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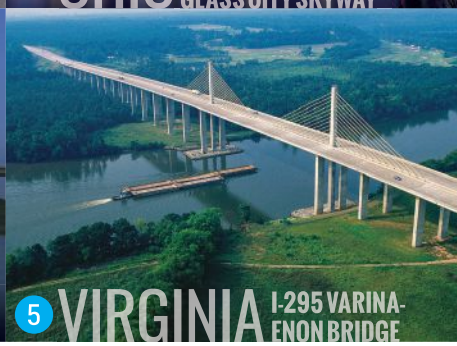

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EDITORIAL

Photo: PCI.

Our Conversation Continues: *Philotimo* and the Future of Bridge Engineering

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

In the last issue of *ASPIRE*® (Winter 2020), we started a difficult conversation about how our industry can best move forward after a tragic event. In that issue's editorial, Gregg Freeby of the American Segmental Bridge Institute and I shared our thoughts following the release of the final National Transportation Safety Board (NTSB) report[1] on the 2018 Florida International University (FIU) pedestrian bridge collapse. In that same issue, Leon H. Grant, vice president of operations at Coreslab Structures (Conn.) Inc., also offered a perspective on how a bridge collapse more than a century ago inspired an oath of professionalism that every engineering student in Canada takes—and the profound influence that oath has had on his own career. Many of you have joined the conversation, and your comments have given me a lot to think about.

One person who wrote to me about the last issue of *ASPIRE* was a retired 87-year-old structural engineer who wants to know why the bridge construction community seems to have shifted away from traditional checks and balances. This commenter worries that we may no longer confidently assume that (a) materials engineers collect and validate the data used by the construction inspectors; (b) project engineers verify the inspection findings; (c) oversight bridge engineers validate field requests from the owner's project managers and contractors; (d) project and field staff are empowered to engage the responsible bridge engineers when they have concerns; or (e) leadership will escalate issues as needed. More broadly stated: Do we still cross-check each other's data and calculations? And are we prepared to take a pause to do what is right?

That final question reminds me of a concept I learned when being taught right from wrong in my youth. My Greek family often spoke of *philotimo*. A rough English translation might be "love of honor" or "friend of honor," but neither phrase fully captures the meaning of the Greek word. In fact, I have been told *philotimo* doesn't have an exact English counterpart. Trying to describe this "Greek word without a meaning," behavioral expert John R. "Jack" Schafer, PhD, a professor in the Law Enforcement and Justice Administration Department of Western Illinois University wrote, "*Philotimo* encompasses the concepts of pride in self, pride in family, pride in community, and doing the right thing. *Philotimo* is an all-encompassing concept that gives meaning to life that stretches well beyond ourselves. Elder Paisiou aptly defined *philotimo* as 'that deep-seated awareness in the heart that motivates the good that a person does. A *philotimos* person is one who conceives and enacts eagerly those things good.'"[2] To my mind, what the retired engineer expressed was a concern that professionals in our industry today might lack *philotimo*.

To move forward as a community and industry that exemplify the values of *philotimo*, we are going to need to grapple with the implications of mistakes we have made, share our points of view, and really engage with the perspectives of others. During the 2020 Transportation Research Board (TRB) Construction of Bridges and Structures Committee meeting on January 13, 2020, Alan Fisher, chief engineer for Cianbro Construction Company, asked the room full of engineers how many had read the thoughtful statement by the NTSB vice chairman (dated October 28, 2019) about what went wrong—and why—in the FIU bridge collapse. Only four people of the 30-plus in attendance had read the letter, which could be a sign that we as a community may not be reckoning with the implications of that event and others like it for our profession. The statement is republished in full in this issue (page 51). I encourage you to read it, reflect on it, and discuss it with your colleagues.

Also on January 13, Steve DeWitt (formerly the North Carolina Department of Transportation construction engineer) moderated the TRB panel discussion on alternative delivery and contract risk sponsored by the TRB Standing Committee on Project Delivery Methods. This discussion suggested to me that the balance in public-private partnerships may

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Erection of the final segment in the Interstate 59/Interstate 20 elevated bridge replacement project in Birmingham, Ala. Photo: Alabama Department of Transportation.

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have shifted too far toward the private sector. This is yet another challenge that we as an industry will need to fold into our ongoing conversations about our future.

Based on the comments we received about the editorial and Leon's perspective in the Winter 2020 issue, we have decided to dedicate space in each issue of *ASPIRE* to broader engineering topics. In this issue, for example, Gregg and I offer a perspective on the human dynamics of closing a bridge or the roadway under a bridge (page 50).

The bridge community needs to revisit what the NTSB vice chairman calls the "safety management system" that should prevent catastrophes. We must work to never allow human habits and the time pressures of project delivery to disrupt our bridge engineer's *philotimo*.

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. https://www.ntsb.gov/investigations/AccidentReports/Reports/HAR1902.pdf.
2. Schafer, J.R. 2015 (August 15). "*Philotimo*: A Greek Word Without Meaning but Very Meaningful" *Psychology Today* [blog post]. https://www.psychologytoday.com/us/blog/let-their-words-do-the-talking/201508/philotimo-greek-word-without-meaning-very-meaningful.

DISCUSSION

The Effects of Strand Debonding

This is a discussion of "The Effects of Strand Debonding" by Dr. Oguzhan Bayrak published in the Winter 2020 issue of *ASPIRE* magazine (pp. 52–53). The explanations given by Dr. Bayrak are clear and welcomed. We would like to bring attention to two items:

1. In the paragraph following Eq. (5.7.3.5-2), Dr. Bayrak explains that stirrups in excess of those required to correspond to $V_s = (V_u/\phi_v)$ should not be used to artificially reduce the amount of longitudinal tie steel required. This is in accordance with the AASHTO LRFD specifications provision and should be followed. However, the equation given below the paragraph and described as the minimum value for the longitudinal tie force can be misleading and its use should be clarified. We recommend clarifying that the minimum requirement for longitudinal tie reinforcement should be the larger of Eq. (5.7.3.5-2) and the following equation given in the article:

$$A_{ps}f_{ps} + A_s f_y \geq \left(0.5\frac{V_u}{\phi_v} - V_p\right)\cot\theta$$

2. The discussion in the article, and also in the AASHTO LRFD specifications, indicates that debonded strands may not be used as part of the longitudinal tie reinforcement. We only partially agree with this statement. It is only valid when the debonded strands are not anchored after detensioning. Debonded strands can be anchored by embedding them into a cast-in-place diaphragm, which is a common detail in much of the United States for beams made continuous for live load. They can also be anchored by other means, such as using strand chucks and secondary end blocks. If debonded strands are anchored, it would be reasonable to assume that they contribute to the longitudinal tie resistance at the strength limit state.

Dr. Amgad Girgis, Micheal Asaad, and Dr. Maher Tadros
e.construct USA LLC
Omaha, Nebraska

Author's response

The author wishes to thank the discussers for their thoughtful comments. The points raised by the discussers are addressed in the same order.

1. The unnumbered equation in the subject article was provided for the purposes of clarifying the lower bound for Equation (5.7.3.5-2). The V_s value used in this equation cannot be greater than (V_u/ϕ_v). Therefore, the interpretation of the discussers is correct: Adding stirrups can only reduce the demand on the longitudinal tie up to a point. Put simply, the definition provided for V_s after Equation (5.7.3.5-1) in the AASHTO LRFD specifications reads "*shear resistance provided by transverse reinforcement at the section under investigation as given by Eq. 5.7.3.3-4, except V_s shall not be taken as greater than V_u/ϕ_v (kip)*." This limit placed on V_s should be taken into account when using Equation (5.7.3.5-2).
2. If positive anchorage is provided to debonded strands by extending them into diaphragms and bending them up or by using strand chucks, at the point where this positive anchorage is provided strand can be adequately developed or assumed to be "fixed." Under additional loads, it is true that the "fixed" or "positively anchored" debonded strand will pick up some additional strain, and associated stress. This creates an interesting distribution of stress along the length of a debonded strand. Additional stresses experienced by the debonded portion of a strand anchored at its end due to additional gravity loads acting on the beam will add to a "zero stress state" and not the effective prestress at the service limit state. So, with this important subtlety acknowledged, the author agrees with the discussers' point on this item. Finally, simply supported beams made continuous for live load creates a boundary condition that starts deviating from a simply supported beam end such as that depicted in Figure C5.7.3.5-1, where there is no moment reaction at the support.

Dr. Oguzhan Bayrak
University of Texas at Austin

CONTRIBUTING AUTHORS

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

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

Frederick Gottemoeller is an engineer and architect who specializes in the aesthetic aspects of bridges and highways. He is the author of *Bridgescape*.

Dr. Thomas Murphy is vice president, chief technical officer, and chairman of Modjeski and Masters Inc.

Dr. Jean A. Nehme is team leader of FHWA's Long-Term Infrastructure Performance programs and oversees the Long-Term Pavement Performance and Long-Term Bridge Performance programs.

Dr. Kyle A. Riding is an associate professor of civil and coastal engineering at the University of Florida.

Miroslav F. Vejvoda is the managing director of engineering and professional development at the Post-Tensioning Institute and the editor-in-chief of the *PTI Journal*.

CONCRETE CALENDAR 2020–2021

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

April 6, 2020
ASBI Grouting Certification Training Course
J.J. Pickle Research Campus
Austin, Tex.

April 22–24, 2020
DBIA Design-Build for Transportation & Aviation Conference
Hilton Anatole
Dallas, Tex.

April 22–24, 2020
PTI Level 1 & 2 Multistrand and Grouted PT Specialist Workshop
Dallas, Tex.

April 25, 2020
PTI Level 1 & 2 Multistrand and Grouted PT Inspector Workshop
Dallas, Tex.

May 3–6, 2020
PTI 2020 Convention & Expo
Hilton Miami Downtown
Miami, Fla.

May 27–29, 2020
PTI Level 1 & 2 Multistrand and Grouted PT Specialist Workshop
Gainesville, Fla.

May 30, 2020
PTI Level 1 & 2 Multistrand and Grouted PT Inspector Workshop
Gainesville, Fla.

June 1–4, 2020
AASHTO Committee on Bridges and Structures Annual Meeting
Branson Convention Center
Branson, Mo.

June 8–11, 2020
International Bridge Conference
David L. Lawrence Convention Center
Pittsburgh, Pa.

August 2–6, 2020
AASHTO Committee on Materials and Pavements Annual Meeting
InterContinental Hotel Miami
Miami, Fla.

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

September 23–26, 2020
PCI Committee Days
Loews Chicago O'Hare Hotel
Rosemont, Ill.

October 6–9, 2020
PTI Committee Days
JW Marriott Cancun
Cancun, Mexico

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

October 26–28, 2020
ASBI 32nd Annual Convention and Committee Meetings
Hyatt Regency Austin
Austin, Tex.

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

February 4–7, 2021
World of Concrete
Las Vegas Convention Center
Las Vegas, Nev.

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.

PERSPECTIVE

Making the Case for Resilient Design

by Evan Reis, U.S. Resiliency Council

My hometown, Half Moon Bay, Calif., is an unassuming and quaint hamlet south of San Francisco. People know it as the center of Prohibition era "rum running" on the northern California coast, and it claims to be the pumpkin capital of the world. Although many tourists and residents cross the Pilarcitos Creek Bridge as they drive or walk to Half Moon Bay's charming downtown, they likely do not comprehend the structure's historical significance. Built in 1900, it was the first reinforced concrete bridge erected in San Mateo County—and a very early attempt at prestressing, too. Functioning continuously for more than 100 years, the bridge remained in operation after the 1906 earthquake and following several flood events in which the creek overflowed its banks. To me, the bridge defines resilience!

Built in 1900, the Pilarcitos Creek Bridge in Half Moon Bay, Calif., was the first reinforced concrete bridge erected in San Mateo County. Photo: Burnell G. West.

Measuring Resilience

When we think of what makes a community resilient after a natural disaster, the measure is how quickly the services that a city provides to its residents (housing, employment, goods and services, emergency care, etc.) can be restored. Three components of the built environment are essential to recovery: the buildings in which these services are provided, the utilities that allow these buildings to function, and the transportation infrastructure that permits people to get to and from places where they live, work, and conduct business.

Transportation infrastructure (roads, bridges, airports, and railways) is often taken for granted. However, when the 2010 and 2011 earthquakes hit Christchurch, New Zealand, residents throughout the earthquake-affected region suffered as transportation routes were shut down because of infrastructure damage. In the United States, we only have to look to the Interstate 35 Mississippi River Bridge collapse in Minneapolis, Minn., in 2007, or bridge collapses caused by flooding and landslides in Cedar Rapids, Iowa, in 2009, and Big Sur, Calif., in 2017, to see that the performance of bridges is an essential link in the chain that allows a community to function during and after a natural or human-caused disaster (see **Fig. 1**).

USRC Mission and Goals

The U.S. Resiliency Council (USRC) was founded in 2011 as a nonprofit organization with the goal of "improving community resilience, one building at a time." Its mission, however, is broader than just looking at building structures. Resiliency must be the focus of every component of community infrastructure, including transportation. As we have seen in disasters within the United States just since Hurricane Katrina, a mere 15 years ago, true sustainability cannot be measured only by our impact on the environment; we must also consider the impact the environment has on us. In other words, resilience is different than and complements green design, and sustainability is about more than just eliminating carbon.

As the USRC has developed its building rating systems for describing the expected impacts of earthquakes, hurricanes, wildfires, and floods on a building, it has used leading engineering and scientific research on performance-based design to quantify performance not only in terms of safety but also in terms of repair costs and recovery time. The latter two metrics allow owners, government agencies, lenders, and insurers to calculate the return on investment of building more resiliently.

The USRC is aware that considerable research on the performance of bridge structures has been done and is ongoing. Quantifying the long-term performance of bridges will help define the benefits of building more resiliently. These benefits can be turned into incentives to cities and builders to use concrete technologies and other means to create durable, high-performing systems.

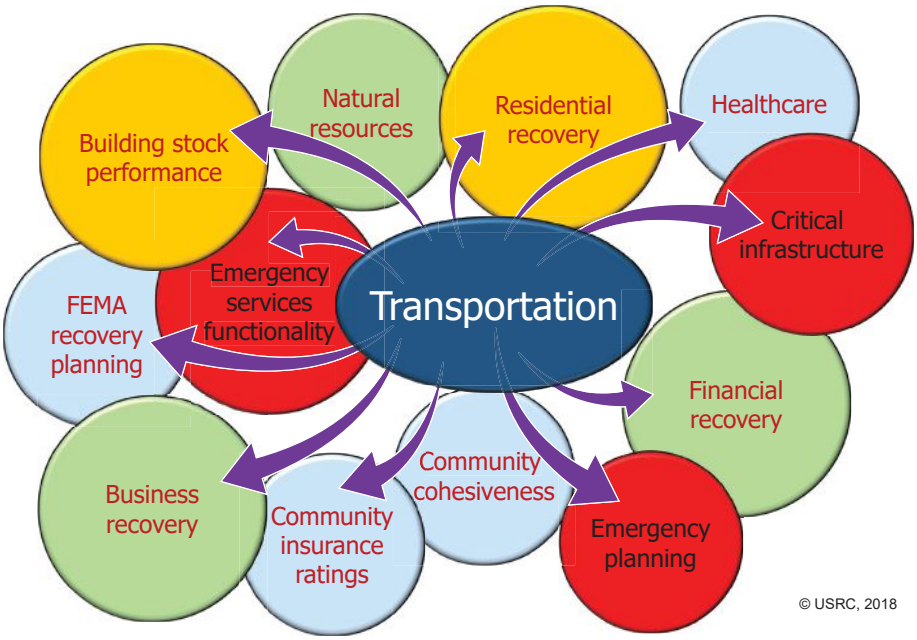


Figure 1. The central importance of transportation in resiliency. Figure: Evan Reis, U.S. Resiliency Council (www.usrc.org).

I worked as a consultant on a project for the renovation of a major freeway corridor in southern California that included the structural risk modeling of more than a dozen overpasses and bridges. My efforts were used by the contractor to obtain proper construction term insurance against the potential for earthquakes or flooding. The evaluation of resilience of the many concrete bridge structures had a direct impact on the amount of insurance that the builder was required to obtain, a cost shared in part by the transportation authority. This was a "concrete" example (forgive the pun) of how the resilience and durability of a specific structural system translated into cost savings.

The USRC encourages the bridge and transportation industries to invest in research that quantifies the performance of bridge structures subject to natural hazards. It is the first necessary step to make the case that using resilient materials such as concrete is a worthwhile investment that more than pays for itself in the short and long term by reducing the social and economic impacts of natural hazards on communities. The USRC would like to expand its rating system to include infrastructure and to make the strongest case for resilience as an essential component of sustainable design. In future articles, I will examine how the detailed process of resilience-based design can be employed by the bridge industry to promote the benefits of precast/prestressed concrete design.

Conclusion

As a structural engineer, I know that the people who work or live in the buildings I design are unlikely to reflect on how the structures are constructed to keep them safe and protect their livelihoods. Bridge engineers also understand that their work, although essential, is not widely appreciated. Most of the folks that visit Half Moon Bay or do their grocery shopping in town and cross the Pilarcitos Creek Bridge probably do not think about the engineers who designed it. And that's okay. But those of us responsible for making sure buildings don't collapse in an earthquake and bridges aren't swept away in a flood must consider the long-term performance of structures and weigh the costs and benefits of resilient design.

EDITOR'S NOTE

See the Perspective article on quantitative assessments of resilience and sustainability in the Spring 2019 Issue of ASPIRE®.

Evan Reis is a structural engineer licensed in California and executive director of the U.S. Resiliency Council.

PERSPECTIVE

Checking Our Work: Methods for Verifying Analyses of Structural Designs

by Dr. Thomas Murphy, Modjeski and Masters Inc.

For designers, there is often a strong incentive to keep things simple. Construction cost, design effort required, and the possibility of unforeseen problems push us to choose uncomplicated, easily understandable structural models for our designs. There are, however, other factors influencing design that can lead to more complex structural systems and behaviors. Aesthetics, geometric constraints, and the need to minimize material quantities are just some of the factors that can lead designers to select systems that are more difficult to analyze and more challenging to understand. The Bronco Bridge in Denver, Colo., is an example of a structure that required a complex analytical model (see photo).

Fortunately, our profession now has the tools to analyze and design virtually any structural system, no matter how complex. The challenge is making sure the analysis method and the corresponding results are adequately checked. This can be a daunting task, but there are methods that engineers can use to give them confidence that the complex behavior has been correctly captured in the results of the analysis.

The most direct approach is to perform an independent analysis and compare the results of the analyses. This can be time-consuming because you essentially perform the analysis twice and then attempt to resolve any differences in the results. There will almost always be differences between any two analyses due to differing assumptions and approaches, but this does not necessarily mean that one analysis is incorrect, or that a design based on the results of either would be deficient. The challenge for the designer in this situation is twofold: ensuring that the base assumptions made in both analyses are valid (that is, the same error was not made in both analyses), and resolving any differences in results until they are acceptably small.

The method of comparing analyses will vary depending on the details of

Designed to optimize the use of precast concrete, the Bronco Bridge in Denver, Colo., features a totally precast concrete rigid frame and is one example of a structural design that required a complex analytical model. Photo: Modjeski and Masters Inc.

the models (see "Element Modeling Options" for more information). If only beam- and frame-type elements are used, a comparison of deflections and key forces will likely suffice. However, if shell or solid elements are included, there is typically an additional step to go from the model output, typically stresses, to design values, typically forces and moments, with many opportunities for errors to creep into the process. A comparison of the final, integrated force and moment values would then be in order.

When performing an independent analysis, it is sometimes useful to intentionally vary the analytical approach from the base analysis as an additional check. For example, if the base analysis uses shell or solid elements, an independent analysis that uses beam elements can provide further assurances that the overall results are reasonable, even if the localized results may be less accurate due to the lower-order elements in the independent analysis.

When time and budget allow, the independent replication of results by a separate analysis is probably the best practice. However, it is not always appropriate, and other methods can be successfully and more efficiently applied.

