deflections, and structural distress.

Several available PT technologies can provide supplemental structural resistance to in-service bridges to address these unanticipated issues. This article will focus on PT technologies applied to external PT systems, such as the typical external PT tendons in a concrete box girder (**Fig. 1**).

Figure 1. Typical external post-tensioning tendons in a box girder. Photo: Teddy Theryo.

Provisional/Future Post-Tensioning

Bridge engineers have always valued techniques and strategies that address unforeseen issues. Since the first edition, the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[1] have included requirements to provide provisional or extra prestressing force to compensate for unexpected prestress losses during construction and future dead loads as well as to control cracking and deflections. In the current edition of the AASHTO LRFD specifications, these provisions appear in Article 5.12.5.3.9.[2] This supplemental prestressing force is typically provided by installing supplemental, unused PT tendon anchorages and deviators to allow installation of additional PT tendons during the bridge's service life. However, the prescribed minimum level of future prestressing force (10% of the primary positive- and negative-moment PT force) may be insufficient to address future needs. In addition, in many instances, the future PT tendons are located in structurally inefficient locations due to the limited area available within the structure's cross section.

External Replaceable PT Tendon Details

In the United States, interest in fully replaceable PT systems is a fairly recent trend, although the technology has been used in other countries for decades. These replaceable systems can be designed to allow tendon detensioning, force adjustment, and full or partial replacement. **Table 1** shows replaceable tendon requirements in four countries.

Replaceable external tendons are not bonded to the concrete at any location along the length of the tendon. Forces from the PT tendon are transferred to the superstructure by bearing plates at the anchorages and bearing of the tendon duct against the concrete at the diaphragms and deviators. **Figure 2** shows a replaceable external tendon that uses the double-envelope concept, where the anchorage and duct hardware pass through guide pipes and diabolos, so they are not bonded to the structure and can be replaced. The details for the fully replaceable tendon allow easy removal due to the outer isolating envelope provided by the guide pipes and diabolo void, which eliminates concrete encasement of the tendon anchorage and deviator components.

There are variations on the concept shown in Fig. 2. The tendon can be filled with either a cementitious grout or flexible filler material, typically wax or grease. One advantage to the use of flexible fillers is that the tendon prestressing force can be detensioned and/or adjusted. However, adjustable tendons will require a nonstandard anchorage, typically with threaded heads. Detensionable tendons also require the strands of the tendon to extend past the wedge plate

Table 1. External Tendon Requirements in Selected Countries

Country	Detensionable	Replaceable	Adjustable
France	✓	✓	
Germany	✓	✓	✓
Japan		✓	
United States		✓*	

Florida and Virginia.

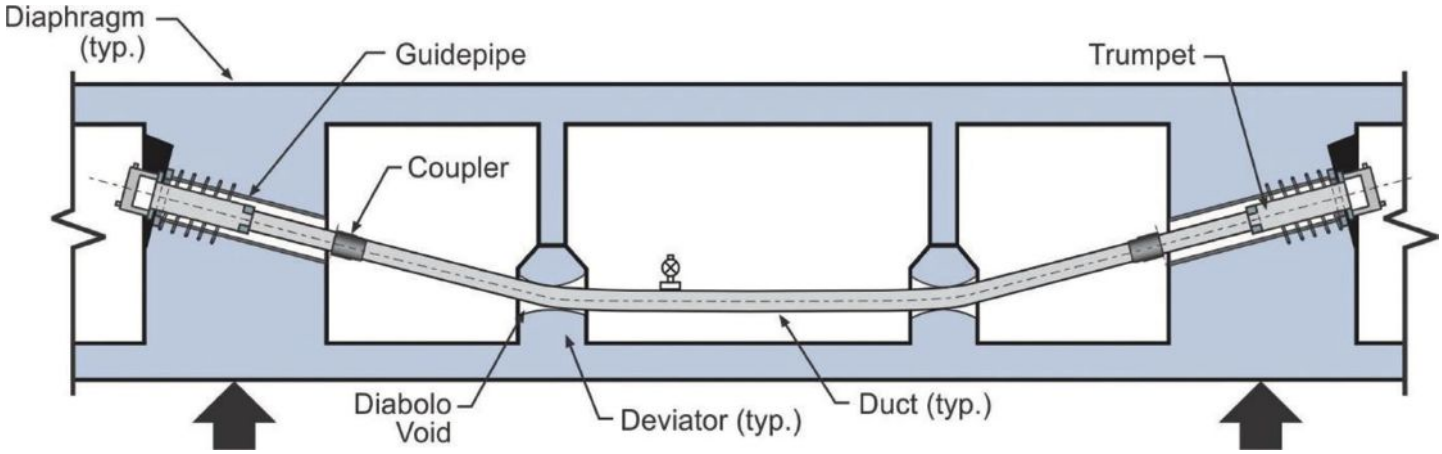
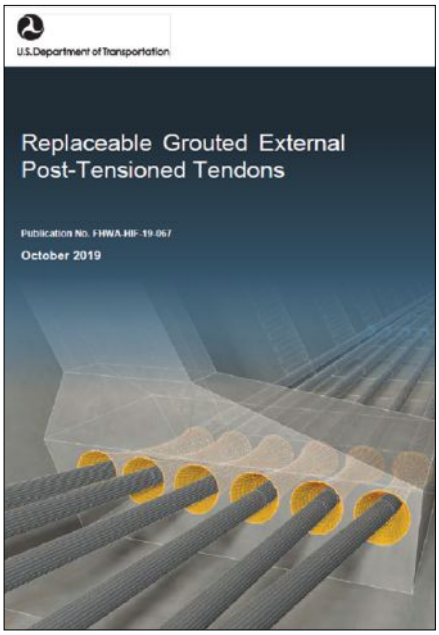


Figure 2. Replaceable external tendons using the double-envelope concept. Figure: WSP.

Figure 3. External tendons using epoxy-coated strands in a box girder. Strands are not enclosed in a duct except where they pass through diaphragms. Photo: Sumiden Wire.



The Federal Highway Administration's *Replaceable Grouted External Post-Tensioned Tendons* guidance document. Figure: Federal Highway Administration.

for a greater than typical length to provide sufficient length to "regrip" the strand. Experience in Europe has shown that removal of a replaceable grouted tendon may be more cost effective than removal of a tendon protected with a flexible filler because effort is required to extract and clean out the flexible filler.

Another replaceable external PT tendon concept slightly different from what is shown in Fig. 2 uses epoxy-coated prestressing strands (see the Concrete Bridge Technology article in the Spring 2020 issue of *ASPIRE*®). The epoxy coating provides a robust layer of protection, which eliminates the need to encase the prestressing strands in a duct and filler material. **Figure 3** shows a replaceable external tendon using epoxy-coated strands.

One significant advantage of the replaceable PT technologies mentioned here is that they can be designed to provide a larger replacement prestressing force, which could provide future benefit when addressing unanticipated issues.

Additional Information

In many bridges, PT tendons provide a significant portion of the load-carrying capacity. Consequently, there is great interest in ensuring that PT tendons perform well during the structure's service life and have the flexibility to address unanticipated in-service issues. Innovative PT technologies that allow removal, adjustment, and addition of prestressing force can offer great value.

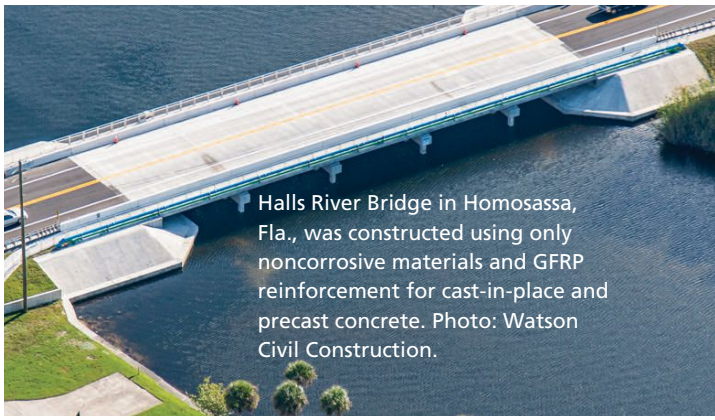
To provide the bridge community with information on promising replaceable PT tendon technologies, the Federal Highway Administration (FHWA) has developed a guidance document on external grouted replaceable PT tendons,[3] which is available on the FHWA Concrete Bridges web page. In addition, a video describing the replaceable PT tendon technology is available on YouTube.[4]

References

1. American Association of State Highway and Transportation Officials (AASHTO). 1994. *AASHTO LRFD Bridge Design Specifications*, 1st ed. Washington, DC: AASHTO.
2. AASHTO. 2020. *AASHTO LRFD Bridge Design Specifications*, 9th ed. Washington, DC: AASHTO.
3. Federal Highway Administration (FHWA). 2019. *Replaceable Grouted External Post-Tensioned Tendons.* FHWA-HIF-19-067. Washington, DC: FHWA. https://www.fhwa.dot.gov/bridge/concrete/hif19067.pdf.
4. FHWA. 2018. *Bridge Technology Series: Replaceable Grouted External Post-Tensioned Tendons.* YouTube video, posted September 2018. https://www.youtube.com/watch?v=N3j8ATiqcAs&index=1&list=PL5_sm9g9d4T3ymcxjfVEdYnd2-Hj-Fozl&t=0s.

Glass Fiber-Reinforced Polymer (GFRP) Reinforcement for Bridge Structures

by Steven Nolan, Florida Department of Transportation, Matthew Chynoweth, Michigan Department of Transportation, and Dr. Antonio Nanni, University of Miami



Halls River Bridge in Homosassa, Fla., was constructed using only noncorrosive materials and GFRP reinforcement for cast-in-place and precast concrete. Photo: Watson Civil Construction.

The development of comprehensive national bridge design standards is paramount to allow the broad and safe deployment of fiber-reinforced polymer (FRP) reinforced concrete in our transportation infrastructure. To respond to this need, the American Association of State Highway and Transportation Officials published the second edition of the *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete* (BDGS-GFRP) in December 2018.[1] Compared to the first edition,[2] changes were intended to achieve the following:

- Reflect state-of-the-art knowledge from peer-reviewed research
- Expand the application of GFRP reinforcement to all reinforced concrete bridge elements
- Improve the design economy of the existing provisions and address issues preventing designers from taking full advantage of the mechanical properties and durability of GFRP reinforcement
- Provide consistency with the *AASHTO LRFD Bridge Design Specifications*[3] for legacy construction materials to facilitate future incorporation of GFRP reinforcement
- Harmonize the BDGS-GFRP design philosophy with other authoritative national and international standards

In 2018, AASHTO also published *Guide Specifications for the Design of Concrete Bridge Beams Prestressed with Carbon Fiber-Reinforced Polymer (CFRP) Systems*.[4] This complementary guide specification is not as extensive in scope as BDGS-GFRP; however, it establishes the theoretical and practical design basis for prestressing concrete with materials that do not exhibit traditional yield behavior, and it provides a foundation for future code development efforts.