Moving away from a full replication of the analysis, approximate methods of analysis can be used to provide what is often termed a "sanity check" on higher-order analytical results. Here, what is desired is a general check on the overall results without an independent evaluation of detailed results. In these cases, designers can take advantage of the rich library of approximate analysis techniques that were developed before the advent of the digital tools we heavily rely on today. There are many and varied examples of these techniques, including Pucher[1] and Homberg[2] charts used to determine moments in box-girder top flanges created by local loads and similar tables of moments in plates for a variety of fixity conditions from old U.S. Bureau of Reclamation publications, such as *Moments and Reactions for Rectangular Plates,*[3] or texts such as *Roark's Formulas for Stress and Strain.*[4] Concepts such as the elastic center method, historically used to simplify hand calculation of frames and arches, and the associated column analogy method of determining the location of the elastic center can still prove extremely useful in fashioning simpler models of complex behavior that can verify the solutions derived from far more involved and complicated numerical models.

There are few references that a bridge designer can consult to address the issue of verification of the results of complex models. One exception to this is the recently published Federal Highway Administration *Manual for Refined Analysis in Bridge Design and Evaluation*.[5] While primarily focused on typical structure types, the manual provides some guidance on the verification and validation of refined analyses, along with examples of how relatively complex models were verified by various methods.

The bridge design profession consists of multiple engineering generations with widely differing attitudes toward analysis.

Currently, the bridge design profession consists of multiple engineering generations with widely differing attitudes toward analysis. In a design office today, you might encounter engineers for whom a finite element analysis is considered something exotic, only to be used in the most extreme circumstances, as well as engineers who would create a shell element model to find the axial load in a member of a statically determinate truss. When developing an approach for the verification and validation of analytical models of complex structural behavior, it is often important to include multiple designers with a mix of engineering backgrounds. On the one hand, the engineers with the least experience may have the most expertise with finite element software packages and complex engineering issues. On the other hand, the engineers with the least numerical modeling expertise may be the most familiar with the shortcuts and approximate approaches that can be so valuable in providing a critical check on analytical results.

Element Modeling Options

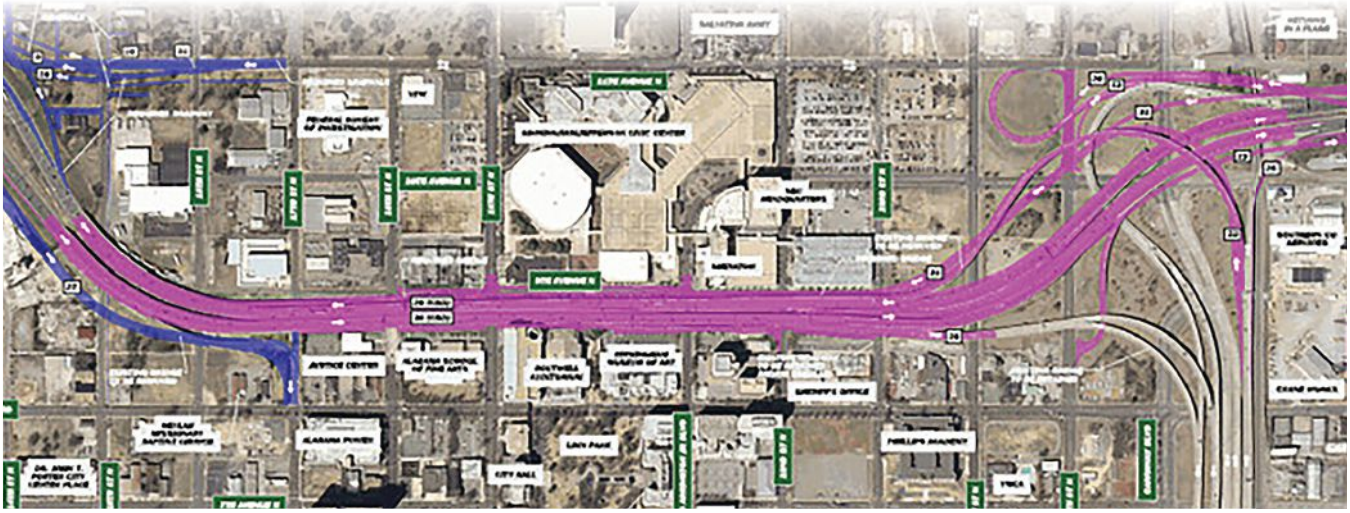
Engineers have many different choices for how to model a structure. Simpler structures can be modeled using beam or frame elements, which idealize behavior as one-way bending, whereas more complex structures require elements that can capture bending in two directions, such as plate elements. Curved plate elements are sometimes referred to as shells.

References

1. Pucher, A. 1977. *Einflussfelder elastischer Platten/Influence Surfaces of Elastic Plates*, 5th ed. Vienna, Austria: SpringerVerlag Wien.
2. Homberg, H., and W. Ropers. 1965. *Fahrbahnplatten mit verändlicher Dicke*,1st ed. Berlin, Germany: Springer-Verlag Berlin Heidelberg GmbH.
3. Moody, W.T. 1990. *Moments and Reactions for Rectangular Plates*. Engineering Monograph No. 27. Washington, DC: U.S. Department of the Interior Bureau of Reclamation. https://www.usbr.gov/tsc/techreferences/hydraulics_lab/pubs/EM/EM27.pdf.
4. Young, W., G. Budynas, and A. Sadegh. 2012. *Roark's Formulas for Stress and Strain*, 8th ed. New York, NY: McGraw-Hill.
5. Adams, A., N. Galindez, T. Hopper, T. Murphy, P. Ritchie, V. Storlie, J. Weisman. 2019. *Manual for Refined Analysis in Bridge Design and Evaluation*. Federal Highway Administration Report FHWA-HIF-18-046. McLean, VA: FHWA. https://www.fhwa.dot.gov/bridge/pubs/hif18046.pdf.

Alabama's Largest Transportation Project Renews Birmingham's Central Business District

by Lloyd Pitts, Volkert Inc., Eric Johnson, Corven Engineering Inc., and William (Tim) Colquett, Alabama Department of Transportation



Aerial view of the Phase III segmental bridge site, shown here in magenta. Five miles of segmental box girders with a total of 2316 segments were erected in only 217 days, and the bridges reopened to traffic ahead of schedule. Figure: Volkert Inc.

For nearly 50 years, millions of motorists have traveled the Interstate 59/Interstate 20 (I-59/I-20) from Meridian, Miss., to Birmingham, Ala. Once the interstates reach Birmingham, a city with a legacy built on steel and iron, they are now supported by elevated concrete structures over the downtown area.

Serving Birmingham's central business district (CBD), this artery has the highest rate of traffic flow in the state of Alabama. Built in 1973, the original elevated structures were designed for 80,000 vehicles per day. Current traffic exceeds 160,000 vehicles per day, and traffic forecasting suggests that this route may be subject to over 225,000 vehicles per day by 2035.

As a result of the age and heavy use of the facility, the bridge decks had begun to deteriorate, often requiring costly repairs that resulted in lane closures. The original bridges featured left-lane entrance and exit ramps, with minimal or no shoulders. Three of these ramps were considered outdated and forced motorists to make unanticipated and unsafe lane crossings, sometimes resulting in accidents that forced traffic slowdowns or stoppages (see the State article on Alabama in the Spring 2019 issue of *ASPIRE*®).

Rebuilding the Bridges

Considering the safety concerns, traffic congestion, and maintenance issues, the Alabama Department of Transportation

profile

INTERSTATE 59/INTERSTATE 20 BIRMINGHAM CENTRAL BUSINESS DISTRICT BRIDGES / BIRMINGHAM, ALABAMA

BRIDGE DESIGN ENGINEERS: Volkert Inc., Mobile, Ala. (prime consultant, substructure and foundations); Corven Engineering Inc., Tallahassee, Fla. (segmental superstructure)

CONTRACTOR'S CONSTRUCTION ENGINEERS: McNary-Bergeron & Associates, Broomfield, Colo.; Infrastructure Consulting & Engineering, Columbia, S.C.

PRIME CONTRACTORS: Johnson Bros. Corp., a Southland Company, and Oscar Renda Contracting Inc. Joint Venture, Roanoke, Tex.

(ALDOT) deemed this infrastructure functionally obsolete and determined that immediate action was required. Multiple options were considered, including burying the interstates, rerouting the corridor, or simply redecking the existing bridges. Some of these options were projected to take 20 years and billions of dollars to complete, and yet they would not adequately solve the corridor's problems.

ALDOT determined that the best course of action would be to replace some CBD bridges and widen others, with the interstates completely shut down during construction. Compared to long-term construction phasing, this plan required the shortest turnaround time and thus would cause the least disruption to the traveling public.

ALDOT engaged a single consulting firm team to provide the engineering design and construction inspection services for the $710 million megaproject. The project, with 36 bridge sites for either new bridges or widening of existing bridges, was separated into four bid packages—Phases A, I, II, and III. The concrete bridges for the project totaled 14.3 miles of precast, prestressed concrete girders and 5 miles of segmental box girders. The $440 million Phase III project included the segmental box girder bridges.

Work on the Phase III CBD bridge replacement project was performed in concert with many other improvements in the CBD region to improve traffic movements, replace or modernize existing facilities, and revitalize the downtown experience for the citizens and visitors of Birmingham.

Substructure Challenges and Solutions

To minimize the impact of the project on downtown Birmingham, a diverse

A typical span after erection of the four lines of box-girder segments. A longitudinal closure pour about 3 ft 6 in. in width was used to join the segments for each of the two bridges. The completed eastbound and westbound structures are separated by a 6 in. gap. Photo: Volkert Inc.

Falsework towers were used to support segments during erection. Photo: Volkert Inc.

ALABAMA DEPARTMENT OF TRANSPORTATION, OWNER

POST-TENSIONING CONTRACTOR: Structural Technologies, Columbia, Md.

CASTING MACHINES AND ERECTION SYSTEM: Structural Technologies, Columbia, Md.

BRIDGE DESCRIPTION: 6500-ft-long precast concrete segmental box-girder bridges with 2316 segments total

STRUCTURAL COMPONENTS: Five miles of precast concrete segmental box girders, 172 spans, and 2316 segments, precast concrete piers

SEGMENTAL BRIDGE CONSTRUCTION COST: $195 million

A precast concrete pier cap with fluted aesthetic detailing is erected on a column element. Segments are connected using grouted couplers. Photo: Volkert Inc.

business hub with a bustling food and art scene, only 14 months of full interstate closure were allowed for construction; also, access to certain CBD public venues was required throughout the project. Additional challenges included limited right-of-way, utility conflicts, varying geotechnical conditions along the project corridor, and limited vertical clearance under the existing structures to perform foundation work before the interstates were closed.

Aesthetics were a priority for ALDOT, so the design team decided to use single-column piers under each segmental girder line. The piers are accented with vertical fluted lines in the near and far faces, and the pier caps flare out at the top to complement the sloping lines of the segmental girders. The contractor elected to precast the piers as two separate pieces, columns and caps, for faster construction. The reinforcement for the pier sections was made continuous with grouted couplers. This allowed the piers to be constructed efficiently once the demolition and removal operations for the existing structures were completed.

The first two-thirds of the project had sound rock for foundation design. Near 22nd Street North, there was a

AESTHETICS **COMMENTARY**

by Frederick Gottemoeller

Faced with a request for a viaduct that would "revitalize the downtown experience for the citizens and visitors of Birmingham," the project's designers thought creatively about the appearance of the space below the structure. Such spaces are often dark and uninviting, filled with haphazardly parked cars and drifting waste paper, depressing the activities around them. Improving the appearance of such a space requires conceiving of it as a huge outdoor "room," with the superstructure as its ceiling and the bridge piers articulating the room-like impression.

The attractiveness of this "room" depends, first of all, on long, uninterrupted sight lines in both the transverse and longitudinal directions, so that the whole area can be seen and understood at once, so that it can be organized for uses beyond parking, such as farmers markets and art fairs, and so that there are few opportunities for concealment. The concrete box girders contribute to this goal by minimizing the number of pier legs both longitudinally (by allowing relatively long spans) and transversely (by requiring only four pier legs per pier line). The thin piers also avoid a problem that sometimes results when designers are asked to provide a structure with architectural grandeur: They attempt to do so with physical mass and "architectural" detail. The result can be an agglomeration of massive piers with nonstructural decorative details. Thus, an individual looking along the bridge sees the piers line up one behind the other, visually filling the "room" with concrete. In contrast, the thinness of the Birmingham piers keeps the long views open, and the "room" inviting. The piers' only architectural details are the closely spaced vertical grooves that visually reinforce their thin appearance.

The concrete box girders also keep the longitudinal views simple. The sight lines are not blocked by transverse pier caps, and there are no braces or diaphragms to catch the eye. The wide spacing between box webs means that light can reach to the underside of the deck slab, and the whole underside of the bridge stays bright. Finally, a reflective white coating on the underside of the structure keeps light bouncing around the "room," meaning the space is brighter during the day and easier to light at night.

It is heartening to see this high level of aesthetic quality achieved within the discipline of accelerated bridge construction. Birmingham has met its schedule while achieving an "aesthetically pleasing area for public events"—all at the same time.

Erection of a precast concrete box-girder segment on temporary support towers for the first span of the central business district bridges. Photo: Volkert Inc.

geological shift, and the last one-third of the project had karst limestone conditions. In areas with sound rock, the footings used either drilled shafts or micropiles. In karst conditions, the foundations used steel H-piles with driving shoes. The span layouts for the project considered existing foundation locations and minimized conflicts for the new substructure locations.

Prior to the closure of I-59/I-20, the contractor built as many of the footings as possible. Because of the limited vertical clearance, low-overhead equipment was used to install the drilled shafts, micropiles, and steel H-piles. The footings were then covered with fill to protect them during the demolition of the existing overhead structures.

Creating the Superstructure

When ALDOT decided to allow I-59/I-20 to be closed for 14 months of construction, the designers had to find a design solution that could be achieved within this tight schedule. The contractor was allowed to fully close the interstates for 14 months without penalty, and would be awarded bonuses of $250,000 per day for finishing early, with total bonuses capped at $15 million. However, at the end of the 14-month window, penalties would be assessed at a rate of $250,000 per day, with no cap. These time constraints led the design team to choose a precast concrete segmental superstructure for Phase III of the project.

Because of the limited right-of-way and the need to preserve existing buildings along the project corridor, the replacement bridges were built within the same footprint as the original bridges. The eastbound and westbound structures are each approximately 6500 ft in length and are separated by a 6 in. gap.

Each bridge comprises twin precast concrete segmental box girders, and every mainline bridge includes entrance and exit ramps. Along the transition regions for these ramps, the mainline structures comprise three segmental box girders.

With a total deck area of over 1 million square feet, the bridges are composed of 172 spans, with nominal lengths of 165 ft. There are 2316 precast concrete segments, with typical segment lengths between 11 ft 6 in. and 12 ft 6 in. The overall box-girder segment depth is 9 ft, with an additional ½ in. of sacrificial deck surface that was milled to achieve a smooth deck surface after segment erection.

The lane configurations for the bridges vary along the project corridor from four to six lanes, resulting in a variation of out-to-out bridge width. To accommodate this variation in overall bridge width, the individual box girders were categorized into three groups: 33 ft 6 in. wide for the four-lane configuration; 39 ft 6 in. wide for the five-lane configuration; and 45 ft 6 in. wide for the six-lane configuration. Individual box girders were joined with a longitudinal deck closure strip approximately 3 ft 6 in. wide to make up the total deck width for one bridge. To facilitate the precasting operations, all box girders used a constant core dimension with only the wing lengths varying.

The top slab of each box-girder segment was transversely post-tensioned with four-strand tendons that anchor at the end of each segment wing. To accommodate the variation in the width of the top slab, two transverse post-tensioning spacings were used: four tendons per segment for box-girder widths of 39 ft 6 in. or less, and five tendons per segment for box-girder widths greater than 39 ft 6 in.

The longitudinal post-tensioning in each box-girder span consists of external draped tendons. Eight permanent tendons (four per web) were anchored at the ends of the spans in the pier segment or expansion-joint segment diaphragms. The longitudinal post-tensioning tendon sizes vary depending on demand, with hardware sized to accommodate a maximum of 22 strands per tendon. Wider sections with longer span lengths feature one additional 12-strand tendon per web, anchored

The area beneath the completed mainline spans after painting. City officials intend to use this area for public events. Photo: Johnson Bros.

at the deviation diaphragms. Each span also includes hardware to accommodate two 12-strand future post-tensioning tendons. Project-specific specifications for the installation and grouting of the post-tensioning tendons included many aspects of the *PTI/ASBI M50.3-12 Guide Specification for Grouted Post-Tensioning*.[1] (A newer version of this specification is now available—see the Concrete Bridge Technology article in the Summer 2019 issue of *ASPIRE*.)

Advantages of the Precast Concrete Segmental Design

The precast concrete segmental bridge design was chosen for several reasons stemming from ALDOT's decision to completely shut down I-59/I-20 during construction. In particular, the selected design minimized the time of interstate closure by using off-site fabrication and rapid construction methods.

The casting yard was located approximately 4 miles from the project, which allowed efficient transport of the precast concrete segments to the construction site for placement. This expedited the construction process compared to the traditional methods for building bridges. By the time the interstates were closed to traffic, approximately 1000 of the 2316 segments were already cast and ready for erection.

Segment Erection

The original design assumed a traditional span-by-span erection method, with all precast concrete segments supported by longitudinal erection trusses. The contractor elected to use a unique technique, for which the design was verified, erecting each precast concrete segment within a span on individual shoring towers. This erection technique meant the contractor could work on as many as eight spans at any given time and facilitated nonlinear construction. The method proved to be successful, with all 2316 segments (172 spans) erected in only 217 days. (For more details on this technique, see the Concrete Bridge Technology article on page 30 of this issue of *ASPIRE*.)

The Finish Line

The challenges and time constraints of this project proved how beneficial precast concrete segmental bridge design and construction can be. The I-59/I-20 CBD bridges reopened to traffic ahead of schedule on January 17, 2020, with a closure of only 12 months compared with the 14 months allowed in the contract.

Reference

1. Post-Tensioning Institute (PTI) and American Segmental Bridge Institute (ASBI). 2012. *PTI/ASBI M50.3-12: Guide Specification for Grouted Post-Tensioning*. Farmington Hills, MI: PTI.

Lloyd Pitts is vice president and chief bridge engineer with Volkert Inc. in Mobile, Ala., Eric Johnson is a bridge engineer with Corven Engineering Inc. in Tallahassee, Fla., and William (Tim) Colquett is the state bridge engineer for the Alabama Department of Transportation in Montgomery.

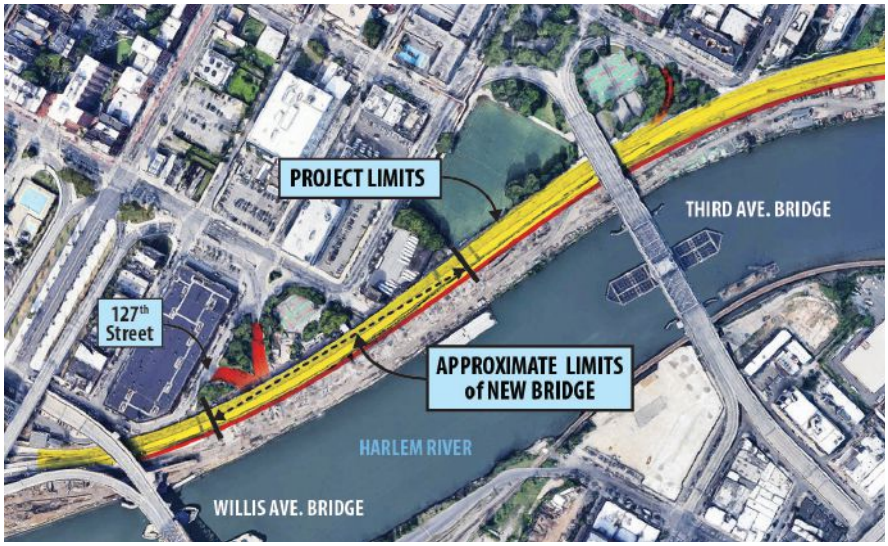
Harlem River Drive Bridge over East 127th Street

by Silvio Garcia, Hardesty & Hanover LLC

Harlem River Drive is a limited-access passenger vehicle highway adjacent to the Harlem River in the New York City borough of Manhattan. The section of Harlem River Drive included in this project extends south to north from the Willis Avenue Bridge to the Park Avenue entrance ramp, a total length of approximately 4500 ft. As part of this project, the 676-ft-long Harlem River Drive Bridge over East 127th Street was reconstructed to improve the operation of this segment of highway and address deficiencies of a structure that carries mainline traffic over the East 127th Street on- and off-ramps.