BDGS-GFRP is compatible with the eighth edition of the AASHTO LRFD specifications, sharing the same structure and organization and minimizing any differences in design equations to make BDGS-GFRP easier for designers to use. BDGS-GFRP diverges from the AASHTO LRFD specifications only to adjust design parameters and material properties to account for the different behavior of GFRP reinforcement compared with steel reinforcement. Importantly, the second edition of BDGS-GFRP refers to the GFRP reinforcing bar material specifications recently published by ASTM.[5]

A major limitation of the first edition of the BDGS-GFRP was that it addressed only bridge decks and open-post traffic railings. The second edition covers all reinforced concrete members in a bridge structure. The new edition is the first specification to cover GFRP-reinforced concrete substructures, and is arguably the most complete guide for GFRP-reinforced concrete design. Its provisions have been developed and implemented on a number of recently completed structures in Florida, such as the Innovation Bridge[6] and Halls River Bridge,[7] as well as several bridges under construction in southern Florida (for example, U.S. Highway 41 over North Creek and 23rd Avenue NE over Ibis Waterway), with more projects currently in final design (40th Avenue North over Placido Bayou, Barracuda Boulevard over Indian River North, County Road 30A over Western Lake, and West Wilson Street over Turkey Creek).

There are ample opportunities to further refine the current conservatism in the reduction factors for environmental degradation, fatigue, sustained load, and shear strain limits applied to GFRP-reinforced concrete design. Concurrently, manufacturers of GRFP reinforcement continue to improve the mechanical performance of their products by improving fiber content density, resin, and sizing formulations, thereby increasing both reinforcing bar strength and stiffness. These efforts can help increase the economy, durability, and sustainability of our infrastructure.

References

1. American Association of State Highway and Transportation Officials (AASHTO). 2018. *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete*, 2nd ed. Washington, DC: AASHTO.
2. AASHTO. 2009. *AASHTO LRFD Bridge Design Guide Specification for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings*. Washington, DC: AASHTO.
3. AASHTO. 2017. *AASHTO LRFD Bridge Design Specifications*, 8th ed. Washington, DC: AASHTO.
4. AASHTO. 2018. *Guide Specifications for the Design of Concrete Bridge Beams Prestressed with Carbon Fiber-Reinforced Polymer (CFRP) Systems*. Washington, DC: AASHTO.
5. ASTM International. 2017. *Standard Specification for Solid Round Glass Fiber Reinforced Polymer Bars for Concrete Reinforcement*. ASTM D7957/D7957M-17. West Conshohocken, PA: ASTM International.
6. Rossini, M., S. Spadea, and A. Nanni. 2019. "Pedestrian Bridge as Clarifying Example of FRP-RC/PC Design." *ACI Special Publication* 333(6): 96–118.
7. Rossini, M., E. Bruschi, F. Matta, C. Poggi, and A. Nanni. 2018. "Case-Specific Parametric Analysis as Research-Directing Tool for Analysis and Design of GFRP-RC Structures." *ACI Special Publication* 327(34): 1–12.

Steven Nolan is a senior structures design engineer for the Florida Department of Transportation in Tallahassee. Matthew Chynoweth is chief bridge engineer for the Michigan Department of Transportation in Lansing, and chair of the AASHTO Technical Committee on Fiber Reinforced Polymer Composites (T-6). Dr. Antonio Nanni is a professor at the University of Miami in Coral Gables, Fla.

Glass fiber-reinforced polymer reinforcement being placed in the form for a substructure element for the Halls River Bridge in Homosassa, Fla. Photo: Florida Department of Transportation.

PCI Offers New Transportation eLearning Modules

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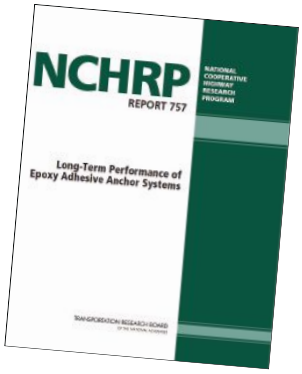
AASHTO The Voice of Transportation

PCI Precast/Prestressed Concrete Institute

Anchors in Concrete: Guidance for Bridge Engineers—Part 1 of a four-part series



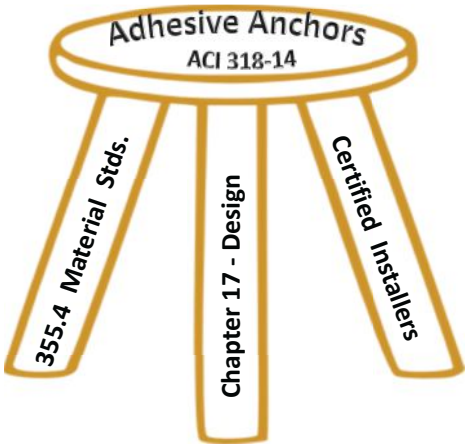
by Dr. Donald F. Meinheit

In 2006, adhesive-bonded anchors supporting a concrete-panel suspended ceiling in the Boston Interstate 90 Seaport Connector Tunnel ("Big Dig") project failed and 26 tons of concrete and hardware fell onto the roadway.[1] This tragic event resulted in one fatality and one injury and spurred an examination of the use of adhesive-bonded anchors carrying sustained tensile loads. Since that time, research has been performed,[2,3] and specifications have been introduced and improved for all anchor types. Furthermore, new resources to inform bridge engineers of design methodology and best practices are on their way.

NTSB Recommendations

A National Transportation Safety Board (NTSB) report[1] identified the creep of the epoxy adhesive as the cause of the Boston anchorage failure. The NTSB report contained safety recommendations for many parties, including the American Concrete Institute (ACI) and American Association of State Highway and Transportation Officials (AASHTO), to address the creep characteristics of the adhesive anchors.

There are three legs to the adhesive anchor quality stool: the design procedure in ACI 318-14 Chapter 17,[10] the qualification protocol in the ACI 355.4 material standard,[11] and an operational installer certification program. Figure and all Photos: Dr. Donald F. Meinheit.



Additionally, the NTSB report recommended that the Federal Highway Administration (FHWA) and all state departments of transportation (DOTs) prohibit the use of adhesive-bonded anchors in overhead tensile-loaded applications until testing standards and protocols were developed and implemented. That recommendation was based on what was known at that time. Prior to 2006, few studies on the sustained-load behavior of adhesive-bonded anchors had been done. The National Cooperative Highway Research Program managed by the Transportation Research Board (TRB) has subsequently published two studies on adhesive anchors in concrete under sustained loading.[2,3] That research, as well as improvements in regulations, specifications, and certification programs, contributed to a January 2018 FHWA Technical Advisory (T5140.34),[4] which canceled the previously issued T5140.30,[5] which strongly discouraged the installation of certain fast-set epoxy anchors. FHWA T5140.34 gives recommendations for the design of adhesive anchors in new Federal-Aid projects and guidance for inspection, retrofit, or replacement of anchors on existing projects.[6]

Improved Specifications

The first design provisions for cast-in-place and some types of post-installed anchors appeared in ACI's *Building Code Requirements for Structural Concrete* (ACI 318-02) *and Commentary* (ACI 318R-02),[7] and adhesive anchor design provisions were added in ACI 318-11.[8] Prior to ACI 318-11, design provisions for adhesive anchors were not included in any widely used U.S. design standard. When the eighth edition of the *AASHTO LRFD Bridge Design Specifications*[9] was published in 2017, concrete anchors were covered in a reorganized Section 5. Instead of writing its own design requirements for concrete anchors, AASHTO's Technical Committee T-10 Concrete Design adopted, with a few amendments, ACI 318-14[10] (see AASHTO LRFD articles in the Summer 2015 and Winter 2017 issues of *ASPIRE*®).

Recognizing the Human Factors

In its report on the Boston tunnel failure,[1] NTSB recommended that ACI use its "building codes, forums, educational materials, and publications to inform the design and construction agencies of the potential for gradual deformation (creep) in anchor adhesives…" Following that recommendation, ACI has been on a campaign to inform the engineering design community of the design provisions in ACI 318 regarding anchoring to concrete.

Additional factors that contributed to the Big Dig incident were the irregular installation of the adhesive anchors in an overhead condition and the inspectors' failure to understand the critical implications when they saw anchors displaced from their embedment holes.

Although addressing the human factors that can contribute to failures is recognized as an important step toward improving the performance of installed adhesive anchors, there remain substantial knowledge gaps to overcome. Undergraduate and graduate courses in concrete design typically do not cover concrete anchors. Also, bridge designers tend to be less familiar than designers of other types of facilities with anchoring to concrete because the AASHTO LRFD specifications did not include guidance until 2017, about

Concrete breakout failure of an anchor where the tension load strength has been influenced by the distance from the center of the anchor to the free edge of the member.

15 years after ACI 318-02 was published. It has therefore become evident to many stakeholders in the concrete bridge community that educational programs for structural designers are urgently needed to improve their understanding of anchors in concrete and the relevant parts of ACI 318 and the AASHTO LRFD specifications.

New Educational Opportunities

To respond to this need, the Prestressed/Precast Concrete Institute (PCI) submitted a proposal to TRB to prepare a comprehensive training program for highway bridge engineers on implementation of the new provisions on concrete anchors. PCI envisioned working with the TRB panel on a series of webinars so designers in every state would consistently receive the same important information on design, specification, approval, installation/construction, and inspection of all types of anchors to concrete.

An example of an external support anchored using post-installed concrete anchors. Anchors are loaded in combined tension and shear.

To lead this effort, PCI selected a team of subject matter experts to explain the unique characteristics of concrete anchors to designers and specifiers. The PCI team is composed of Dr. Ronald A. Cook, professor emeritus, University of Florida-Gainesville; Neal S. Anderson, staff consultant, Simpson, Gumpertz, and Heger, Chicago, Ill.; and Dr. Donald F. Meinheit, retired principal, Wiss, Janney, Elstner Associates Inc., Chicago, Ill. These subject matter experts have collaborated to develop and present two pilot, daylong seminars to highway bridge engineers. These modules and materials will be made available to the bridge community as 90-minute webinars with an additional 30 minutes for questions and answers.

To design, install, and inspect an anchor embedded in concrete, one needs more than an ability to follow rote guidelines. A thorough understanding of the failure modes of tension-loaded anchors and shear-loaded anchors is an important step in understanding the design provisions. Because concrete anchors are complex structural elements, the subject matter expert team, the FHWA, and industry committees have taken a holistic approach to educate the bridge community. The program developed by the team of experts assembled by PCI has major sections devoted to the following:

- Understanding the failure modes in tension and in shear
- Reviewing the basic code design equations for single anchors and anchor groups
- Reviewing the basic code design equations accounting for edge distance, anchor spacing, eccentricity, and cracking of the concrete
- Discussing which concrete anchor is best suited for a particular application
- Reviewing free design software options
- Discussing at length why anchors must be qualified against a standard and where to find the qualification data for a particular anchor from a particular manufacturer
- Reviewing how to procure the anchor specified on contract drawings and in specifications
- Discussing why there are two certification programs, one for adhesive anchor installers and one for anchor inspectors
- Reviewing field implementation and proof testing

All participants receive information for each course as a download, which includes course resource documents and copies of the slide presentation along with a transcript of the presenter's comments for each slide.

Pilot presentations of the seminar to two state highway bridge design groups were well received, and plenty of technical questions were asked. Once the TRB panel accepts the deliverable, which is anticipated by mid-2020, all information will be available on the TRB and PCI websites as a free download that can be used and customized by individual state highway agencies.