A Perilous Stretch of Highway

The existing bridge carried three lanes of southbound traffic and two lanes of northbound traffic plus a wide, striped shoulder on the northbound side. The 11-span structure consisted of seven steel multigirder spans plus two concrete cellular-type spans at each end. The seven main spans were supported by steel columns on concrete footings and founded on concrete-filled steel pipe piles socketed in rock. The cellular spans were composed of a structural reinforced concrete slab supported by reinforced concrete girders and retaining walls in the longitudinal direction founded on steel H-piles driven to bedrock.



Aerial view of project limits. The yellow area indicates the project limits, which extended beyond the bridge replacement to accommodate the updated vertical profile and safe traffic patterns during the staged construction. The red areas indicate on- and off-ramps that were impacted by the project and therefore included in the project limits. Photo: Google / Hardesty & Hanover.

This section of Harlem River Drive was categorized as one of the worst segments of highway in New York state, based on the high traffic accident rates and severe congestion, particularly in the southbound direction. The causes of traffic accidents in this area included a weaving condition on southbound Harlem River Drive at the exit to Second Avenue, and substandard stopping sight distance on the bridge. The new bridge addresses these deficiencies with the addition of a left lane exit to Second Avenue, thereby eliminating the weaving condition, and modifications to the vertical profile on the bridge and approaches to increase the stopping sight distance.

New Southbound and Northbound Bridges

The single existing structure was replaced with two adjacent bridges, one for southbound traffic and one

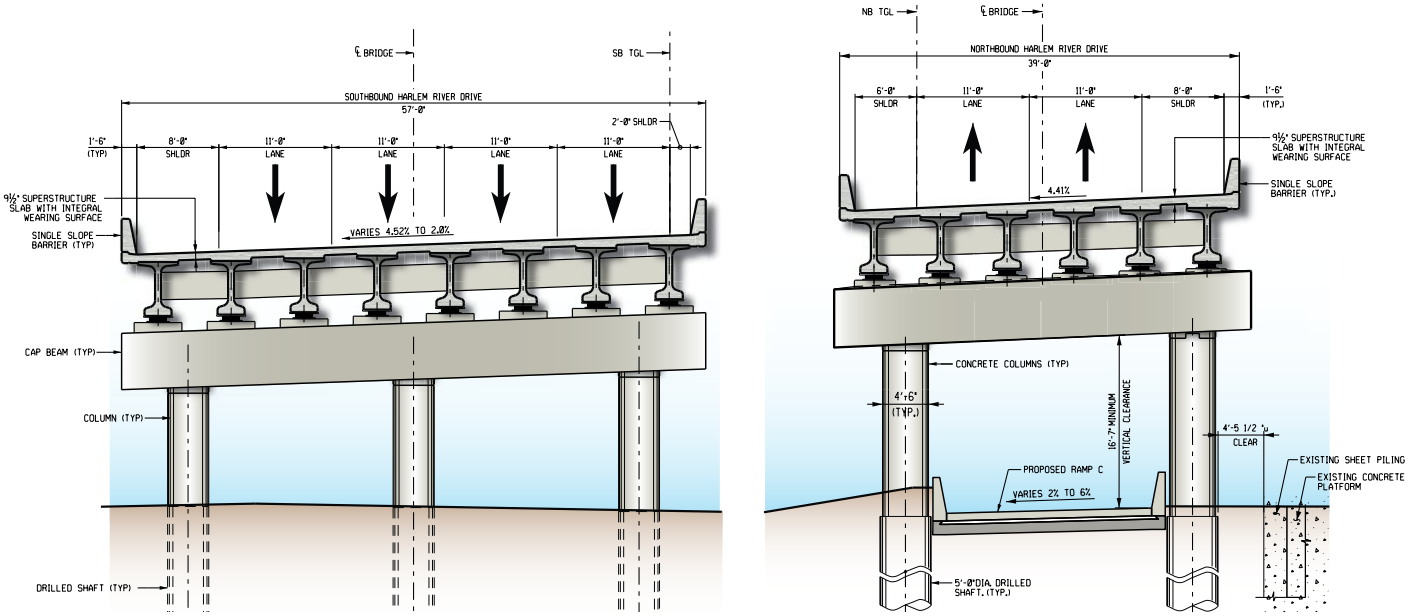
profile

HARLEM RIVER DRIVE OVER EAST 127TH STREET / NEW YORK, NEW YORK

BRIDGE DESIGN ENGINEER: Hardesty & Hanover LLC, New York, N.Y.

PRIME CONTRACTOR: Defoe Corporation, Mount Vernon, N.Y.

PRECASTER: Coastal Precast Systems, Chesapeake, Va.—a PCI-certified producer



Typical cross sections of the new (a) southbound bridge and (b) northbound bridge over an on-ramp. Figure: Hardesty & Hanover.

for northbound traffic. Each bridge is approximately 990 ft long. Two bridges were required to accommodate the new left-hand exit from southbound Harlem River Drive to Second Avenue. The southbound bridge carries four lanes of traffic, and the northbound bridge carries two lanes of traffic. Both bridges include wide shoulders that ease congestion in the case of emergencies.

The new southbound and northbound bridges consist of nine and ten spans, respectively; and the spans range in length from 65 to 132 ft. The bridges are composed of prestressed concrete bulb-tee beams (PCEF-63). The beams are composite with a 9.5-in.-thick reinforced concrete deck and are supported on multicolumn concrete piers founded on 4- and 5-ft-diameter drilled shafts. The prestressed concrete beams have a design concrete compressive strength of 10 ksi with 270-ksi, 0.6-in.-diameter low-relaxation strands. The piers and drilled shafts have a design concrete compressive strength of 4 ksi. All elements, including the beams, use 60-ksi epoxy-coated reinforcing bars.

The alignment for the southbound bridge is on a compound horizontal curve with radii of 1885 and 2828 ft followed by a tangent. The northbound alignment is on a compound horizontal curve with radii of 2920 and 1937 ft followed by a tangent. The bridge profiles have a 5% maximum grade and the decks are superelevated, with the cross slopes varying from 2.00% to 4.52%.

Why Precast Concrete Bulb Tees?

A structure justification study was performed during the preliminary design to determine the best superstructure/substructure combination for the replacement bridges. The study considered both steel and concrete alternatives, including several prestressed concrete girder shapes, and evaluated them for future maintenance requirements, construction duration, construction cost, life-cycle cost, geometry needs, staging needs, and other parameters.

The life-cycle cost analysis assumed a 4% discount rate and a 75-year service life. For the estimated future costs, a 3% inflation rate was used. Periodic steel painting, joint replacement, and bearing replacement were considered. A full bridge rehabilitation including deck replacement was assumed after 45 years of service.

The proposed bridges would be located adjacent to an existing park that includes a large mural, "Crack Is Wack," painted by the artist Keith Haring, as well as the planned Harlem River Waterfront Park; therefore, it was important to provide a structure that blended into its surroundings. Accessibility for future painting of the bridges would have been problematic given their location. Because the prestressed concrete superstructures would not require painting, both maintenance costs and potential for future park disruptions were minimized.

Prestressed concrete bulb tees, specifically the PCEF-63, were found to be the best alternative for the superstructure because they provided the lowest life-cycle cost of all the superstructure types analyzed, eliminated the need for painting, and offered a neutral, nonintrusive appearance. Also,

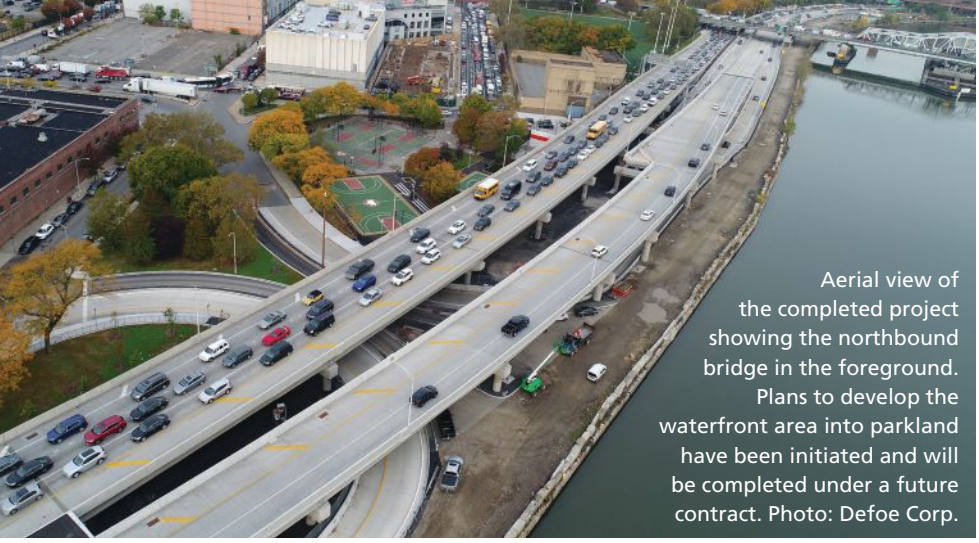
NEW YORK CITY DEPARTMENT OF TRANSPORTATION, OWNER

BRIDGE DESCRIPTION: Twin 990-ft-long bridges on a horizontally curved alignment with precast, prestressed concrete bulb-tee beams placed on chords, a cast-in-place 9.5-in.-thick concrete deck, and multicolumn concrete piers supported on 4- and 5-ft-diameter drilled shafts

STRUCTURAL COMPONENTS: 139 PCEF-63 bulb-tee beams, 9.5-in.-thick cast-in-place concrete deck, cast-in-place concrete pier caps, columns, and drilled shafts

BRIDGE CONSTRUCTION COST: $126.9 million

AWARD: 2020 Diamond ACEC New York Engineering Excellence Award



Aerial view of the completed project showing the northbound bridge in the foreground. Plans to develop the waterfront area into parkland have been initiated and will be completed under a future contract. Photo: Defoe Corp.

In stage 2 of the project, a portion of the southbound bridge was constructed. The close proximity of the new construction (left) to the existing structure (right) required extensive coordination and surveying throughout the project. Photo: Hardesty & Hanover.

the bulb-tee shape best accommodated construction staging and vertical clearance requirements.

Substructure Selection

For any project in an urban environment, utilities play a large role in the selection of the substructure and foundation. For the Harlem River Drive project, multicolumn concrete piers supported on drilled shafts were the preferred substructure/foundation option due to the numerous underground utilities that needed to be maintained and protected during construction. Underground utilities within the project limits included electrical lines, intelligent traffic system lines, communications lines, water mains, and stormwater systems.

Among the underground utilities within the project limits are approximately fifteen 270-kV electrical submarine cables, owned by Con Edison, which supply electricity to Manhattan's East Side. To minimize the risk of damaging the lines, the following requirements were placed on the contractor for all shaft and substructure excavation work adjacent to these high-voltage facilities:

- Con Edison needed to locate the facilities prior to any substructure work. This entailed hand digging or test pitting to verify the location of shallow buried facilities and a gyroscope survey for the submarine cable river crossing. The survey established the limits of the zones in which no drilled shafts or driven piles could be installed.
- Vibration velocity was limited to less than 0.5 ft/s and was to be monitored via vibration monitoring.
- Outage time was restricted, with no summer outages permitted.
- Minimum clearance of 11 ft was required between drilled shafts and submarine cables.

Construction in a High-Traffic Urban Environment

Northbound and southbound lanes of Harlem River Drive have annual average daily traffic in excess of 28,000 and 68,000 vehicles, respectively. Because of the high traffic volumes and the highway's importance as a main route into and through Manhattan, all traffic lanes and the major on- and off-ramps needed to be maintained throughout construction.

The space limitations imposed by a dense urban environment and the need to maintain all lanes required that the project be built in five stages, from west to east.

In stage 3 of the project, construction of the southbound bridge continued, while northbound traffic was maintained on the at-grade temporary roadway. Note that the prestressed concrete bulb tees are set on chords of the horizontal curves. Photo: Defoe Corp.

The new southbound bridge was initially opened for traffic in both directions, with its shoulders used as temporary lanes. The temporary roadway along the river was removed and the northbound bridge was constructed. Photo: Defoe Corp.

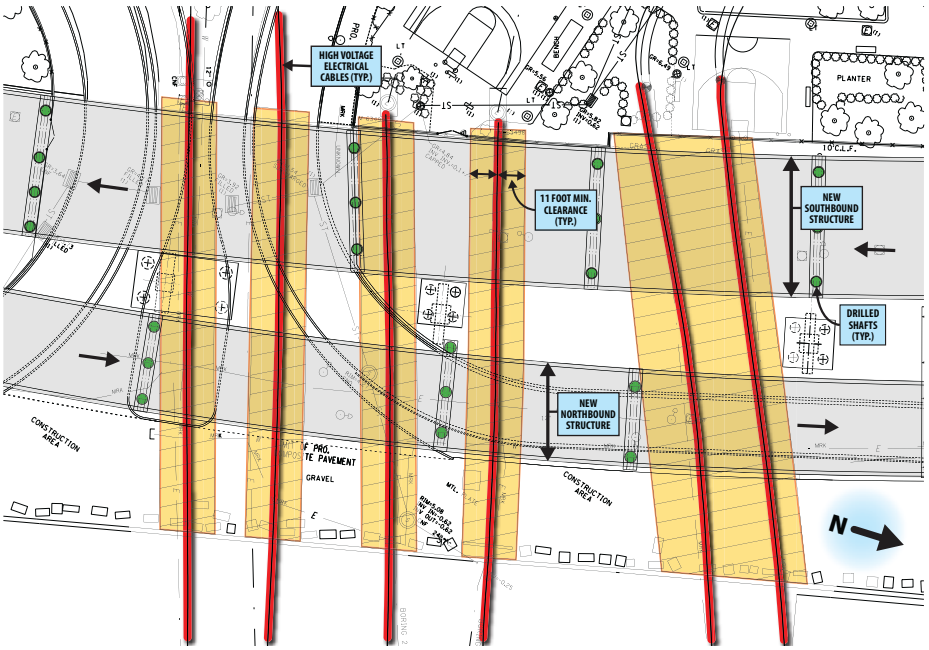
The prestressed concrete bulb tees were transported by barge from the producer to the site and lifted directly from the barge onto the new piers. Photo: Defoe Corp.

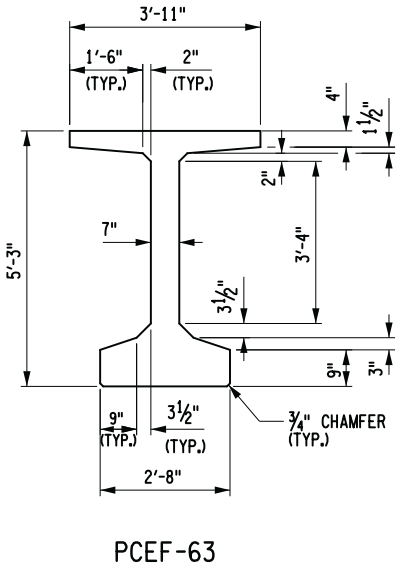
During stage 1, a temporary at-grade roadway was constructed adjacent to the river. Northbound traffic was moved to the temporary roadway and the southbound traffic shifted to the east side of the existing structure. This accommodated the demolition of the first half of the existing bridge and construction of approximately two-thirds of the new southbound bridge (stage 2).

In stage 3, southbound traffic was moved onto the new structure. The remainder of the southbound bridge was built during this stage, which required additional coordination and safety precautions as construction occurred between active traffic on the new southbound bridge and the temporary roadway carrying northbound traffic along the river. The prestressed concrete bulb tees were transported by barge from the producer to the site and erected directly from the barge onto the new piers. Intermittent night closures of northbound traffic lanes were approved by local authorities to allow the bulb tees to be lifted from the barge, over the temporary roadway, and into place.

Once stage 3 was completed, all traffic was shifted to the new southbound bridge, with the wide shoulders used to accommodate temporary lanes during construction. In stage 4, the temporary roadway was removed, and the northbound bridge was constructed. During the fifth and final stage, tie-ins and approaches at each end were completed.

Drilled shafts were strategically placed outside the required minimum 11 ft clearance limit from each of the Con Edison submarine high-voltage cables. Figure: Hardesty & Hanover.





The cross section of the 63-in.-deep prestressed concrete bulb tee (PCEF-63) from New York State Department of Transportation (NYSDOT) standards.[1] Based on a justification study, the PCEF-63 was the best alternative and was used for all spans. Span lengths ranged from 65 to 132 ft. Figure: NYSDOT.

Conclusion

The project was substantially complete and open to traffic in May 2019. Continuous communication and coordination among all parties ensured the success of this project, which provides a safer facility with minimal required maintenance of the new bridges. The new structures will provide at-grade access to the new waterfront park and a direct connection between the Robert F. Kennedy Bridge and northbound Harlem River Drive via a ramp currently being constructed by the Triborough Bridge and Tunnel Authority.

Reference

1. New York State Department of Transportation. 2017. "Prestressed Concrete PCEF and AASHTO I-Beam Typical Sections." https://www.dot.ny.gov/main/business-center/engineering/cadd-info/bridge-details-sheets-repostitory-usc/BD-PC14E.pdf. Accessed January 11, 2020.

Silvio Garcia is a senior associate with Hardesty & Hanover LLC in New York, N.Y.

GENESEE AVENUE VIADUCT

The First Curved Precast Concrete Spliced U-Girder Light-Rail Transit Viaduct in California

by Ali Seyedmadani and Pooya Haddadi, WSP

The Mid-Coast Corridor Transit Project is a 10.8-mile-long, $2.2 billion light-rail transit (LRT) link connecting downtown San Diego, Calif., to the La Jolla, Calif., business center on Genesee Avenue. This transit line uses the existing Los Angeles–San Diego–San Luis Obispo (LOSSAN) rail corridor in the southern portion of the line. It crosses over the San Diego River in parallel with Interstate 5 (I-5), then crosses over I-5 in the vicinity of the San Diego VA Medical Center, runs near the University of California San Diego (UCSD) campus, crosses over I-5 again, and terminates in the median of Genesee Avenue near Westfield UTC mall in La Jolla.

The entire Mid-Coast Corridor Transit Project includes 12 bridges, four aerial stations with side platforms, three viaducts (each over 1 mile long), and more than 70 retaining walls of various types. As the project navigates through heavily urbanized and less-urban areas, there are also significant changes in elevations. Soil conditions vary, and there may be perched water tables within foundation footprints. In addition, because Southern California is in a high-seismic zone and project alignment crosses over a seismic fault, special bridge design elements were required for potential ground movement due to fault rupture.

Project cost (guaranteed maximum price) and schedule were the major driving forces during delivery of the Mid-Coast Transit Project. When it was being planned, the owner, San Diego Association of Governments (SANDAG), was executing and overseeing the delivery of several transit projects simultaneously, and the association decided that the construction manager/general contractor (CM/GC) alternative delivery method would be the right approach for this large, complex project.

Project Delivery

When SANDAG selected the CM/GC for the project, a review of the project schedule and cost was undertaken. Through this process, it was determined that changing two bridges from the preliminary design of cast-in-place (CIP) post-tensioned (PT) box-girder designs to precast concrete girder structures would save time and minimize construction impact on the communities. The 1758-ft-long Rose Creek LRT Overhead Bridge that crosses the alignment from the east side of the existing LOSSAN track to the west side and the 5726-ft-long Genesee Avenue Viaduct were identified for redesign as precast concrete girder bridges.

Genesee Avenue is the business, shopping, and office center of La Jolla and a major arterial street. For the Genesee Avenue Viaduct, the contractor concluded that a precast concrete girder bridge would eliminate several months

Mid-Coast Transit Corridor alignment near the San Diego VA Medical Center. Photo: Mid-Coast Transit Constructors.

profile

GENESEE AVENUE VIADUCT / LA JOLLA, CALIFORNIA

BRIDGE DESIGN ENGINEER: WSP, Orange and Sacramento, Calif.