Future Articles in This Series

Future articles in this four-part series based on the PCI training program

on concrete anchors will focus on anchor qualification, specifications and procurement, and inspection and compliance testing.

References

1. National Transportation Safety Board (NTSB). 2007. *Ceiling Collapse in the Interstate 90 Connector Tunnel, Boston, Massachusetts, July 10, 2006.* Highway Accident Report NTSB/HAR-07/02. Washington, DC: NTSB. https://www.ntsb.gov/investigations/AccidentReports/Reports/HAR0702.pdf.
2. Cook, R.A., E.P. Douglas and T.M. Davis. 2009. *Adhesive Anchors in Concrete Under Sustained Loading Conditions*. National Cooperative Highway Research Program (NCHRP) Report 639. Washington, DC: Transportation Research Board. https://doi.org/10.17226/23026.
3. Cook, R.A., E.P. Douglas, T.M. Davis, and C. Liu. 2013. *Long-term Performance of Epoxy Adhesive Anchors Systems*. NCHRP Report 757. Washington, DC: Transportation Research Board. https://doi.org/10.17226/22470.
4. Federal Highway Administration (FHWA). 2018. FHWA Technical Advisory T5140.34 *Use and Inspection of Adhesive Anchors in Federal-Aid Projects.* January 16, 2018. https://www.fhwa.dot.gov/bridge/t514034.pdf.
5. FHWA. 2008. FHWA Technical Advisory T5140.30. *Use and Inspection of Adhesive Anchors in Federal-Aid Projects.* March 21, 2008 (canceled January 16, 2018). https://www.fhwa.dot.gov/bridge/t514030.cfm.
6. Bland, T.J. 2020. "Federal Changes for Post-installed Adhesive Anchors." *STRUCTURE* 27(4): 16–17. https://www.structuremag.org/?p=15787.
7. American Concrete Institute (ACI) Committee 318. 2002. *Building Code Requirements for Structural Concrete* (ACI 318-02) *and Commentary* (ACI 318R-02). Farmington Hills, MI: ACI.
8. ACI Committee 318. 2011. *Building Code Requirements for Structural Concrete* (ACI 318-11) *and Commentary* (ACI 318R-11). Farmington Hills, MI: ACI.
9. American Association of State Highway and Transportation Officials (AASHTO). 2017. *AASHTO LRFD Bridge Design Specifications*, 8th ed. Washington, DC: AASHTO.
10. ACI Committee 318. 2014. *Building Code Requirements for Structural Concrete* (ACI 318-14) *and Commentary* (ACI 318R-14). Farmington Hills, MI: ACI.
11. ACI Committee 355. 2020. *Qualification of Post-Installed Adhesive Anchors in Concrete* (ACI 355.4-19) *and Commentary.* Farmington Hills, MI: ACI.

Column Ties for Nonseismic Applications

by Dr. Oguzhan Bayrak, University of Texas at Austin

In its 2019 meeting, the American Association of State Highway and Transportation Officials (AASHTO) Committee on Bridges and Structures (COBS) approved three changes to Section 5 of the *AASHTO LRFD Bridge Design Specifications*.[1] In the Fall 2019 issue of *ASPIRE*®, this column included in-depth review of two of those three changes. This article covers the third change, which relates to detailing of ties for columns that are not designed for plastic hinging, referred to herein as nonseismic applications. Although this topic was covered in my Fall 2019 article, bridge engineers raised questions about the exact nature of the revisions. More specifically, department of transportation engineers who contacted our *ASPIRE* team wanted to ensure that their new designs were compliant with the new specifications and would minimize issues for the next generation of bridge engineers.

Detailing requirements for column ties used in nonseismic applications were revised to make them consistent with the original intent of the 1980 Interim Revisions to the AASHTO *Standard Specifications for Highway Bridges*[2] and the underlying research.[3] The revised Article 5.10.4.3 in the ninth edition of the AASHTO LRFD specifications[4] includes the following language:

> For columns that are not designed for plastic hinging, the spacing of laterally restrained longitudinal bars or bundles shall not exceed 24.0 in. measured along the perimeter tie. A restrained bar or bundle is one which has lateral support provided by the corner of a tie having an included angle of not more than 135 degrees. Cross-ties with a 135-degree hook at one end and a 90-degree hook at the other end shall be alternated so that the 90-degree hooks are not adjacent to each other both vertically and horizontally.
>
> Where the column design is based on plastic hinging capability, no longitudinal bar or bundle shall be farther than 6.0 in. clear on each side along the perimeter tie from such a laterally supported bar or bundle and the tie reinforcement shall meet the requirements of Articles 5.11.4.1.4 through 5.11.4.1.6.
>
> Where the longitudinal bars or bundles are located around the periphery of a circle, a complete circular tie may be used with the splices in the circular ties staggered and without the need for cross-ties.
>
> Ties shall be located vertically not more than half a tie spacing above the footing or other support and not more than half a tie spacing below the lowest horizontal reinforcement in the supported member.

The commentary to the specification (Article C5.10.4.3) was revised to include an illustration (**Fig. 1**) that provides two examples intended to clarify how the