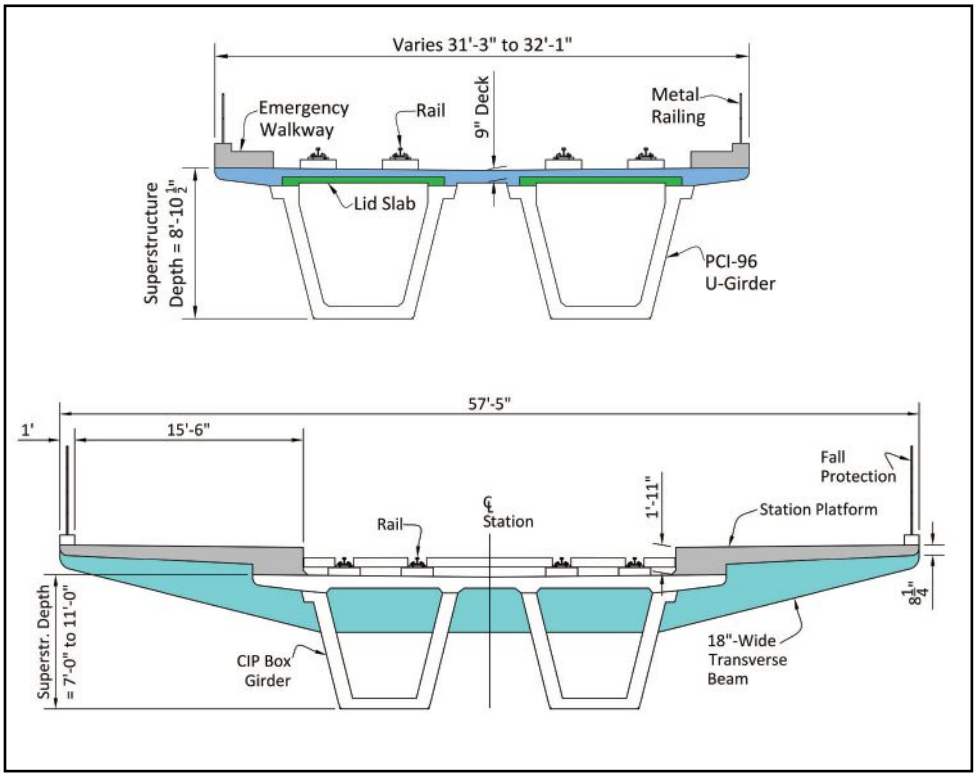
CONSTRUCTION ENGINEER: PGH WONG Consulting Engineers/Kleinfelder, San Diego, Calif.

PRIME CONTRACTOR: Stacy and Witbeck/Herzog/Skanska–Mid-Coast Transit Constructors (MCTC), San Diego, Calif.

PRECASTER: Oldcastle Infrastructure, Perris, Calif.—a PCI-certified producer

POST-TENSIONING CONTRACTOR: DYWIDAG Systems International (DSI), Long Beach, Calif.

Genesee Avenue Viaduct alignment near the University of California San Diego campus. Photo: Mid-Coast Transit Constructors.



Typical Genessee Avenue Viaduct section (top), and typical section at stations (bottom). Figure: WSP.

of nighttime construction, reducing the project's impact on surrounding hotels, businesses, residences, and the traveling public without compromising aesthetic appearance.

Viaduct Configuration

The preliminary design of the Genesee Avenue Viaduct using CIP PT box girders had identified column and foundation locations along the median of the arterial and had set the span arrangement based on critical intersections and driveway crossings. Early utility relocation had begun, and the early construction package was underway. Therefore, the superstructure span layout was somewhat set when the design was changed to a precast concrete U-girder system.

However, the precast concrete girder layout for the 5726-ft-long bridge eliminated one span from the CIP configuration, resulting in 35 spans divided into 12 frames (units). Eleven frames have three spans and one frame has two spans. The longest frame is 590 ft and the longest span is 225 ft; most spans are 180 ft long and have three 60-ft-long girder segments. The radius of the curved viaduct is at least 990 ft, except in one section where the radius is 500 ft.

There are three types of frames on the viaduct. The first type is a precast concrete frame with three precast concrete spliced girder segments per span per girder line, integral at the bents. The second type is a hybrid of precast concrete spliced girders and CIP hammerhead pier tables to extend the typical 180-ft-long spans to accommodate the crossing over the La Jolla Village Drive intersection. The third type is a traditional CIP PT concrete box girder; this was used for the two aerial stations with side platforms. There are five in-span internal hinges on the viaduct, one at each end of the two station units and one at the interface with the UCSD Viaduct. These in-span hinges will isolate the seismic displacement of the CIP frames from the remaining frames of the viaduct during extreme seismic events.

The Genesee Avenue Viaduct's typical two U-girder-line section is 32 ft 1 in. wide, except at the stations, where sections are widened to 57 ft 5 in. The

SAN DIEGO ASSOCIATION OF GOVERNMENTS, OWNER

OTHER MATERIAL SUPPLIERS: Cosmic Inc., Athens, Tex. (spherical bearings); GERDAU, San Diego, Calif. (reinforcing steel fabricator)

BRIDGE DESCRIPTION: 5726-ft-long light-rail transit bridge consisting of 35 spans in 12 frames with two lines of straight or curved precast concrete spliced girders

STRUCTURAL COMPONENTS: One hundred forty modified PCI U-96 precast concrete girders (48 were curved), maximum of 60 ft in length and 100 tons maximum haul weight, supported on cast-in-place pier caps, 7- to 9-ft-diameter columns, and 9- to 10-ft-diameter drilled shafts

BRIDGE CONSTRUCTION COST: \$60 million (\$320/ft^2), including station platforms

Typical precast concrete straight U-girder segment. Photo: WSP.

The precast producer used two straight and one adjustable-curve PCI U-96 girder forms. Photo: WSP.

curved precast concrete section is the same size as the typical section and uses PCI U-96 girders with 10 in. webs and a 9-in.-thick bottom slab. To control torsional forces induced in the curved, open-section girders, 4¼-in.-thick CIP lid slabs were constructed in the fabrication plant to close the top of the U-girders. There are four $3\frac{13}{16}$-in.-diameter ducts with nineteen 0.6-in.-diameter strands in each web; four ducts were used for splicing, and four ducts were used for PT continuity. There are also four $3\frac{1}{8}$-in.-diameter ducts with twelve 0.6-in.-diameter strands at the curved section in the bottom slab of the girder for self-weight post-tensioning; pretensioned strand was used for the straight girder sections.

Each precast concrete U-girder was designed to carry one LRT track loading in addition to dead loads. The LRT loading is approximately 50% heavier than the standard highway truck loading. Because this viaduct is close to the ocean, designers chose to have no tension in the girders to improve the structure's durability and service life. The minimum concrete design strengths were 6000 psi at transfer and 8500 psi at 28 days.

Girder hauling costs in Southern California increase significantly when segment weight is over 100 tons. This weight includes any temporary stiffening braces or lid slabs in curved sections of bridge segments. Lid slabs are used to stiffen the open U-girder section during the hauling and erection operations and to serve as a stay-in-place form for the deck.

On the Genesee Avenue Viaduct project, one strategy used to keep the haul weight of segments below the 100-ton limit was to not include the post-tensioning anchorage hardware and concrete section at the girder end diaphragm. This resulted in the construction of a CIP closure section of about 7 to 9 ft prior to post-tensioning. The CIP section also provided tolerance for forming the tendon deviators and girder flares in the end zone.

Substructure Configuration

The seismic design of the Genesee Avenue Viaduct required similar substructure stiffness in each frame. In addition, it required that adjacent frames have a similar fundamental period of vibration in the longitudinal direction. Therefore, the designers had to adjust column and drilled-shaft sizes as column height and span length varied.

Girder segment with blockout for closure pour and post-tensioning anchorage hardware. This photo also shows both pins in the pier cap. Photo: WSP.

The viaduct substructure consists of pier caps supported by single columns with a drilled-shaft foundation. The column sizes vary from 7 to 9 ft in diameter, and the drilled shafts are 9 to 10 ft in diameter. The pier caps are 7 to 9 ft in the longitudinal direction, approximately 22 ft wide, and vary in depth from 4 ft at the outside edge to 8.5 ft at the column face. The shape of the piers mimicked the shape of the other CIP box-girder viaduct piers except the top of the pier was enlarged to accommodate a seat for the U-girders.

The superstructure/substructure connection is a pin at the interior pier caps and an expansion joint at the exterior pier caps of each frame. There are two types of pins at the pier caps. One type was designed using reinforcing bars to engage during service and post-tensioning operations; the second type has much larger pins and sockets that are designed to remain elastic during major seismic events, allowing the superstructure to rotate. All pier caps with seismic pins include transverse high-strength threaded rods at the top of the pier cap within the pin support area. These rods increase confinement and improve shear capacity of the pier caps during a seismic event.

The drilled shafts were detailed assuming wet ground conditions during construction. Cross-hole sonic testing was performed to ensure quality of the placed

Erection over the La Jolla Village Drive intersection. For the longest span crossing over the La Jolla Village Drive intersection, hammerhead pier tables were constructed on falsework. The two main span U-girders were spliced on the ground in the staging area and then erected on the falsework tower at the ends of the pier tables. The back-span girders were erected on falsework towers and then spliced together. Photo: Mid-Coast Transit Constructors.

concrete. All longitudinal reinforcing steel in the columns and drilled shafts is continuous with no lap splices.

Girder Erection Sequence

The design plans had three basic methods for U-girder erection. The first method was used for frames that do not cross over any intersection or driveway wider than 60 ft. This method involved erection of individual girder segments that are about 60 ft or shorter, weighing less than 100 tons, on temporary towers. Then, the closure pours, diaphragms, and CIP deck were constructed, followed by post-tensioning the full frame in one operation (single stage).

The second method was designed for frames that cross over an intersection wider than 60 ft. For this method, girders were erected on falsework towers at all locations except the intersection. Then, after closure pour construction, the erected U-girders were spliced using two ducts in each web (stage 1 post-tensioning). The U-girder segments for the intersection were spliced on the ground in the staging area, and then erected on falsework towers, allowing the intersection to remain open. Then, the closure pours with diaphragms and the CIP deck were constructed and the remaining two ducts in each web were used for continuity post-tensioning of the full frame (stage 2 post-tensioning).

The third erection method shown in the design plans was a hybrid method and was used to construct the longest frame crossing over the La Jolla Village Drive intersection. Hammerhead pier tables were constructed on falsework. The two main-span U-girders were spliced on the ground in the staging area and then erected on the falsework tower at the ends of the hammerheads. The back-span girders were erected on falsework towers and spliced together. To complete the construction, the diaphragms and CIP deck were constructed and continuity post-tensioning was applied to the full frame.

As the contractor gained experience and confidence in the girder erection, closure pours, and post-tensioning operations, they proposed using a quick-set, high-early-strength concrete mixture with no fly ash. It was hoped that use of this mixture would eliminate the need for a staging-area splicing operation and lifting of already spliced girders. After mock-up testing of the concrete mixture to check strength development and concrete temperature during curing, the proposal was approved. With the approval, the contractor was able to form, place, cure, and achieve required concrete strength of the closure pours in two days. Then, the first-stage post-tensioning could be completed and the intersection opened to traffic within six days. This approach reduced night work, shortened the schedule, and reduced risk.

Splicing operations for two U-girder segments crossing the Esplanade Court intersection. The contractor closed the intersection and used a quick-set, high-early-strength concrete mixture. This span terminates at the in-span hinge of the cast-in-place (CIP) box station frame in the foreground. The falsework is for the hinge and the CIP box frame. Photo: Mid-Coast Transit Constructors.

Rendering of the completed Genesee Avenue Viaduct. Source: WSP.

As of this writing, the contractor had completed the viaduct and was attaching direct fixation rails and utilities to the bridge.

The use of a precast concrete U-girder bridge for this viaduct reduced the total construction duration and significantly reduced night construction. These time savings greatly benefited the nearby residences, businesses, and travelers along this major arterial street. The precaster's initial investment in U-girder forms helps provide future opportunities to design concrete bridges with more complex, longer spans and aesthetically pleasing features, with fewer construction impacts, in Southern California.

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EDITOR'S NOTE

The PCI U-girder sections mentioned in this article were developed by PCI Zone 6. The full set of drawings is available at www.pci.org/PCI_Docs/Design_Resources/Transportation_Resources/PCI%20Zone6%20Curved%20Spliced%20Girders.pdf.

Shear Assessment of Complex Concrete Pier Caps

by Daniel J. Baxter, Emily Thomson, and Dr. Francesco M. Russo, Michael Baker International, and Jessica Wahl Duncan, Minnesota Department of Transportation

The Lake Avenue Bridge over Interstate 35 (I-35) in Duluth, Minn., is a stylish, cast-in-place reinforced concrete box-girder bridge, which was constructed in 1982. Supporting a single-point urban interchange (SPUI), the bridge connects downtown Duluth to the tourist hotspot of Canal Park along Lake Superior and is one of the primary means of access to downtown and Canal Park from I-35. Every June, the structure is a focal point for the thousands of runners who pass under it just before making the final turn to the Canal Park finish line of Grandma's Marathon.

The Lake Avenue Bridge consists of multiple concrete box-girder units with complex geometry to fit the SPUI and the ramps to I-35. When the Minnesota Department of Transportation (MnDOT) required a load rating for this structure, it retained Michael Baker International (MBI) to prepare a load and resistance factor rating (LRFR). MBI developed a three-dimensional (3D) finite-element model of the structure (**Fig. 1**) using midas Civil software, following the grillage analysis methods specified for horizontally curved concrete box-girder bridges in *Design Specifications and Commentary for Horizontally Curved Concrete Box-Girder Highway Bridges,* published by the National Cooperative Highway Research Program.[1]

To perform a load rating of the box-girder web lines and tributary flange widths, the MBI evaluation team used output forces from this model in conjunction with spreadsheets that tracked the many reinforcement transitions along the length of the bridge units. Following the project scope of work, the team also used the demands output from the model to conduct load ratings using strut-and-tie modeling (STM) of the internal integral cap beams over the piers (**Fig. 2**) and of the bridge's corbel joints between superstructure units (**Fig. 3**).

The intent of the technical approach was to rigorously apply the American Association of State Highway and Transportation Officials (AASHTO) *Manual for Bridge Evaluation* (MBE);[2] however, the evaluation team understood and expected that various elements of the piers would not meet current AASHTO design and detailing requirements. In the eighth edition of the *AASHTO LRFD Bridge Design Specifications,*[3] Articles 5.1 and 5.5.1.2 establish a clear path that leads to the conclusion that STM is the only appropriate *design* approach for new structures with disturbed regions. But when faced with evaluating existing structures designed and detailed using a different standard, the solutions are not always as clear (see the FHWA article in the Fall 2019 issue of *ASPIRE*®).

The evaluation team load rated the bridge for the HL-93 loading at the inventory and operating levels, and the standard Minnesota legal and permit vehicle loadings. The nearly 40-year-old bridge is in good condition, and the superstructure and corbel joints had passing load ratings for all legal vehicles and nearly all permit vehicles. However, the integral cap beams did not have adequate shear capacity for some current legal loads when analyzed with STM or when using a sectional model approach, as would have been used during their initial design. The problematic cap beam regions were nearly all located within a distance extending horizontally outward from the face of the piers equal to the effective shear depth d_v. Within these regions, the existing vertical reinforcing steel area in the integral cap beams was not enough to provide the shear reinforcement contribution capacity V_s required to produce passing load ratings using sectional analysis, or the required reinforcing steel area to satisfy the vertical tie demands from STM for the same applied loads.

AASHTO LRFD specifications require that the shear capacity within a distance extending d_v from the face of the support be evaluated if concentrated loads are located within this distance. On the Lake Avenue Bridge, most of the integral cap beams support box-girder web lines within

Lake Avenue Bridge looking north on Interstate 35. Photo: Minnesota Department of Transportation.

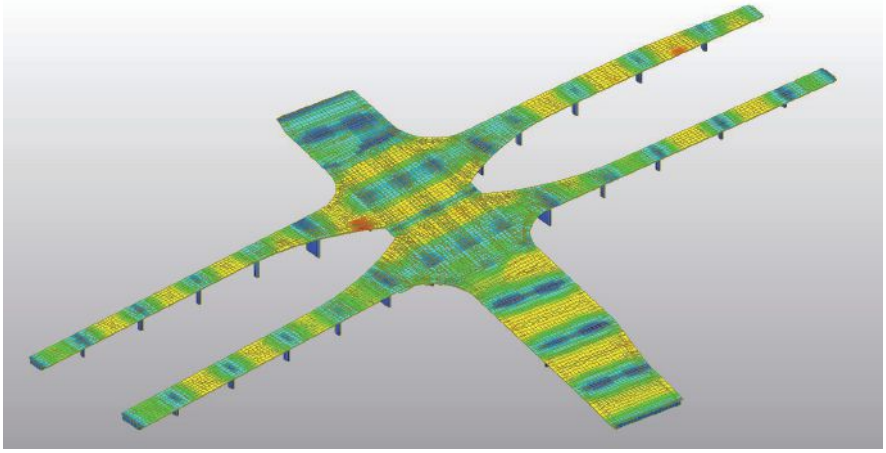


Figure 1. Rendering of the three-dimensional finite-element model of Lake Avenue Bridge. Figure: Michael Baker International.

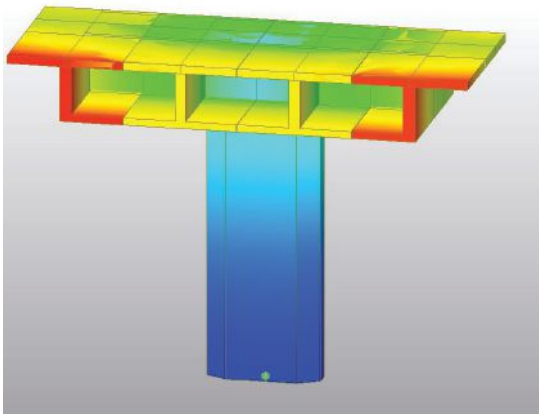


Figure 2. Cross section of the three-dimensional finite-element model showing the cast-in-place reinforced concrete box-girder unit, pier column, and integral pier cap beam for a Lake Avenue Bridge ramp structure. Figure: Michael Baker International.

this distance; the evaluation team initially treated these as concentrated loads, with the critical section being taken at the face of support. The team also considered taking the critical section at a distance of d_v from the face of the support if a detailed bridge inspection found no signs of shear cracking in the d_v region. They reasoned that if an inspection of these regions found no shear cracks after nearly 40 years of service, the performance of the bridge would be demonstrating that the loads from the webs and integral flanges were effectively acting more like distributed loads, for which AASHTO LRFD specifications permit the critical section for shear to be taken at d_v from the support face for a sectional analysis. MnDOT's District 1 bridge inspection staff performed a supplementary inspection of multiple cells of the bridge adjacent to the integral cap beams, which revealed narrow, diagonal cracking in the cap beams near the support faces, as shown in **Fig. 4**.

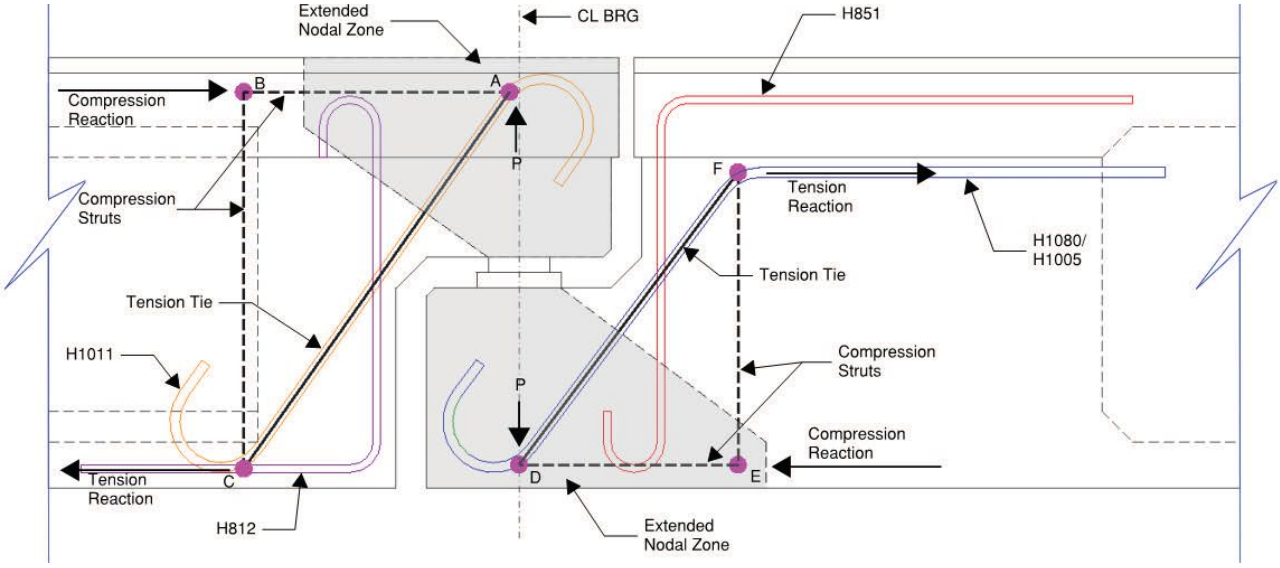
Given the multiple, fine shear cracks in the integral cap beams within the d_v distance and poor rating results using an all-sectional or all-STM approach, the observed fair performance of the integral cap beams needed to be reconciled with the poor results of multiple calculation models that indicated more extensive shear cracking with wider crack widths should be present. The evaluation team had to either resolve these discrepancies or use the results of the models to load post the bridge, which would potentially be followed by costly retrofit repairs.