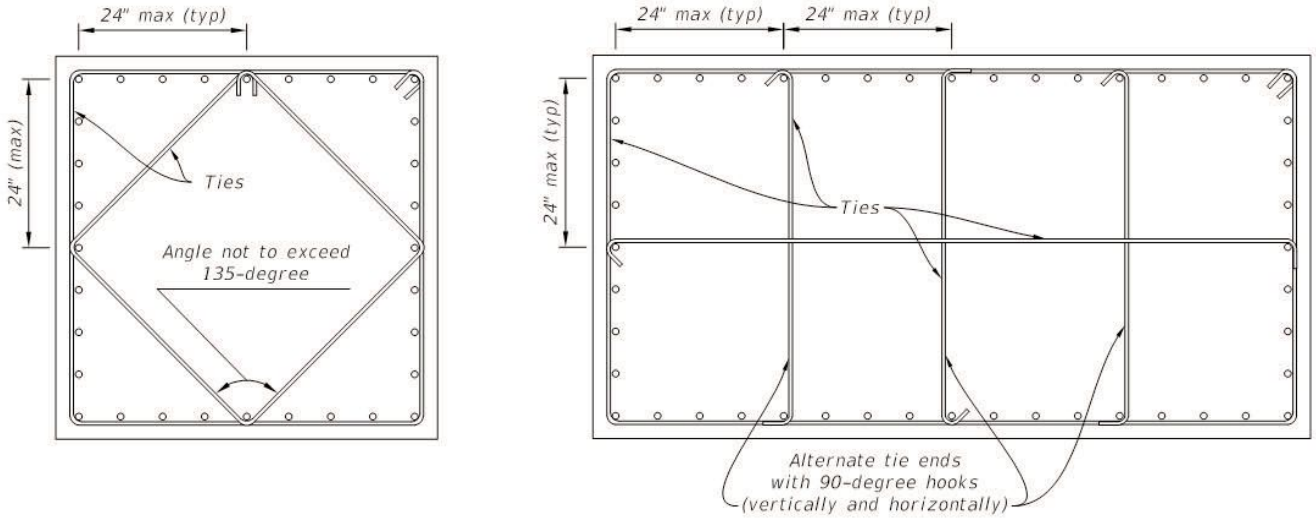


Figure 1. Detailing of column ties in locations not designed for plastic hinging. Reproduced by permission from AASHTO (2020).

new column tie detailing rules are to be applied. An examination of the cross sections shown in the figure shows the simplicity with which the new detailing rules can be summarized visually. Verbal descriptions, though essential in the context of mandatory language of specifications, may be more difficult to digest without the help of an illustration. The remaining detailing rules in Article 5.10.4.3 remain unchanged.

It is worth noting that the 24.0-in. limit in the updated specification has been revised a number of times since 1974. Until 1974, the AASHTO standard specifications used the 24-in. limit. In 1974, this limit was made more stringent and consistent with the American Concrete Institute's *Building Code Requirements for Reinforced Concrete* (ACI 318-71)[5] detailing rules. In 1980, the 24.0-in. limit was reinstituted to address constructability issues encountered with the more stringent limits. The 2007 AASHTO LRFD specifications[6] increased this limit to 48 in., the least restrictive limit in decades. As noted previously, the ninth edition of the AASHTO LRFD specifications returns the limit to 24.0 in.

The changes made to the detailing rules will improve the effective lateral support provided to longitudinal bars in column cages. Furthermore, the effectiveness of the confining forces provided to the structural core will also improve compared with designs controlled by the detailing rules of the eighth edition of the AASHTO LRFD specifications. Because the 24.0-in. limit was previously used in bridge columns, the changes summarized in this article are not expected to have adverse effects on constructability. Finally, the potential plastic hinge regions of reinforced columns and seismic detailing rules remain unchanged in the ninth edition of the AASHTO LRFD specifications.

References

1. American Association of State Highway and Transportation Officials (AASHTO). 2017. *AASHTO LRFD Bridge Design Specifications,* 8th ed., Washington, DC: AASHTO.
2. AASHTO. 1977. *Standard Specifications for Highway Bridges*, 12th ed. with 1980 Interim Revisions, Washington, DC: AASHTO.
3. Pfister, J.F. 1964. "Influence of Ties on the Behavior of Reinforced Concrete Columns." *American Concrete Institute Journal Proceedings* 61 (5): 521–538.
4. AASHTO. 2020. *AASHTO LRFD Bridge Design Specifications*, 9th ed., Washington, DC: AASHTO.
5. American Concrete Institute (ACI) Committee 318. 1971. *Building Code Requirements for Reinforced Concrete* (ACI 318-71). Farmington Hills, MI: ACI.
6. AASHTO. 2007. *AASHTO LRFD Bridge Design Specifications*, 4th ed., Washington, DC: AASHTO.

Dr. Oguzhan Bayrak is a professor at the University of Texas at Austin. He was inducted into the university's Academy of Distinguished Teachers in 2014.

Engineering Judgment and the Florida International University Pedestrian Bridge Collapse

by Dr. William D. Lawson, Murdough Center for Engineering Professionalism, Texas Tech University

This article offers observations about *engineering judgment* relative to the Florida International University (FIU) pedestrian bridge collapse of March 2018. Information about the FIU pedestrian bridge collapse comes mostly from my review of several publications about this incident—in particular, the National Transportation Safety Board (NTSB) Highway Accident Report[1] issued in October 2019. I relied on the NTSB report as the basis for many of my observations, but I am aware that there are other detailed reports, studies, and project data that I have not reviewed.

About Engineering Judgment

Judgment is central to engineering and many other professional activities.[2] For example, engineering licensure laws identify sound judgment as a requirement for the professional practice of engineering.[3] Judgment is the means by which "evidence is recognized, supporting evidence compiled, conflicting evidence reconciled, and evidence of all kinds weighed according to its perceived significance."[4]

Engineers in certain disciplines intentionally consider how judgment influences their work, and here I think geotechnical engineering holds some prominence. The book *Judgment in Geotechnical Engineering*[5] presents lectures, papers, and other writings by eminent geotechnical engineer Ralph B. Peck. Building on his legacy, judgment remains an active and vital aspect of geotechnical engineering today. National Academy of Engineering member Allen Marr recently noted, "We must grapple with uncertainty in all aspects of our work: the project environment, the site data, limitations of our models, unknowns about construction, and others."[6] That is the nature of engineering judgment, or how geotechnical engineers see it, anyway.