The analysis team obtained higher load-rating results that better reflected the good condition of the cap beams by using a hybrid analysis method incorporating both sectional analysis and STM. They used STM to evaluate the integral cap beams within d_v from the face of the column, and applied the sectional method in the span beyond d_v from the column face because that method discretely considers the concrete and shear reinforcement contributions to shear, V_c and V_s, both of which appear to be intact. At the interface between the B- and D-regions (as described in AASHTO LRFD specifications), the hybrid analysis ensures there is a load path to couple the B- and D-regions together and carry the demand to the columns.

In the STM for the example cap beam shown in **Fig. 5**, forces V_A and V_B each equal one-half of the total shear demand V_u at a distance d_v from the left face of the column. This analysis causes vertical tie *AB* in the model to only carry half of the total shear demand V_u. The other half of the total shear demand V_u is applied to top node *A* and transferred by the diagonal strut *AD* and other struts and ties into model supports *D* and *F* (which represent the pier), without creating demand in tie *AB*.

If the entire cap beam had been modeled using STM, vertical tie *AB* would be required to carry the entire shear demand at distance d_v from the face of the

Figure 3. Lake Avenue Bridge corbel joint showing the primary reinforcement and strut-and-tie models used for evaluation. Note: Solid and dashed lines represent tension ties and compression struts, respectively. Figure: Michael Baker International.



column, assuming no concentrated loads are applied at nodes *A* or *B*. Flexural demands were similarly addressed but are not shown for clarity.

Horizontal forces H_A and H_B are obtained from the moment *M* at the interface between the sectional and strut-and-tie models. These forces are equal and opposite, and are equal to the moment *M* divided by the vertical distance between nodes *A* and *B*. The forces are applied in the direction needed to induce tensile and compressive axial forces in the horizontal chords of the strut-and-tie model that are of the same sense as the top and bottom flexural stresses induced by the moment *M* at the interface. For the Lake Avenue Bridge integral pier caps, this procedure resulted in the top chords of the strut-and-tie models being in tension over the piers, and the bottom chords in compression.

This hybrid analysis approach recognizes the contribution of concrete to shear strength by not requiring that vertical tie *AB* transfer the entire shear demand V_u up to node *A*, as would be needed for the STM of the entire cap. The existing vertical reinforcing steel area in the cap beams was sufficient to satisfy the vertical tension demand in tie *AB* obtained using the hybrid approach. The overlapping diagonals *CF* and *DE* in the example model of Fig. 5 were analyzed as compression-only truss elements, which effectively removes one of the two diagonals for different unbalanced live-loading patterns. This technique ensures that all ties are horizontal or vertical and align with the existing reinforcement layout.

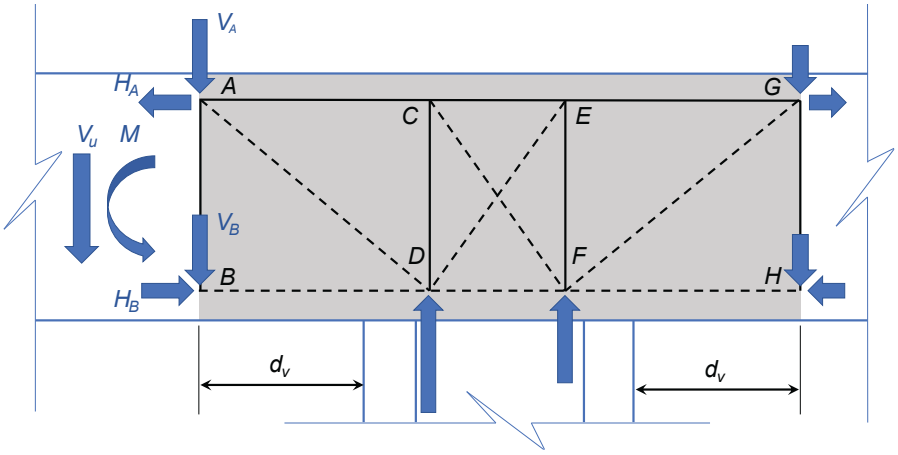


Figure 5. Strut-and-tie model of the hybrid analysis of the integral cap beam. V_A and V_B each equal one-half of the total shear demand V_u. Note: Solid and dashed lines represent tension ties and compression struts, respectively. Strut-and-tie modeling was used in the D-region of the cap beam (shaded gray). The sectional model was used in the B-region of cap beam (not shaded). Figure: Michael Baker International.

All portions of the integral cap beams that did not have passing legal load ratings using the original sectional method checks were evaluated using the hybrid approach. The evaluation team's 3D finite-element model was used to generate concurrent live-load forces for use in the hybrid method to evaluate capacity.

Using this hybrid analysis method produced passing ratings for Minnesota legal loads and permit loads for all the Lake Avenue Bridge integral pier cap beams. These results conform with the condition observed in these regions of the bridge and avoid the need for load posting or structure retrofits. The Lake Avenue Bridge should provide many more decades of service to Duluth's citizens and visitors, and will continue to serve as a landmark for weary marathon runners nearing the finish line.

Figure 4. Diagonal cracking (dashed lines) noted during inspection of an integral cap beam of the Lake Avenue Bridge. Cap beam is on the left, and girder web is on the right. Photo: Minnesota Department of Transportation.

References

1. National Cooperative Highway Research Program (NCHRP). 2008. *Design Specifications and Commentary for Horizontally Curved Concrete Box-Girder Bridges: Final Report*. NHCRP Project 12-71. Washington, DC: Transportation Research Board. *http://onlinepubs.trb.org/onlinepubs/archive/notesdocs/nchrp12-71_fr.pdf*
2. American Association of State Highway and Transportation Officials (AASHTO). 2018. *Manual for Bridge Evaluation*, 3rd ed. Washington, DC: AASHTO.
3. AASHTO. 2017. *AASHTO LRFD Bridge Design Specifications*, 8th ed. Washington, DC: AASHTO.

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EDITOR'S NOTE

This is the first part of a two-part Concrete Bridge Technology article series on strut-and-tie modeling (STM) by these authors. The second article will discuss how to use STM for portions of new design that are more challenging than what is typically shown in example problems, as well as some of the techniques Michael Baker International uses for complex designs.

Epoxy-Coated Prestressing Steel Strand

Three types of surface finish are available for epoxy-coated strands (from left): smooth, for unbonded applications; fine grit; and coarse grit. The latter two are for bonded applications. Note individual coating of each wire. Photo: Sumiden Wire Products Corporation.

by Jon Cornelius and Jeff Feitler, Sumiden Wire Products Corporation

Prestressed concrete was introduced in the 1950s and is now widely recognized as a durable and superior construction method for many structural applications, including bridges of all sizes and types, wind towers, and single-story and high-rise buildings. To further increase the service life of prestressed concrete structures, owners, design engineers, and contractors want cost-effective and reliable methods to prevent premature corrosion of the reinforcing steel. Carbon fiber, stainless steel, and epoxy coatings have all been successfully used to help eliminate or significantly reduce the risk of premature reinforcing steel corrosion. This article focuses on the development and success of epoxy-coated prestressing steel strand (as specified in ASTM A882).

History

Epoxy-coated strand was first developed in the early 1980s by Florida Wire and Cable with the help of a grant from the Federal Highway Administration. When the product was initially developed, the epoxy coating was only applied to the outside of the strand. Field experience with this first-generation epoxy-coated product soon proved this coating practice had a serious flaw: water could enter the strand from the exposed ends and migrate through the unfilled interstitial spaces between the wires. When this was recognized, the epoxy-coating process was redesigned to remove this vulnerability by filling the interstitial spaces with the epoxy-coating material in addition to applying the coating on the outside of the strand.

In the following years, the technology for making epoxy-coated strand was further refined and improved, with advancements made in the uniformity of coating thickness, coating adhesion, and other characteristics of the product. For example, when the coating was initially developed for bonded applications, sand was impregnated on the epoxy coating to enhance the bond performance. This coarse-grit finish was effective at developing bond, but it proved to be problematic for thin members because resulting transfer lengths were very short. The coarse grit (sand) epoxy-coated strand was also abrasive to handle and caused unwanted wear when it rubbed against tooling, as it would when passing through the holes in prestressing form bulkheads. To address these issues, fine-grit epoxy-coated strand was developed using aluminum oxide impregnated on the epoxy surface. The fine-grit finish on the epoxy-coated strand resolved the handling and wear concerns associated with the coarse-grit finish, and its transfer length is similar to bare strand. Currently, three types of surface finish are available: smooth, for unbonded applications; fine grit; and coarse grit–the latter are both for bonded applications. Epoxy-coated prestressing strands are available in the same diameter ranges as uncoated steel strands.

Distinctive Characteristics

Although epoxy-coated prestressing steel strand has been available in the U.S. market since the 1980s, many in the prestressed concrete community are not familiar with this product. When first hearing about epoxy-coated strand, many people may associate this product with its distant cousin, epoxy-coated reinforcing bars. While this association is understandable, there are significant differences between the two products.

One of the primary differences between the two products is the surface preparation prior to epoxy coating. Surface preparation for an epoxy-coated reinforcing bar primarily consists of a shot-blast process to remove the mill scale remaining after the steel hot-rolling manufacturing process, a subsequent cleaning and removal of the shot blast media and related debris, and a heating process at ~450°F; then, the epoxy coating is applied.

For epoxy-coated strand, the mill scale is removed from the hot-rolled wire rod before the production of the individual wires. Next, the descaled wire rod is cold-drawn through a series of dies, which ensures the wire surface is uniform and free of any loose debris. During the stranding process, the individual wires are stranded and then heated to over 700°F, burning off a large portion of any remaining surface contaminants. This heated strand is rapidly cooled in a water bath, which also acts as another cleaning process. Prior to epoxy coating, the bare strand is passed through an acid-cleaning operation, a subsequent water rinse, and a heating process at ~400°F. After all of these processing and cleaning steps, the 15-

Bite-through wedges, which have longer teeth, must be used when tensioning epoxy-coated prestressing steel strand. Coated strand at left shows how the teeth have penetrated the coating to engage the surface of the strand. Photo: Sumiden Wire Products Corporation.

to 45-mil epoxy coating is finally applied. When epoxy-coated prestressing steel strand is coated, each individual wire is coated as well as the overall strand surface. The strand is untwisted and then retwisted during the epoxy coating process to ensure that all surfaces of the wires are covered and the interstitial spaces are filled. As a result, if the coating is damaged in the field, only the wires with damage have been exposed to potential corrosion. Additionally, because of the superior adhesion of the coating, corrosion does not migrate or travel along the strand if the epoxy is damaged and bare steel wire is exposed. Only the exposed surfaces can corrode, and the adjacent epoxy coating does not blister or soften. This remarkable adhesion property has been confirmed in 3000-hour salt fog tests with the strand tensioned at 70% of its guaranteed ultimate tensile strength.

The tensioning properties of epoxy-coated strand are essentially the same as those of traditional bare strand, but there are slight differences in handling and care. For example, bite-through wedges must be used when tensioning the epoxy-coated strand, unless the user elects to strip off the epoxy in the gripping sections. Also, the epoxy-coated strand is delivered on wooden spools, whereas conventional bare prestressing steel strand is delivered with reel-less packaging; therefore, a different payout device must be used when working with epoxy-coated strand.

The epoxy coating on epoxy-coated strand is both durable and ductile. If the coating were to become damaged, a two-part epoxy-patching compound is available for repairs of any size. The patching process can be easily performed in the field with a small applicator and heat gun. The resultant patch has the same adhesion and elongation properties as the original epoxy coating. The coating material is also specially engineered to exceed the elongation demands needed in both the longitudinal direction and when bending the strand in radii as tight as 32 times the nominal strand diameter.

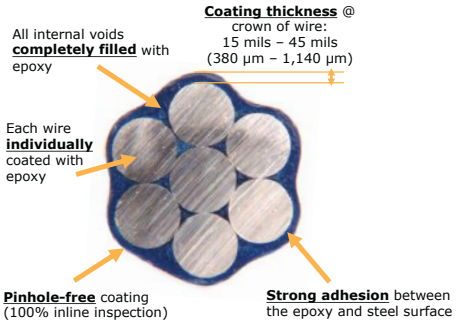
Applications for Bridges and Other Transportation Structures

Since the early 1990s, over 700 projects in Asia, primarily in Japan and South Korea, have successfully used epoxy-coated prestressing steel strand in bridge construction for applications such as external tendons for segmental bridges, transverse tendons, internal tendons, and extradosed/stay cable tendons. Epoxy-coated strand has also been used for ground anchors and mine roof bolts. In the United States, epoxy-coated strand has been successfully used in pretensioned concrete girders, piling, and deck panels; post-tensioned stay cables; ground anchors; and several other specialty applications.

One of the largest U.S. pretensioned concrete projects using epoxy-coated strand was the widening of the San Mateo Bridge, crossing San Francisco Bay between Hayward and Foster City, Calif. The total length of this section of the San Mateo Bridge is 4.9 miles and consists of 2160 bulb-tee girders, 826 piles, and 18,293 deck panels (see article in the January-February 2005 issue of the *PCI Journal*). Epoxy-coated strand was selected for all prestressed concrete elements because this structure is very close to the water level with all elements within the marine splash zone and the owner set the objective of achieving a 125-year service life. The bridge was completed in 2002, within budget and on schedule, and has performed as designed without any corrosion concerns.

The U.S. prestressed concrete industry has used epoxy-coated strand in a variety of pretensioned applications, including bridge decks, piling, and girders. Precast concrete H-columns for highway sound walls are a new pretensioned concrete application in areas where deicing salts are frequently used. Another market application is ground anchors and tiebacks where corrosion can be a significant concern.

To date, the most prominent application for epoxy-coated prestressing steel strand in the United States has been cable-stayed bridges. Two iconic cable-stayed bridges using epoxy-coated strand are the Penobscot Narrows Bridge near Bucksport, Maine, and the Veterans Glass City Skyway Bridge in Toledo, Ohio. Two additional signature cable-stayed bridges currently under construction are using a combined total of over 5000 tons of epoxy-coated strand: the Harbor Bridge in Corpus Christi, Tex., and the Houston Ship Channel Bridge in Houston, Tex.



Cross section of an epoxy-coated strand showing the complete coating of each wire. Photo: Sumiden Wire Products Corporation.

Owners are now demanding 100-year-plus service lives for many of their key infrastructure projects. Epoxy-coated prestressing steel strand provides a viable and cost-effective option to meet this challenging durability standard.

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EDITOR'S NOTE

ASTM A882 is listed as "Withdrawn" by ASTM as of this writing. This standard is in the process of being revised and will become active again once it passes the ASTM balloting process.

When the San Mateo Bridge was widened, epoxy-coated strand was used in the prestressed concrete of its 2160 bulb-tee girders, 826 piles, and 18,293 deck panels. Photo: Balfour Beatty Construction.

Time-Saving Construction Techniques—I-59/I-20 Bridge Reconstruction

By Robert W. Sward, Structural Technologies LLC

Since the 1970s, precast concrete segmental bridge construction has been used throughout the United States and many other parts of the world. Precast concrete segmental bridges are highly durable structures and may have predicted service lives greater than 100 years. As with most other forms of precast concrete construction, precast concrete segmental bridges can be constructed relatively quickly and allow the contractor to take advantage of innovative construction techniques. The recently completed Interstate 59/Interstate 20 (I-59/I-20) elevated bridge replacement project in Birmingham, Ala., showcases the benefits of precast concrete segmental construction (see the Project article on page 10 of this issue of *ASPIRE*®). This replacement project is one part of the $750 million Alabama Department of Transportation (ALDOT) reconstruction program for the I-59/I-20 corridor.

Precast concrete column in form with core form being extracted. Photo: Johnson Bros.

Strategies to Achieve an Aggressive Timeline

To minimize the impact of construction on the community, ALDOT opted to shut down the corridor to complete the replacement of the elevated structures in one phase. The maximum construction duration established in the contract was 14 months, but the contractor could maximize early-completion incentives by completing construction in 12 months. Constructing the replacement in multiple phases would have extended the project duration for several years. In May 2017, ALDOT awarded Johnson Bros. Corporation the $474.7 million contract for the I-59/I-20 bridge replacement project. This contract included a substantial bonus for early completion and unlimited liquidated damage penalties for late completion. To successfully complete this project and minimize schedule risk, Johnson Bros. implemented the following strategies.

Advance Work Beneath the Existing Bridge

The contractor decided to complete many of the new foundations and footings prior to shutting down the interstates for demolition. They used a combination of driven H-piles and drilled shafts and, where there was limited head room, implemented micropiles to install the new foundations around and beneath the existing elevated roadway.

Precasting the Concrete Substructures

By precasting 160 pier cap and column substructure elements, the contractor was able to benefit from the nearby casting yard and minimize critical path activities. Delivering and assembling precast concrete elements proved to be much faster than conventionally forming and pouring variable-height piers in place.

Use of Custom Shoring Towers in Precast Concrete Segment Erection

In the original design, the use of longitudinal erection trusses was assumed. This is a linear method of segmental construction in which the erection truss is launched to the next span only after one span is fully erected. To meet an incentive-maximizing 12-month construction schedule using this method, Johnson Bros. would have needed a minimum of six launching trusses working concurrently. This linear method of erection would have been risky because if the erection of a particular span could not progress in sequence, the contractor lacked the flexibility to quickly move the erection equipment to another work area.

Working with Structural Technologies/VSL, Johnson Bros. elected to use custom shoring towers to support the precast concrete segments during segment erection in lieu of longitudinal erection trusses. This method reduced schedule risk

Precast concrete column segments in storage. Note the extended column reinforcement and grout sleeves that are visible on the ends of the segments. Photo: Johnson Bros.

Reinforcing bar cage with transverse tendons being installed into forms. Photo: Johnson Bros.

by providing the contractor the flexibility to deploy erection crews and equipment as needed to maintain the construction schedule, while allowing the use of more traditional construction equipment (cranes) for erection of the segments. If one particular span could not be erected in series, the crew and erection equipment could be redeployed to another area of the project. This method also allowed flexibility in sequencing the work.

The shoring towers supplied were custom designed and fabricated for this project. Shoring towers were preassembled at the fabricator's facility to ensure proper fit-up. The tower bases were fitted with screw jacks for vertical adjustments, and the tops were telescopic and fitted with jacking boxes. The main elements of the shoring towers were color-coded to be easily identified by field crews for their proper locations. Approximately 127 individual shoring towers were provided, which allowed the contractor to work on up to eight fronts at one time. To ensure that underground utilities near the towers would not be damaged by the construction loads imparted by the shoring towers, a foundation load analysis was performed.

Three-Dimensional Modeling

During the bid phase, a virtual three-dimensional model of the entire project was developed. Using this model, each bridge span was studied to confirm that it could be erected using the proposed shoring method. The study revealed that several spans would require special portal frames and/or sliding of the segments due to crane access limitations and ground conflicts, such as railroad tracks and streets that could not be closed.

Minimizing the Number of Segments

The length of typical and deviator segments was increased from 10 ft to 12 ft, which reduced the number of segments to cast, store, transport, and erect by about 400 segments. The team had to account for subsequent increases in segment weights and ensure that limits would not be exceeded for cranes, shoring, and hauling.

Dedicated Batch Plant

A project-dedicated batch plant was installed in the casting yard, including a quality-control lab. The dedicated batch plant ensured consistent delivery and quality of concrete as needed for both the jobsite (approximately 4 miles away) and the precast yard.

Maximizing Efficiency in the Casting Yard

The casting yard for the 2316 box-girder segments included 12 casting cells for the segments—eight for typical segments, two for pier segments, and two for expansion-joint segments. There were also 20 reinforcing bar jigs. In addition to the box-girder segments, 160 pier-cap and column elements were cast.

Casting cells and reinforcing bar jigs were organized to allow a single crawler crane to service four casting cells. The casting cells were fitted with movable shelters to protect both the segments and workers from the weather. To ensure reinforcement was ready when the forms were ready, each cell typically had two reinforcing bar jigs.

The segments included transverse post-tensioning (PT) tendons in the deck with four 0.6-in.-diameter, 270-ksi strands that were installed, tensioned, and

Casting cells for precast concrete box-girder segments with protective covers. Photo: Johnson Bros.

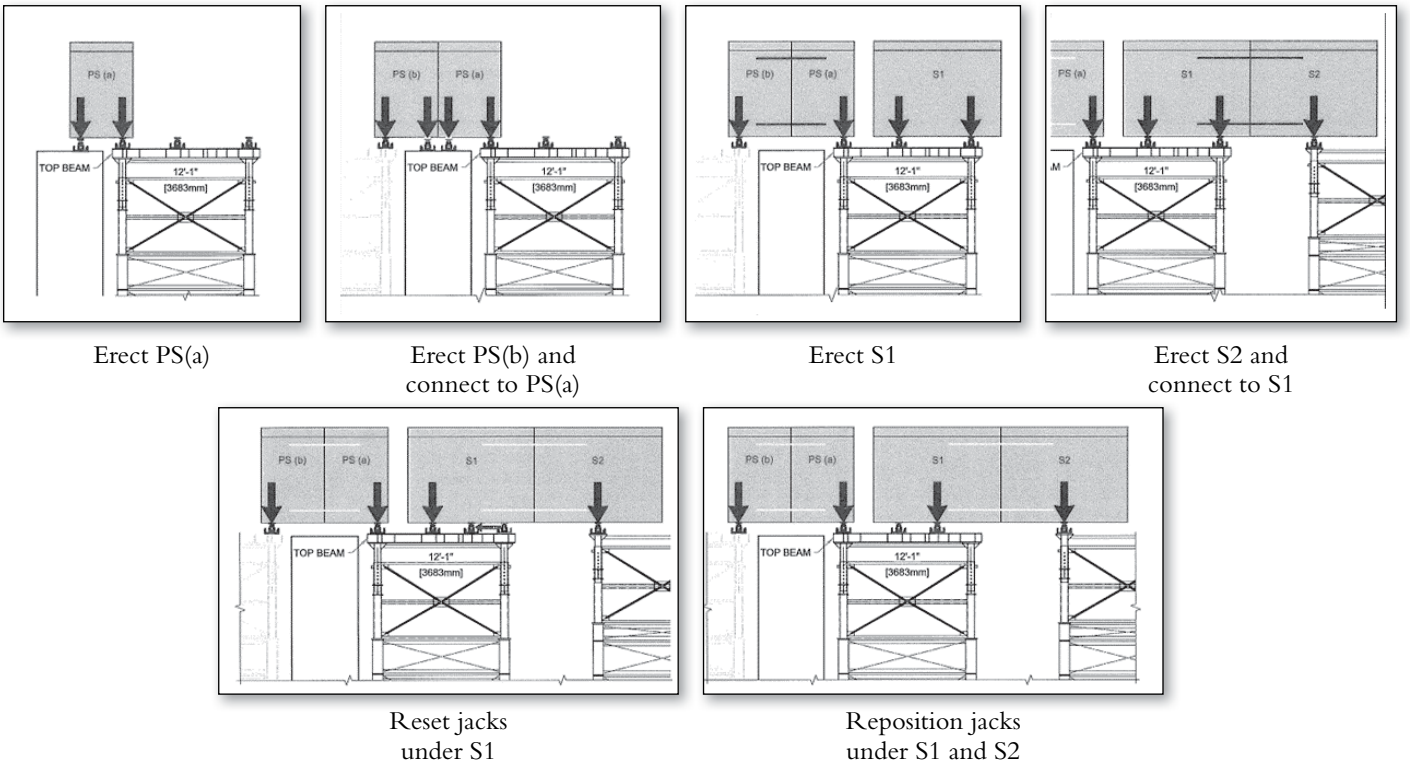


Figure 1. Typical segment erection sequence of a span. Note: PS indicates pier segment and S indicates span segment. Figure: Structural Technologies.

grouted in the casting yard. The 9386 four-strand PT transverse tendons were prefabricated and individually placed in the reinforcing bar cage. To minimize the number of transverse tendon pockets that would need patching, the dead-end anchorages of the four-strand transverse tendons were cast into the deck.

Once a segment was moved off the match-cast station, it was placed on a finishing stand where any blemishes were patched, and transverse tendons were tensioned. Segments were then moved and double-stacked in the storage area where transverse tendons were grouted. With eight different erection fronts, management of segment storage was critical to ensure that proper segments were delivered to the correct span and in the correct sequence.

Heavy segment haulers were used to transport the segments to the different erection fronts. During segment transit, the haulers were escorted by local police.

Multiple Work Fronts

During segment erection, the jobsite was organized into eight work fronts. Each had its own dedicated superintendent, crews, and erection equipment. Using multiple work fronts fostered a competitive environment that motivated crews to adhere to quality, schedule, and safety goals.

Typical Span Erection

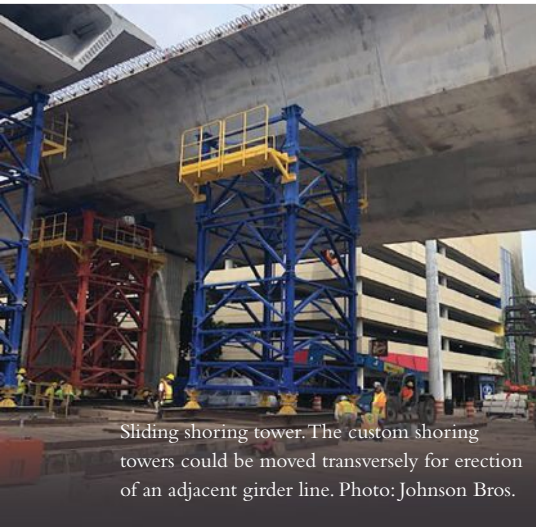
The shoring method for segment erection provided an additional benefit by reducing the time it took to erect segments. With a truss-supported method, the truss must be loaded with all segments before the segments can be joined together; this requires handling and moving the segments several times during the span erection cycle. In contrast, the shoring method does not require as much handling and moving of segments.

The span erection cycle began with the placement of shoring towers. Because the bridge typically had four parallel lines of box girders, towers could be moved transversely from one girder line to the next or set in position using a crane. Once the shoring towers were positioned, the typical span erection progressed by placing the leading edge pier segments, followed by the span segments (**Fig. 1**). The PT tendons were then installed and tensioned to an initial force. Closure joints were placed and the tendons tensioned to full force once the closure pour concrete achieved the minimum required strength. At this point, the shores were released and moved to the next span. An advantage of

Using a three-dimensional model, engineers determined that several spans would require portal frames. Here, two portal frames support segments over an existing bridge. Photo: Structural Technologies.

The erection of a split-pier segment. Photo: Structural Technologies.

The erection of the first span was completed on March 11, 2019. The main elements of the shoring towers were color coded to be easily identified by field crews for their proper locations. Approximately 127 individual shoring towers were provided, which allowed concurrent work on up to eight fronts. Photo: Structural Technologies.



Sliding shoring tower. The custom shoring towers could be moved transversely for erection of an adjacent girder line. Photo: Johnson Bros.

this shoring method was that the segments could be placed and aligned in the same operation, reducing the number of times the segments needed to be handled during span erection.

Conclusion

In the I-59/I-20 project, the project team leveraged precast concrete segmental bridge construction to deliver a durable structure efficiently and maximize an early completion incentive. The strategy of using custom-designed shoring towers to allow nonlinear construction, using longer but fewer segments, and the use of precast concrete substructure elements all served to enhance constructability and reduce the contractor's risks. The project team demolished 180 spans of existing structure in 41 days, precast 160 pier caps and columns as well as 2316 box-girder segments in 18 months, and erected the 172 spans of the new bridge in just over 7 months. This replacement structure opened in mid-January 2020 and will serve the people of Birmingham and the traveling public for years to come.

Robert W. Sward is a vice president with Structural Technologies LLC in Fort Worth, Tex.

EDITOR'S NOTE

Visit aspirebridge.org for additional information about and photos of this and other projects.

Investigating Existing Grouted Tendons

by Miroslav Vejvoda, Post-Tensioning Institute

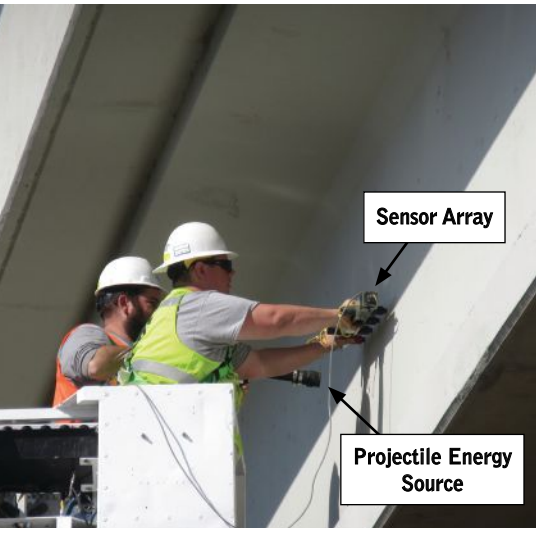
Cover of the July issue of 2019 *PTI Journal*, which featured grouted post-tensioning for bridges. All Photos and Figures: *PTI Journal*.

Post-tensioning (PT) tendons enable designers to build large, economical concrete bridges with low maintenance costs. PT tendons are the primary reinforcement relied on for the bridge's structural integrity. Tendons are typically grouted for two reasons: the grout protects the strand against corrosion, and it provides the bond with the surrounding concrete that achieves strain compatibility in design.

Owners' Concerns

Given the significance of the grouted tendons, we need to ensure adequate durability to satisfy the design expectations. Although the vast majority of grouted tendons perform as designed, there have been limited cases of tendon corrosion due to grout defects or entry of contaminants. The observed grout defects included voids in the tendons and soft grout. Both can lead to reduced protection of the strands as well as local corrosion that can lead to strand breakage or, in extreme cases, tendon failure.

The owners of major PT bridges have two main concerns about grouted tendons, which the industry has been striving to address. First, owners want to be assured that quality grouting can be consistently achieved. Great progress has been made in this direction (see the Concrete Bridge Technology articles in the Winter 2017 issue of *ASPIRE*®). The fourth edition of *PTI M55.1-19: Specification for Grouting of Post-Tensioned Structures*,[1] was issued in 2019. This document includes requirements for the design, testing, and installation of grout, along with detailed grouting procedures. When the specification is followed in its entirety, good results are achieved. Also, the PT industry has moved to prepackaged grout mixtures that achieve consistent results when mixed and handled properly.



Impact-echo testing. See Reference 2.

Second, owners want to know how to detect grout defects in an already installed tendon. To address this concern, three papers on the subject have been recently published in the *PTI Journal*. The articles are available for download on the Post-Tensioning Institute website using the links provided in the references.

Nondestructive Tests to Detect Defects

The most recent of the three *PTI Journal* articles is "Nondestructive Testing for Voided and Soft Grout in Internal Post-Tensioning Ducts," by Paul Fisk and Benson Armitage.[2] This July 2019 article investigates the use of two nondestructive testing methods that can be used in detecting grout defects in tendons. The ground-penetrating radar (GPR) method is used to accurately locate the internal ducts to a precision of ½ in. Then, sonic/ultrasonic impact-echo testing can be used to detect voids or soft grout. Once a defect location is identified, workers can drill into the duct and use a borescope to document the tendon condition. The impact-echo system uses a projectile impact-energy source, a sensor array, and a computer to achieve quality display of data. If a duct is fully grouted, the compression wave velocity generated and recorded by the impact-echo system remains constant. When voids or soft grout exist in the tendon, the resonant frequency is significantly lower or not measurable, indicating anomalies along the wave path.

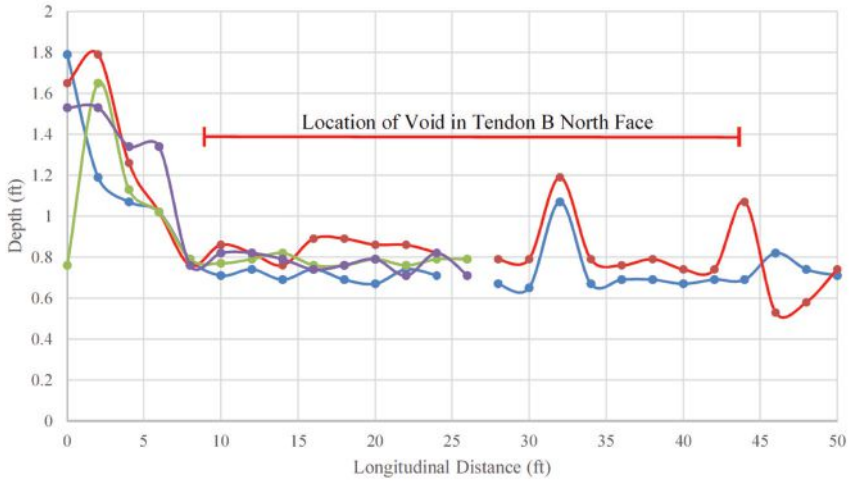
There are limits to the effectiveness of these testing methods related to the thickness of the concrete member. Most of the time, these limitations are not an issue when evaluating a typical thin-walled segmental or box-girder bridge. However, GPR and impact-echo methods have limited accuracy in thicker areas, such as tendon high points. Even with the limitations, the impact-echo system is a promising method that can help identify issues in existing tendons.

The second article is "NDT Investigation of PT Ducts," by David Corbett.[3] This December 2018 article also describes how GPR is used to locate tendons; once they are located, potential grouting defects are identified through an ultrasonic pulse and echo (UPE) method. Objects in concrete are detected through reflections that occur at the boundary between two materials with different properties. GPR works with reflections occurring when the two materials have different dielectric properties, whereas UPE works with reflections occurring when the two materials have differing acoustic impedance. GPR waves are totally reflected at a concrete-steel boundary and partially reflected at an air or plastic boundary. UPE waves are totally

Pulse echo instrumentation includes a transducer and an indicating device (upper photo). The lower photo shows the transducer directly above a duct. See Reference 3.

reflected at a concrete-air boundary and partially reflected at a steel or plastic boundary. Because of that, UPE enables PT duct detection through reinforcing bar configurations that would limit GPR. More significantly, the total reflection from air may make detection of voids in the duct possible. A comprehensive evaluation using the amplitude and phase analysis techniques of the pulse-echo technology is now possible as three-dimensional imaging is available in commercial equipment used on site.

The third article of note is "Grouted Post-Tensioning Tendon Evaluation," by Liao Haixue.[4] This December 2017 article discusses the causes of corrosion in tendons and describes evaluation techniques (GPR for locating tendons and sonic/ultrasonic impact echo for detecting grout voids or soft grout). Additionally, the paper discusses a case study of a bridge evaluation in British Columbia. Two possible voids, approximately 25 to 30 ft long, were identified using GPR and the impact-echo method. Subsequent drilling into the duct and borescope investigation confirmed the voids to be 28 to 33 ft in length, demonstrating that the nondestructive testing method identified the voids with good accuracy. Once the issues are located and evaluated, corrosion mitigation and repair solutions can be designed.



Sonic/ultrasonic impact-echo testing identified potential voids and soft grout areas. See Reference 4.

These three *PTI Journal* articles highlight how GPR can be used for locating tendons and the impact-echo method can be used to locate tendons in congested areas and to detect potential voids or soft grout in ducts. Together, these methods have been used to successfully identify grouting deficiencies, typically voids but sometimes also soft grout. Subsequent drilling into the ducts of those areas and the use of a borescope gives the investigators sufficient information to devise an effective repair.

Conclusion

As always, the best defense against corrosion of the tendons is to do the job right the first time. This starts with the owner and the designer using the state-of-the-art specifications *PTI/ASBI M50.3-19: Specification for Multistrand and Grouted Post-Tensioning*[5] and *PTI M55.1-19: Specification for Grouting of Post-Tensioned Structures*[1] (see the Summer 2019 issue of *ASPIRE* for an article discussing these specifications). These specifications are intended to be used together and should both be referenced in their entirety in the project specifications. Doing the job right the first time also means specifying and enforcing the field personnel certification requirements as outlined in the M50.3 specification. A properly trained workforce is much less likely to make errors in the installation and grouting processes. The grouting plan, as required in the M55.1 specification, should be followed from the beginning to the end, including any contingencies that are part of the plan. With all parties pulling together, tendon grouting deficiencies can be largely avoided.

References

1. Post-Tensioning Institute (PTI). 2019. *PTI M55.1-19: Specification for Grouting of Post-Tensioned Structures*. Farmington Hills, MI: PTI.
2. Fisk, P., and B. Armitage. 2019. "Nondestructive Testing for Voided and Soft Grout in Internal Post-Tensioning Ducts." *PTI Journal* 15 (1): 31–35. https://www.post-tensioning.org/Portals/13/Files/PDFs/Publications/Reprints/PTIJOURNALFisk-Armitage.pdf.
3. Corbett, D. 2018. "NDT Investigation of PT Ducts." *PTI Journal* 14 (2): 31–34. https://www.post-tensioning.org/Portals/13/Files/PDFs/Publications/Reprints/PTIJOURNALCorbett.pdf.
4. Haixue, L. 2017. "Grouted Post-Tensioning Tendon Evaluation." *PTI Journal* 13 (2): 23–27. https://www.post-tensioning.org/Portals/13/Files/PDFs/Publications/Reprints/PTIJOURNALHaixue.pdf.
5. PTI and American Segmental Bridge Institute (ASBI). 2019. *PTI/ASBI M50.3-19: Specification for Multistrand and Grouted Post-Tensioning*. Farmington Hills, MI: PTI.

EDITOR'S NOTE

The editors thank the Post-Tensioning Institute for making links available for the three articles discussed so that readers can obtain them at no charge.

Guidelines for Inspection and Acceptance of Epoxy-Coated Reinforcing Steel at the Jobsite

by Pete Fosnough, The Epoxy Interest Group

Inspection of epoxy-coated reinforcing steel prior to concrete placement is critical to ensure that optimum corrosion protection is provided to the structure. In particular, repair of coating damage on the reinforcing bars prior to concrete placement and the use of plastic-headed vibrators should be addressed. This article presents information on the inspection of epoxy-coated reinforcing steel and may be used to supplement state or other agency requirements for epoxy-coated reinforcement.

Epoxy-coated reinforcing steel placed on epoxy-coated wire and plastic bar supports. All Photos: EIG.