Judgment is central to engineering and many other professional activities.

Engineering Judgment in the NTSB Report

The term "judgment" appears in the NTSB report 12 times: twice in the Executive Summary; three times in Chapter 1, "Factual Information"; six times in Chapter 2, "Analysis"; and once in the Conclusions. A closer look reveals the report's sharpest comments about judgment apply to design errors and misinterpretation of precollapse distress:

- "…used poor judgment when it determined that the bridge was a redundant structure" (p. 72).
- "…used poor engineering judgment and … chose not to use the higher demand model results…and did not provide a rationale for the engineering judgment it used when selecting modeling results" (p. 78).
- "…displayed poor engineering judgment by failing to recognize the extensive, large cracks observed in the member 11/12 nodal region as being abnormal for a reinforced concrete structure" (p. 92).
- "…this decision was based on judgment that returning the main span to its preexisting condition… as the right thing to do.…The NTSB does not agree" (pp. 94–95).

In addition to these specific instances, the NTSB report indicates poor engineering judgment and response to precollapse cracking by all parties—the design-builder, the designer, the construction project administrator/inspector, the owner/construction manager, and the state transportation agency—contributed to the severity of the collapse outcome.

Different Perspectives for Different Disciplines

I find it significant that the term "engineering judgment" appears so prominently in NTSB's analyses pertaining to causation of a structural engineering failure. NTSB vice chairman Bruce Landsberg states, "A bridge-building disaster should be incomprehensible in today's technical world," and "the science should be well sorted out by now."[7] The implication is that structural engineering risk has been handled probabilistically through research that underlies published code provisions. The (unwritten) corollary to such a view is critical: When an engineer "follows the code," engineering judgment is already handled and thus does not come conspicuously into play. This is strikingly different from branches of civil engineering where—because of limited knowledge or information (for example, geotechnical) or because of the randomness and variability of nature (for example, water resources)—these engineers are *often* thinking in terms of engineering judgment.[8]

How do *structural* engineers see the matter? Senior principal structural engineer and Fellow of the Structural Engineering Institute of the American Society of Civil Engineers (ASCE) J.G. Soules has commented:

> There is a growing number of regulators who believe the codes have solved all of the problems and if you follow the codes without question, you will not have problems or failures on your projects. As

> the vice chair of ASCE 7 [Committee on Minimum Design Loads for Buildings and Other Structures], I can tell you the codes are minimum requirements and that engineering judgment is very necessary in the design of safe structures. We who write the codes understand we cannot possibly cover every situation with rules. We also do not want to stifle innovation with draconian rules. I also know a growing number of younger engineers who believe they can analyze any problem (correctly) with today's software. While today's software makes many unique structures possible, a computer model is simply a simplified representation of reality based on a lot of assumptions. Many younger engineers accept the software defaults for modeling as gospel—they basically substitute a programmer's judgment for their own when they do this. Engineering judgment is still sorely needed in our profession (personal email communication, April 19, 2020).

Soules persuasively advocates that, notwithstanding the necessity and importance of codes, practicing structural engineers value engineering judgment and give it priority. But barriers exist for both the cultivation and the practice of judgment. What do we do about that?

"We who write the codes understand we cannot possibly cover every situation with rules."

Lessons We Must Learn

As a civil engineer educator, I routinely assign senior undergraduate students to read articles and write an essay defining engineering judgment and explaining how it is obtained. My personal experience indicates that prior to the assignment, students "rarely" have encountered a definition of engineering judgment and most are "not sure" how to obtain judgment. A survey of engineering faculty shows they "strongly agree" that engineering judgment is important for problem-solving, but most struggle to "name specific things students can do to obtain judgment," and more are "not sure" how to assess judgment.

My point is this: If engineering judgment is important for engineering practice but faculty do not know how to teach or assess engineering judgment, and if students neither know what judgment is nor understand how to obtain it, we should not be surprised when "poor judgment" is prominently cited as the cause of a tragic bridge failure. I know there is more to it, but Landsberg asks, "Why?" My response is that if complacency existed in this failure, it was related to the lack of a proper understanding and use of engineering judgment. Such disasters point to a need for engineers to intentionally and continuously cultivate engineering judgment, not only during college but also through the internship period, and throughout their careers.