Handling

Just like any material used on a jobsite, epoxy-coated reinforcing steel requires appropriate handling. These steps are aimed at reducing damage to the coating that could reduce its corrosion-protection performance. Handling and storage requirements for epoxy-coated reinforcing steel may be included in contract documents or individual agency specifications by reference to many different sources including ASTM A775, A934, A1055, A1078, and D3963,[1–5] as well as ACI 301.[6] The following summarizes the main requirements found in these specifications:

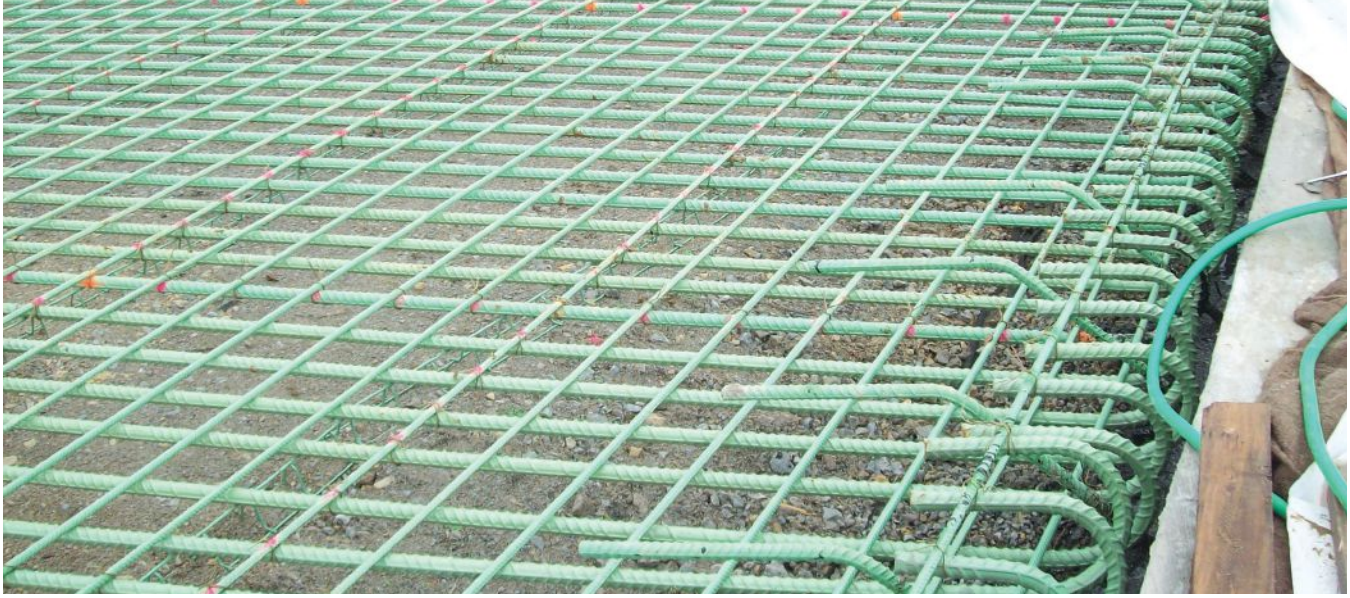
- Epoxy-coated reinforcing steel should be lifted using a spreader bar or strongback with multiple pickup points to minimize sag. During sagging, steel reinforcing bars may rub against each other, causing damage to the coating.
- At no time should epoxy-coated reinforcing steel be dragged.
- Nylon or padded slings should be used, and at no time should bare chains or cables be permitted.
- Epoxy-coated reinforcing steel should be unloaded as close as possible to the point of concrete placement to minimize rehandling.
- Bundles of epoxy-coated reinforcing steel should be stored on suitable material, such as timber cribbing. At no time should reinforcing steel be stored directly on the ground.
- If the epoxy-coated reinforcing steel is to be exposed outdoors for more than 30 days, it should be covered with a suitable opaque material that blocks ultraviolet light and minimizes condensation.
- Epoxy-coated reinforcing steel and uncoated steel should be stored separately.

Placement

To protect epoxy-coated reinforcing from oil contamination, steel forms should be oiled prior to placement of the reinforcing bars. Bars should not be dragged or placed directly on the forms, as this may result in oil contamination of the bar surface. Bars should be placed on supports coated with or made of nonconductive material, such as epoxy-coated or plastic bar supports, and these should meet Class 1A requirements, as defined in the CRSI *Manual of Standard Practice*.[7] Bars should be tied using coated tie wire. This wire is typically 16.5 gauge or heavier and black annealed. When used with epoxy-coated reinforcing bars, tie wire is typically coated with polyvinyl chloride (PVC). Coated bars may be cut using power shears, chop saws, or bandsaws; the cut ends should be patched using a two-part epoxy. Bars must not be flame cut. Bars may be bent at the jobsite, but only with the permission of the engineer responsible for the project, and this should be documented. If bending is to be conducted on site, it should be conducted at ambient temperatures.

Coating Damage

If the epoxy-coated reinforcing steel has more than 2% of its area damaged in any given 1 ft section of coated reinforcement,



With proper inspection and handling, epoxy-coated reinforcing steel effectively protects against corrosion in reinforced concrete applications such as this bottom mat of reinforcement for a bridge approach slab.

it may be rejected. ASTM D3963[5] further requires that the total bar surface area covered by patching material shall not exceed 5% in any given 1 ft section of coated reinforcement. These limits on damaged and repaired areas do not include sheared or cut ends.

Coating Repair

All damage to epoxy-coated steel reinforcement should be repaired to ensure maximum corrosion-protection performance. The areas to be repaired should be prepared for patching by using a wire brush to remove any rust or other contaminants. It is recommended for all epoxy repairs to use a two-part patch material that has been recommended by the powder-coating manufacturer. This material should be mixed according to the manufacturer's recommendations and should be used within the specified pot life. Patch repair materials should be provided enough time to cure prior to concrete placement, which may be up to 8 hours for some patch materials. All repairs should be conducted in strict accordance with the written instructions furnished by the patch material manufacturer. For epoxy-coated reinforcing steel to comply with ASTM A775,[1] the patch material must meet all of the requirements in Annex A2 of ASTM A775.

Inspection

Inspection prior to concrete placement is critical as this is the final opportunity to document that the coated reinforcing steel is installed according to the structural design and that the coating will provide optimum durability. Prior to concrete placement, the following should be inspected and appropriately documented:

- *Bar spacing, size, and type*: Bars should be placed in accordance with the structural drawings for the particular project, and conformance to the required specifications should be documented. Current practice often uses photographic documentation for recording mill and bar strength information, which is stamped onto the bar. Information obtained from the mill and fabrication reports may also be obtained.
- *Bends*: The coating at bends should not exhibit any cracking or fractures. Particular care should be taken to inspect the condition of the coating in these regions because damage may occur during fabrication.
- *Lap lengths*: Lap lengths approved by the structural engineer should be measured and documented. Laps are only permitted at locations approved by the structural engineer.
- *Mechanical splices*: Where mechanical splices are used, they should be epoxy-coated. Any damage to the coating after the bars are connected should be repaired.
- *Tolerances and clear cover*: Concrete cover should be measured. Additional information on placing tolerances is provided by ACI Committee 117[8] and CRSI.[7] If concrete cover on the reinforcing steel is inadequate, force is sometimes used to move the reinforcement. When such methods are employed, damage to the coatings must be avoided.
- *Repair of all damage*: All damage should be repaired.
- *Bar supports*: Epoxy-coated reinforcing steel should be placed on supports coated with or made of nonconductive material, such as epoxy-coated or plastic bar supports.
- *Tie wire*: Epoxy-coated reinforcing steel should be tied using a coated tie wire.
- *Bar samples*: Some agencies require inspectors to collect coated steel samples from the jobsite. These samples should be clearly identified prior to submittal to the appropriate laboratory for testing.
- *Welding*: Welding should only occur with the permission of the engineer. Any welded surfaces should be cleaned and repaired with patch material.

Protection of Coated Reinforcement Before, During, and After Concrete Placement

A meeting with the concrete contractor prior to concrete placement may be beneficial to discuss precautions that should be taken to protect coated reinforcement. After placement of epoxy-coated reinforcing steel, minimize traffic over it. At no time should stands or rails used for concrete placement machines be welded to the epoxy-coated reinforcing steel. Care should be used to ensure that activities during the concrete placement do not result in damage to the epoxy-coated reinforcing steel. Avoid placing concrete hoses on installed reinforcing steel; they may damage the coating as they are moved. Care should also be taken to ensure that items such as unprotected couplers for concrete delivery hoses are not dragged across the steel; these actions may result in coating damage. Consider a runway if necessary. Concrete pumps should be fitted with an "S" bend to prevent free fall of concrete

directly onto the coating. Plastic-headed vibrators should be used to consolidate concrete; steel vibrators may cause coating damage. Bars that are partially cast in concrete and then exposed for extended periods, should be protected against exposure to ultraviolet light, salts, and condensation. Wrapping bars with plastic or individual tubing is suitable for providing long-term protection.

Conclusion

Epoxy-coated reinforcing steel has been successfully applied to concrete bridge construction since the 1970s. As materials and processes have been improved through the years, bridges using epoxy-coated reinforcement have consistently demonstrated good field performance. Following the recommendations in this article helps ensure that the highest quality of corrosion resistance is provided for every project.

References

1. ASTM International. 2019. *ASTM A775/A775M-19 Standard Specification for Epoxy-Coated Steel Reinforcing Bars*. West Conshohocken, PA: ASTM International.
2. ASTM International. 2019. *ASTM A934/A934M-19 Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars*. West Conshohocken, PA: ASTM International.
3. ASTM International. 2016. *ASTM A1055/A1055M-16 Standard Specification for Zinc and Epoxy Dual-Coated Steel Reinforcing Bars*. West Conshohocken, PA: ASTM International.
4. ASTM International. 2019. *ASTM A1078/A1078M-19 Standard Specification for Epoxy-Coated Steel Dowels for Concrete Pavement*. West Conshohocken, PA: ASTM International.
5. ASTM International. 2015. *ASTM D3963/D3963-15 Standard Specification for Fabrication and Jobsite Handling of Epoxy-Coated Steel Reinforcing Bars*. West Conshohocken, PA: ASTM International.
6. American Concrete Institute (ACI) Committee 301. 2016. *Specifications for Structural Concrete* (ACI 301-16). Farmington Hills, MI: ACI.
7. Concrete Reinforcing Steel Institute (CRSI). 2018. *Manual of Standard Practice*, 29th ed. Schaumburg, IL: CRSI.
8. ACI. 2010. *Specification for Tolerances for Concrete Construction and Materials* (ACI 117-10). Farmington Hills, MI: ACI.

About the Epoxy Interest Group

The Epoxy Interest Group (EIG) of the Concrete Reinforcing Steel Institute (CRSI) is a not-for-profit trade association providing an authoritative resource for information related to the use of epoxy-coated reinforcing steel in reinforced concrete applications. Epoxy-coated reinforcing steel is also known as epoxy-coated rebar (ECR), as well as fusion-bonded epoxy-coated rebar (FBECR).

The association serves the needs of specifiers, engineers, architects, fabricators, and end users with the most recent information about how and where epoxy-coated reinforcing steel is used, as well as recent technical changes, educational seminars, and promotional activities.

EIG serves the construction market in the United States, Canada, and Mexico.

Our Mission

The mission of the EIG is to promote the use and advance the quality of epoxy-coated reinforcing steel. Epoxy-coated reinforcing steel is used instead of conventional reinforcing bars to strengthen the concrete and protect against corrosion. The epoxy coating is applied to the steel in a factory prior to shipping.

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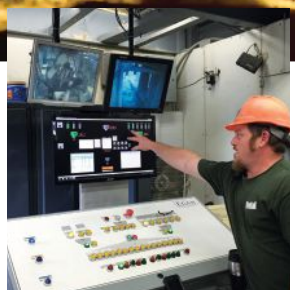
Photo courtesy of USG/Gus Rueles



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InfoBridge: Easy Access to the National Bridge Inventory and Much More—Part 2

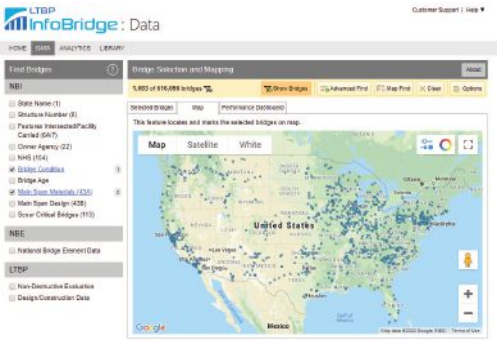
by Dr. Jean A. Nehme, Federal Highway Administration

In January 2019, the Federal Highway Administration (FHWA) released the inaugural version of the Long-Term Bridge Performance (LTBP) InfoBridge—a user-friendly, centralized portal for efficient access to bridge performance-related data and information (https://infobridge.fhwa.dot.gov). InfoBridge includes tools to facilitate bridge data analytics and provides for storage, retrieval, dissemination, analysis, and visualization of data collected through state, national, and LTBP program efforts. In January 2020, the second version of the web portal was released. This article, the second of two about InfoBridge, discusses enhancements implemented in the new release and describes some of the portal's advanced features (see the Winter 2020 issue of *ASPIRE*® for part 1).

Advanced Querying of Bridge Data

InfoBridge users are presented with an intuitive interface that enables querying on almost all data fields available within the portal. For example, the user could query the database for all bridges that have prestressed concrete as their main span material, have a structure length between 150 and 500 ft, and are in poor condition as defined in Chapter 23 of the Code of Federal Regulations (23 CFR 490.409). The query generates a table of results, which the user may customize and export. InfoBridge also displays the results on a map, which may be exported and used in reports or presentations (**Fig. 1**).

Figure 1. Queries can be made to form subsets of bridge data based on almost any attribute stored within InfoBridge. All Figures: Federal Highway Administration.



New to the second version of InfoBridge is the ability to query on geographical boundaries of metropolitan planning organizations (MPOs) and political districts such as U.S. congressional districts, state senate districts, and state house districts. In addition to MPOs and political districts, this latest version of InfoBridge allows querying based on special projects. Currently, data from three FHWA research projects are included: ultra-high-performance concrete, timber bridges, and weathering steel. More projects will be added in the future.

Historical Changes to Bridge Specifications

InfoBridge allows the user to explore changes to national bridge specifications over the years. This is a crucial feature for researchers investigating the performance of bridge assets. This tool lists specification changes chronologically and allows for searches based on a single keyword or on combinations of keywords. It also lists all pertinent references to each specification.

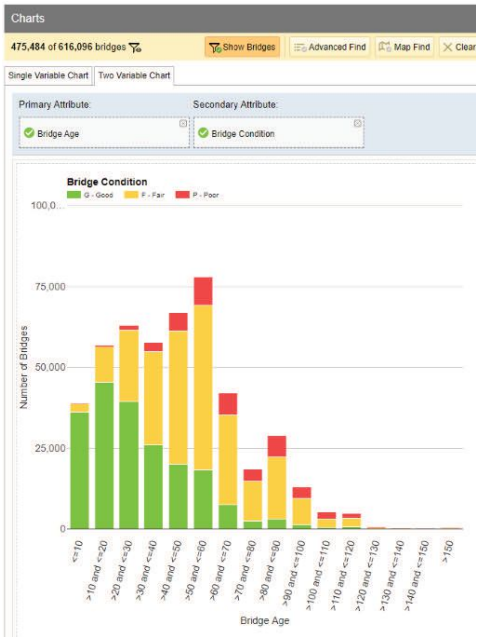
Currently, the following types of specifications are included:

- Reinforcing steel
- Loads—braking force, centrifugal force, earth pressure, earthquake load, ice load, pedestrian live load, temperature load, vehicular live load, water load, wind load
- Load combinations

In the future, the following types of specifications will be added:

- Minimum and maximum reinforcement
- Shear design of concrete
- Prestressed concrete design
- Lightweight concrete
- Load rating
- Approximate methods of analysis
- Steel bridge design

Figure 2. An InfoBridge two-variable chart displaying the correlation between bridge age and bridge condition.



Data Analytics and Visualization

It has been said that a picture is worth a thousand words; therefore, InfoBridge contains multiple tools to transform data and information into graphs, charts, contour plots, dashboards, and maps. All analytics and visualization tools are available on the selection of bridges that results from a user's query. An example of a two-variable chart is provided in **Fig. 2**.

Models for Bridge Deck Condition Forecasting

InfoBridge now includes three newly developed models to forecast bridge deck conditions: (1) time in condition, (2) deep learning, and (3) proportional hazards deterioration models (**Fig. 3**). All three models use historical National Bridge Inventory data in addition to data from other sources, such as the National Aeronautics and Space Administration Modern-Era Retrospective Analysis for Research and Applications,

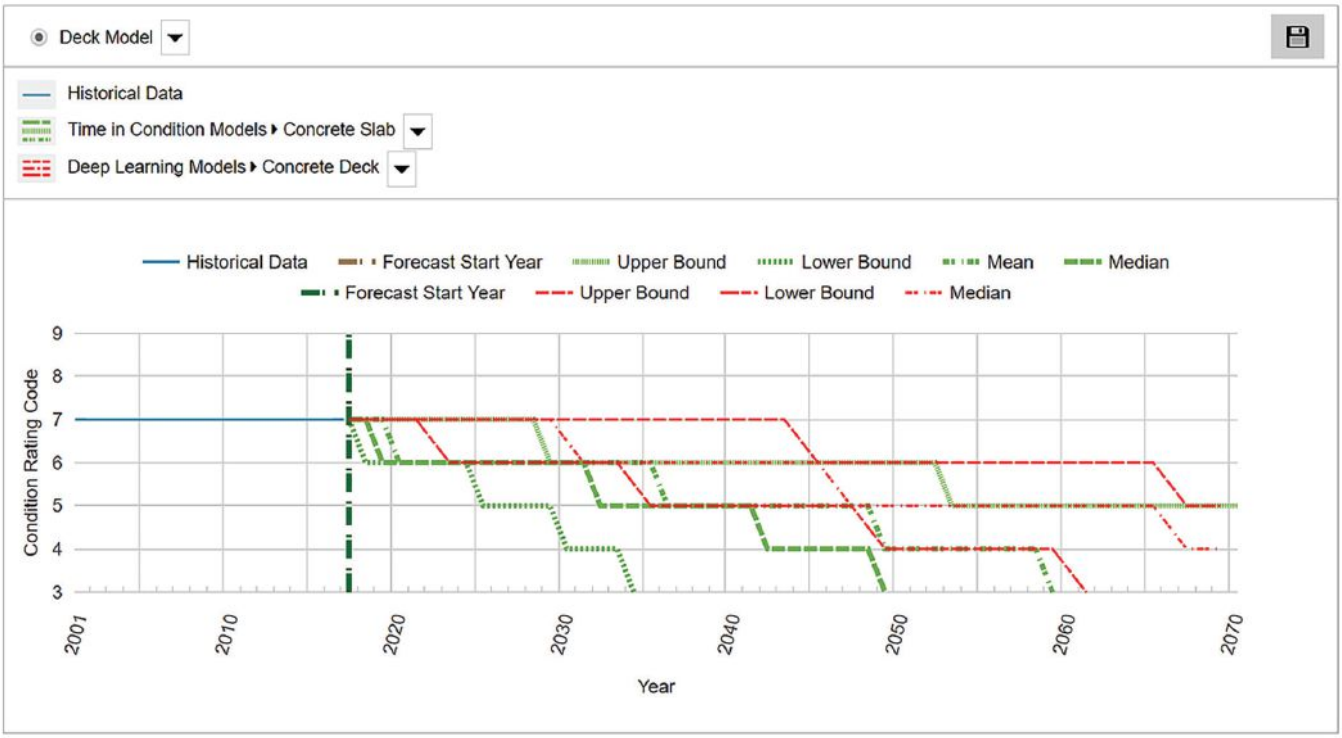


Figure 3. Three new models are now available to forecast bridge deck conditions.

Version 2 climatic data, to forecast bridge deck conditions.

These models are intended to be used for research. Improvements are being planned to increase their forecasting accuracy and will be included in future releases of InfoBridge. Documentation for the models is published in the Library module of InfoBridge. Technical papers describing the models are currently being prepared for publication in peer-reviewed journals.

Design and Construction Data

As-built plans for 1600 bridges have been transposed into searchable data tables (**Fig. 4**). This effort aims to help users better understand the impacts of design and construction practices on bridge performance. Currently, the tables are being restructured to enable more consistent querying and improve the accuracy of research conclusions drawn from InfoBridge design and construction data. A stand-alone interface is being designed to facilitate the inclusion of additional data by bridge owners.

Nondestructive Evaluation Data

InfoBridge has a tabular interface that provides access to downloadable files containing nondestructive evaluation (NDE) bridge data and metadata. In addition, tools for NDE data visualization are included under the *Bridge Information/Graph* tab (**Fig. 5**). Future versions of InfoBridge will include a tool to display the progression of bridge deck deterioration for bridges with multiple rounds of NDE assessments.

EDITOR'S NOTE

InfoBridge is intended for research, but its data and tools can also be useful to bridge designers. To access InfoBridge, visit https://infobridge.fhwa.dot.gov. *For more information on the portal and the LTBP program, visit* https://highways.dot.gov/long-term-infrastructure-performance/ltbp/long-term-bridge-performance, *or contact Jean Nehme at (202) 493-3042 or jean.nehme@dot.gov.*

Figure 4. Bridge design and construction data for a project in the LTBP database.