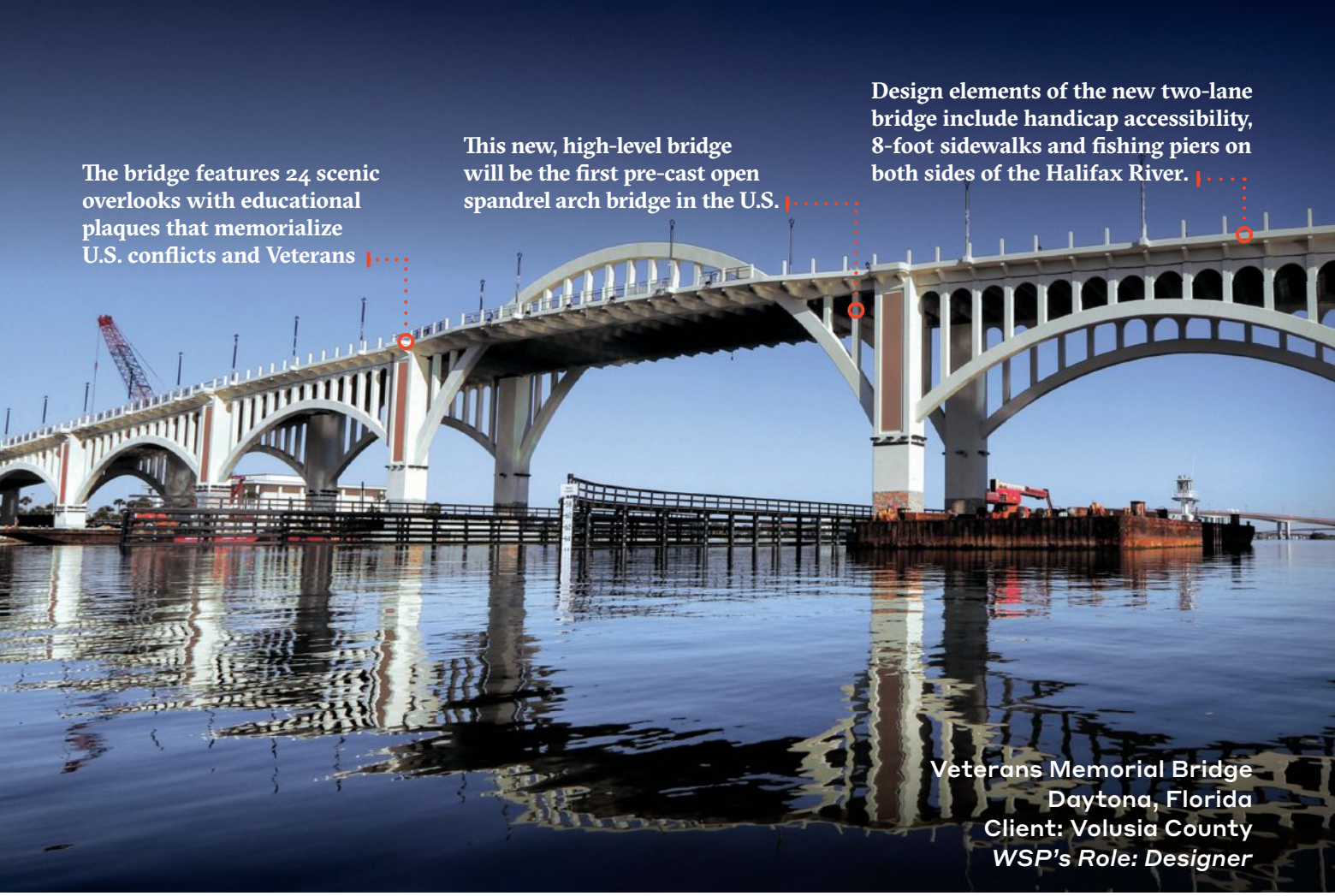
There is some good news on this point. The Accreditation Board for Engineering and Technology (ABET) has required that, effective fall 2019, "engineering judgment" is to appear in student learning outcomes. In the "Criteria for Accrediting Engineering Programs," Criterion 3: Student Outcomes,[9] now lists:

- Outcome 4: An ability to recognize ethical and professional responsibilities in engineering situations and make informed *judgments*, which must consider the impact of engineering solutions in global, economic, environmental, and societal contexts (emphasis added)
- Outcome 6: An ability to develop and conduct appropriate experimentation, analyze and interpret data, and use *engineering judgment* to draw conclusions (emphasis added)

This is an important start, a beginning to be celebrated. Moving forward, if redemption from tragedy is possible, perhaps no better path exists to honor the people lost in the FIU pedestrian bridge collapse than for engineering leaders, including those directly involved in this incident, to invest in our profession—especially the next generations of engineers—by teaching the important lessons judgment requires we must all learn.

References

1. National Transportation Safety Board (NTSB). 2019. *Pedestrian Bridge Collapse Over SW 8th Street, Miami, Florida, March 15, 2018*. Highway Accident Report NTSB/HAR-19/02 PB2019-101363. Washington, DC: NTSB. https://www.ntsb.gov/investigations/AccidentReports/Reports/HAR1902.pdf.
2. Davis, M. 2012. "A Plea for Judgment." *Science and Engineering Ethics* 18 (4):789–808. doi:10.1007/s11948-011-9254-6.
3. Florida Administrative Code. 2019. "Chapter 61G15 Board of Professional Engineers Organization and Purpose." Florida Board of Professional Engineers website. https://fbpe.org/wp-content/uploads/2020/01/61G15-as-of-12-29-19.pdf.
4. Vick, S.V. 2002. *Degrees of Belief: Subjective Probability and Engineering Judgment*. Reston, VA: American Society of Civil Engineers.
5. Dunnicliff, J., and D.U. Deere, eds. 1984. *Judgment in Geotechnical Engineering: The Professional Legacy of Ralph B. Peck*. New York, NY: Wiley.
6. Marr, W.A. 2020 (January-February). "Geotechnical Judgment." *GEOSTRATA*. http://www.readgeo.com/geostrata/jan_feb_2020/MobilePagedArticle.action?articleId=1551319#articleId1551319.
7. Landsberg, B. 2019. "Board Member Statement." In: National Transportation Safety Board. *Pedestrian Bridge Collapse Over SW 8th Street, Miami, Florida, March 15, 2018*, pages 106–107. Highway Accident Report NTSB/HAR-19/02 PB2019-101363. Washington, DC: NTSB. https://www.ntsb.gov/investigations/AccidentReports/Reports/HAR1902.pdf.
8. Baecher, G.B. 2019. "Putting Numbers on Geotechnical Judgment." Companion Whitepaper to the 19th Buchanan Lecture, presented at Texas A&M University, College Station, TX, 18 Oct 2019. https://www.dropbox.com/s/eukf85id6jk8jxj/19th%20Buchanan%20Lecture%20%282019-10-08%29%20posted.pdf?dl=0.
9. Accreditation Board for Engineering and Technology (ABET). 2018. "Criteria for Accrediting Engineering Programs, 2019–2020." Baltimore, MD: ABET. https://www.abet.org/wp-content/uploads/2018/11/E001-19-20-EAC-Criteria-11-24-18.pdf.



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