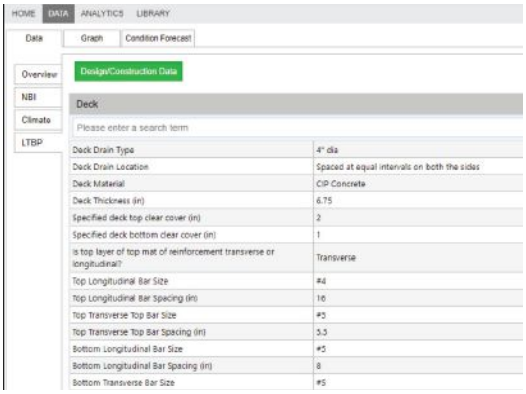
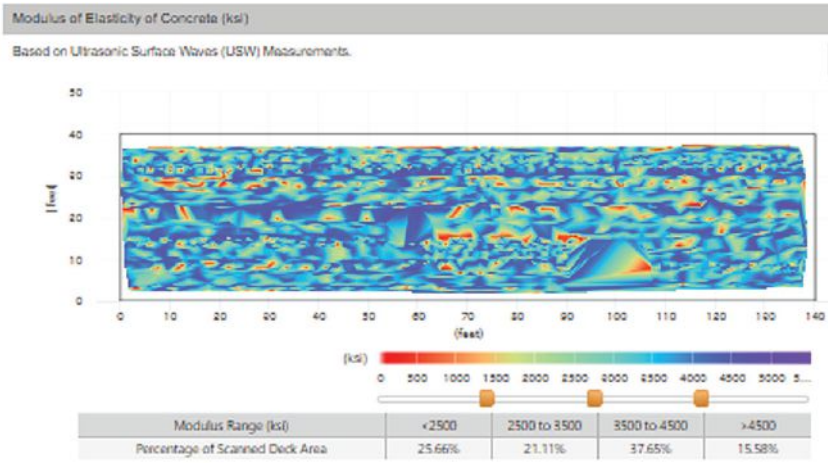


Figure 5. Nondestructive evaluation contour plots with dynamic value ranges enable users to adjust the ranges and instantly display the resulting change in values.



A PROFESSOR'S PERSPECTIVE

Elevating the Role of Materials Science in Bridge Engineering Education

by Dr. Kyle A. Riding, University of Florida

As a new decade begins, we are inclined to reflect on progress we have already made and plan and project our efforts going forward. For those of us working in education, this is also an opportune time to assess how we can best prepare our students and new engineers for future success.

A Century of Progress

In the bridge community, I think it is especially useful to highlight the enormous progress we have made over the last century or more. The timeline in **Fig. 1** shows a few bridges that I view as technological pioneers. While there are many important and noteworthy bridges in communities around the world, I selected these bridges to highlight because they represent early examples of technologies that have made bridges faster to construct, lighter, more cost-effective, more durable, and—most importantly—safer.

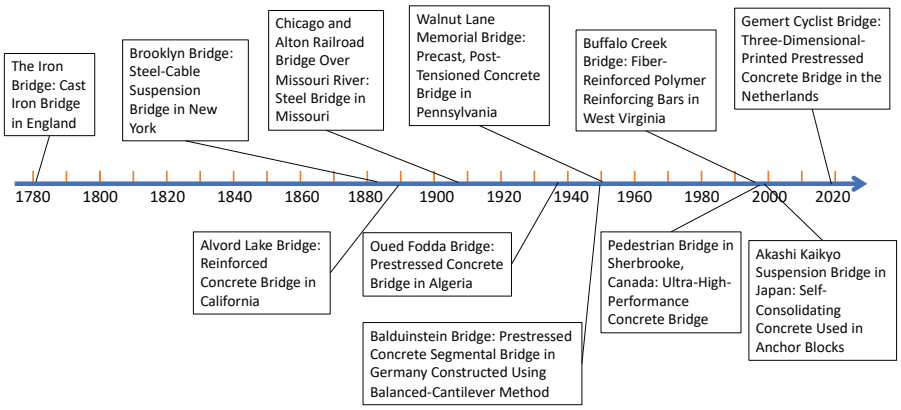


Figure 1. Timeline showing the year of construction for bridges that introduced innovative technologies. All Figures and Photos: Dr. Kyle A. Riding.

These bridges also have something else in common. Each was constructed using new structural engineering designs enabled by the development of new materials. For example, the Chicago and Alton Railroad Bridge over the Missouri River and the Brooklyn Bridge in New York City took advantage of cheap and plentiful steel made possible by the new Bessemer steel process. The Oued Fodda Bridge in Algeria, the Balduinstein Bridge in Germany, and Walnut Lane Memorial Bridge in Philadelphia, Pa., were showcases for the benefits of prestressing concrete in bridge structures. While the concept of prestressing had been theorized for several decades, higher strength concrete and steel made it practical for these bridges.[1]

In recent history, bridge designers have focused on simpler construction and greater durability by using high-performance materials, including fibers in concrete mixtures. Going forward, bridge construction can be semiautomated by leveraging three-dimensional printing, machine learning, and concrete with tailored flow and strength-gain properties.

Pioneering engineers make progress by combining a solid grasp of fundamental engineering mechanics, new knowledge in materials science, and practical construction in their designs. One such pioneer was Eugène Freyssinet. While other researchers had previously tried to embed tensioned wires into concrete to introduce prestressing, Freyssinet successfully designed modern, functional prestressed concrete members for the Oued Fodda Bridge project. He accomplished this by using high-strength steel and concrete and by calculating concrete creep and shrinkage losses.[1] He also pioneered long-line prestressing beds still used by many precast, prestressed concrete plants to cost effectively fabricate bridge members.[2]

When pioneering technology is shown to work well and be economical, it can soon become commonplace. For example, in Columbia, ultra-high-performance concrete (UHPC) proved to be so economical and successful in a four-span, 361-ft-long segmental, post-tensioned pedestrian bridge constructed in 2017 in Medellín, that a second UHPC segmental, post-tensioned bridge, two spans and 98 ft long, was built in Manizales in 2018.[3]

Preparing Engineers to Build the Bridges of Tomorrow

As bridge technologies have become increasingly sophisticated, engineers have needed greater knowledge and control over their materials. Structural engineers in the United States are now called upon to evaluate material options and submittals and to be the final word on their suitability to meet the performance expectations of owners.

However, even as the need for knowledge of materials has increased, civil engineering programs have come under pressure by state legislatures and university administrations to reduce credit hour requirements.[4] For example, in 2018 the University of Florida reduced the number of credit hours required to obtain a bachelor of science degree in

This segmental, post-tensioned pedestrian bridge in Medellín, constructed in 2017, is the first ultra-high-performance concrete bridge in Colombia.

civil engineering from 131 to 128; many other universities in the United States have made similar reductions.

Today, undergraduates majoring in civil engineering typically take only one course on materials, with a couple of weeks at most dedicated to each civil engineering material. Engineers certainly need a deeper knowledge base than that to make critical decisions about material performance in bridges and to make breakthroughs in new bridge technologies. So how do we elevate the level of materials science knowledge in the structural engineering community? Here are a few suggestions.

First, structural engineering master's programs should include a materials science course as a core competency. Partly as a response to dwindling credit hour requirements at the undergraduate level, master's degrees have become the de facto entry-level degree for many structural engineering firms. Adoption of a materials science course as a master's requirement could help raise the knowledge level of engineers responsible for specifications, acceptance, and design. I believe that a concrete materials or composite materials course would give structural engineering students a good foundation for future learning on the topic. If your university does not offer a graduate course in structural materials, it may be possible to take an online course through another university's distance education program and transfer the credits to your university.

Universities looking to train faculty to teach a graduate concrete materials course will find that the annual professors' workshop on concrete materials, pavements, and structures cosponsored by the American Concrete Institute (ACI) Foundation and the Portland Cement Association (PCA) Education Foundation is an excellent place to start.

A second way to increase materials science knowledge is by using professional development hour (PDH) requirements as an opportunity to learn. Peers and new hires should also be encouraged to participate. Excellent materials science courses are offered by several organizations. Here are some prominent offerings:

- The PCA course *Design and Control of Concrete Mixtures* is offered regularly. For more information, visit *https://www.cement.org/Learn/education/design-and-control-of-concrete-mixtures-course*.
- The National Ready-Mixed Concrete Association (NRMCA) offers a series of courses on concrete technology with different levels of certification as a concrete technologist. NRMCA courses on concrete fundamentals, concrete durability, and other topics are offered around North America. For more information about the program, schedule, and locations of courses, visit *https://www.nrmca.org/products/conferences.asp*.
- The ACI University has a wide range of web-based courses with continuing education credits. To learn about ACI courses on a variety of concrete topics, visit *https://concrete.org/education/aciuniversity.aspx*.
- Local ACI chapters often host speakers at events that include a dinner or other activity. To find your local chapter, visit https://concrete.org/chapters/findachapter.aspx.

My final suggestion is to encourage new hires to use free web-based resources. For example, free lectures by world-renowned professors and engineers have been posted on YouTube. Also, ACI offers free videos of excellent presentations on a variety of new materials and structural concepts, including recent technology developments at https://concrete.org/education/freewebsessions/completelisting.aspx.

Conclusion

Bridge engineering has come a long way in the past century. Much of this progress was steered by clever engineers using materials science to enable new bridge load-support systems. Materials science will continue to yield great advancements in bridge design and durability and should be an integral part of the formal and continuing education of bridge engineers.

References

1. Sanabra-Loewe, M., and J. Capellà-Llovera. 2014. "The Four Ages of Early Prestressed Concrete Structures." *PCI Journal* 59 (4): 93–121.
2. Freyssinet, E., and J. Seailles. 1930. *France Patent No. 680547.*
3. Núñez, A., J. Patiño, S. Arango, and W. Echeverri. 2019. Review on First Structural Applications of UHPC in Colombia. Second International Interactive Symposium on UHPC, 1–10. https://www.extension.iastate.edu/registration/events/2019UHPCPapers/UHPC_ID118.pdf.
4. Myers, J.J. 2018. "Developing a Greater Feel for Structural Engineering." *ASPIRE* 12 (2): 36–38.

EDITOR'S NOTE

Technical institutes play a vital role in promoting continuing education opportunities for new and experienced engineers. PCI offers eLearning modules on many transportation topics, including "Materials and Manufacturing of Precast, Prestressed Concrete." See page 41 of the Winter 2020 issue of* ASPIRE® *for additional information and a sampling of other course options.

PCI Offers eLearning Modules

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

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

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

All Transportation courses on the PCI eLearning Center are completely FREE and include PDHs or other continuing education credits.

Start with PCI eLearning Series T100 Courses

Preliminary Precast, Prestressed Concrete Design (T110)

Materials and Manufacturing of Precast, Prestressed Concrete (T115)

Design Loads and Load Distribution (T120)

Additional courses are available at www.pci.org/eLearning

This web-based training course was developed by the Precast/Prestressed Concrete Institute (PCI) for the Federal Highway Administration (FHWA) through a contract with the American Association of State Highway and Transportation Officials (AASHTO).

U.S. Department of Transportation Federal Highway Administration

AASHTO The Voice of Transportation

PCI Precast/Prestressed Concrete Institute

CONCRETE CONNECTIONS

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.ntsb.gov/investigations/AccidentReports/Reports/HAR1902.pdf
This is a link to the National Transportation Safety Board (NTSB) Highway Accident Report for the 2018 pedestrian bridge collapse in Miami, Fla., which is referenced in the Editorial on page 2 and the Perspective article on page 50.

https://www.ntsb.gov/news/events/Pages/2019-HWY18MH009-BMG.aspx
This is a link to information presented at the NTSB meeting on October 22, 2019, to determine the probable cause of the 2018 pedestrian bridge collapse in Miami, Fla., which is referenced in the Editorial on page 2 and the Perspective article on page 50.

https://www.fhwa.dot.gov/bridge/pubs/hif18046.pdf
This is a link to FHWA's *Manual for Refined Analysis in Bridge Design and Evaluation*, which is mentioned in a Perspective article on page 8.

https://www.al.com/news/2020/01/alabama-your-700-million-i-5920-bridge-through-birmingham-is-ready.html
This is a link to photos and a video of the Interstate 59/Interstate 20 bridges through Birmingham's central business district. The bridges are featured in the Project article on page 10 and the Concrete Bridge Technology article on page 30.

https://www.pci.org/PCI_Docs/Design_Resources/About_Precast/PCI%20Zone6%20Curved%20Spliced%20Girders.pdf
This is a link to information on PCI curved and straight precast concrete U-girders Curved U-girders are mentioned in the Project article on page 20 and the AASHTO LRFD article on page 48.

https://www.youtube.com/watch?v=Hmk6W27r92w&feature=youtu.be
This is a link to a video and renderings of the Mid-Coast Corridor Transit Project, a light-rail transit project in Southern California. A Project article on the Genesee Avenue Viaduct, which is part of this project, begins on page 20.

https://www.fhwa.dot.gov/bridge/loadrating/pubs/hif18061.pdf
This is a link to the Federal Highway Administration (FHWA) report *Concrete Bridge Shear Load Rating Synthesis Report*. Shear load rating of an existing bridge is the focus of a Concrete Bridge Technology article on page 24.

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Structural Design Based on Strut-and-Tie Modeling

by Dr. Oguzhan Bayrak, University of Texas at Austin

Structural design and detailing of reinforced and prestressed concrete members and structures requires a thorough understanding of load transfer mechanisms. Since its introduction in the late 19th and early 20th centuries,[1,2] strut-and-tie modeling (STM) has enabled structural designers to appropriately visualize the transfer of loads from their points of application down to structural foundations. This explicit visualization then helps guide the associated structural detailing based on key load transfer mechanisms. In the United States, the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications* has included STM design provisions since 1994. To illustrate the power of STM, this article discusses an integral superstructure and substructure connection and how STM provides insights on load transfer.

Options for Modeling Load Transfer

Figure 1 shows three lines of prestressed concrete U-beams rigidly connected to an integral straddle bent. For the sake of simplicity, let us assume symmetrical loading in forward and back spans that frame into the straddle cap and all loads are vertical. Live and dead loadings on the deck result in substantial beam reactions that need to be transferred to the cap and through the cap to the columns. Where beam lines 1, 2, and 3 frame into the straddle cap, the loads (U-beam reactions) need to be transferred to the cap in a realistic manner. To envision this transfer of vertical loads to the cap at beam line 2, the designer has three options:

Option 1: Apply beam reaction at nodes A and B
Option 2: Apply beam reaction at nodes C and D
Option 3: Apply beam reaction at nodes A, B, C, and D

It must be emphasized that the detailing of the end of the U-beam must be consistent with the assumed transfer of loads into the cap, as noted in the following discussion.

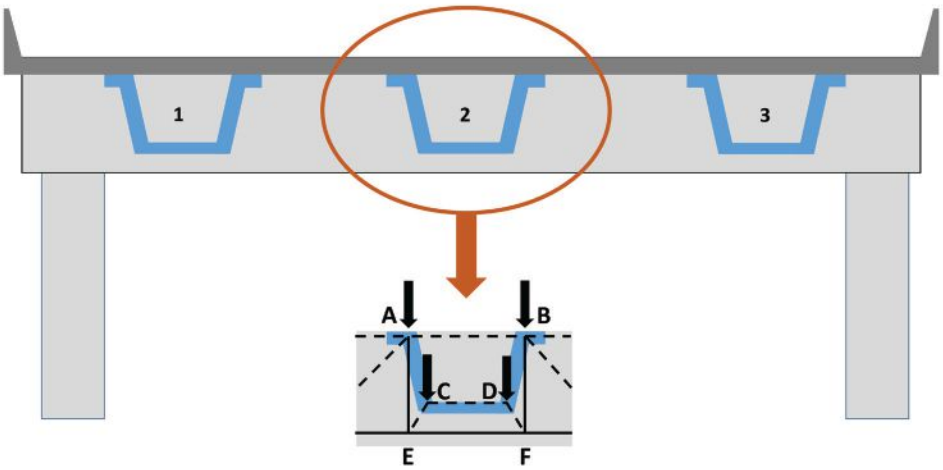
Option 1—Reaction Applied at Nodes A and B

The reaction for beam 2 is divided in half and applied at nodes A and B. Because no loads are being applied at nodes C and D, the demand for the hanger reinforcement (ties AE and BF) is zero. This load transfer mechanism calls for the U-beam design and detailing to be consistent with the assumed load application to the straddle cap. Therefore, a substantial quantity of hanger reinforcement must be provided within the prestressed concrete U-beam at or within the immediate vicinity of its connection to the straddle cap. Picking up the entire reaction with stirrups placed within the U-beam webs at the face of the cap presents a substantial challenge for detailing because the hanger reinforcement provided must be centered on the vertical tie envisioned within the end region of the U-beam. Structural detailing of the U-beam-to-straddle-cap connection must accommodate the assumed load transfer mechanism, which is hard to accomplish with standard details that involve anchorage of U-beam reinforcement into the straddle cap and continuity post-tensioning passing through the cap, in addition to providing a substantial amount of hanger reinforcement within the U-beams. The required detailing renders this option impractical in most situations, and the effectiveness of such a load transfer mechanism is questionable.

Option 2—Reaction Applied at Nodes C and D

For this option, the reaction for beam 2 is again divided in half but is applied at nodes C and D. This concept for load application assumes that the inclined webs of beam 2 are supported directly beneath the bottom flange; therefore, the support reaction introduces compression into the webs of the U-beam. This assumption is overly conservative in most common structural connection details. Hanger reinforcement sufficient to transfer 100% of the beam reaction must be provided within the cap where the superstructure meets

Figure 1. Strut-and-tie modeling (STM) for integral superstructure-to-substructure connection for U-beams framing into a straddle cap. Figure: Dr. Oguzhan Bayrak.



the substructure. Furthermore, the reinforcement needs to be centered on ties AE and BF. To resist the two forces from the beam reaction (notably, in this case, the beam reaction is presumed to be significant), a substantial quantity of hanger reinforcement must be provided in the cap where the bottoms of the slanted webs of the U-beam intersect the straddle cap. Although this assumption is conservative, it may result in an excessive amount of hanger reinforcement in the cap; detailing of this reinforcement may present constructability challenges.

Option 3—Reaction Applied at Nodes A, B, C, and D

For this option, the reaction for beam 2 is divided into four equal vertical loads and applied at nodes A, B, C, and D. With this assumed load application, 25% of the beam reaction would transfer to each of the diagonal compression struts framing into nodes A and B, with the remaining vertical load going into struts CE and DF. A compression strut is also required between C and D. The vertical components of the forces in struts CE and DF need to be picked up by ties AE and BF. Option 3 results in a more practical quantity of hanger reinforcement that is easier to detail and construct than either of the other options with half the force in the tie reinforcement in the U-beam as for Option 1. It is also worth noting that the compressive forces in struts CE and DF influence the tie-force distribution along the length of the longitudinal tie at the bottom of the cap. With respect to U-beam design, one-fourth of the total beam reaction must be carried by the stirrups within each web of the U-beam, such that load application at the top nodes A and B can be justified. As required in the AASHTO LRFD specifications, the sectional shear design of the U-beams must be done using provisions based on simplified Modified Compression Field Theory (MCFT) (AASHTO LRFD specifications article 5.7.3).[3] The quantity of stirrups required in the U-beam, on the basis of MCFT design, is typically sufficient to satisfy the application of one-half of the beam's reaction to the top nodes (A and B). Furthermore, the skin reinforcing bar detailing of the webs and extension of those bars into the cap, along with the roughened surfaces of the webs of the precast concrete U-beam, all help justify dividing the beam reaction among the four nodes. In my view, and in most structural details I have seen to date, this balanced approach constitutes a good structural engineering solution.

Once the load transfer model for the U-beam-to-straddle-cap connection is selected for beam 2, this model can be repeated for the other U-beam-to-straddle-cap connections. The complete truss visualized in the straddle cap can then be used to transfer the loads into the columns and subsequently into the foundations.

Conclusion

The superstructure-to-substructure connection discussed in this article illustrates the decision-making process in STM. As can be observed in the example, STM drives a designer to consider the implications of model selection, construction methods, and details. In structural engineering, we do not get something for nothing. We must reconcile how loads are transferred from their points of application into the concrete component, then from one element to the next, and eventually to the foundations. In my view, the fact that STM stimulates structural designers to think through the load path is the most important attribute of this technique. As we aspire to design concrete bridges to last no less than a century, STM plays a role in the design of unique structures, critical bridge components, and the bridge infrastructure that will serve our communities.

References

1. Ritter, W. 1899. "Die Bauweise Hennebique" (The Hennebique System). *Schweizerische Bauzeitung* 33 (7).
2. Mörsch, E. 1902. *Der Eisenbetonbau, seine Theorie und Anwendung* (Reinforced Concrete: Theory and Application). Stuttgart, Germany: K. Witter.
3. American Association of State Highway and Transportation Officials (AASHTO). 2017. *AASHTO LRFD Bridge Design Specifications*, 8th ed. Washington, DC: AASHTO.

Why Didn't They Just Close the Road?

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

After the release of the National Transportation Safety Board (NTSB) report on the Florida International University (FIU) pedestrian bridge collapse,[1] many have asked, "Why didn't they just close the road?" While this article will not attempt to answer this event-specific question, we will provide the perspective of two former state bridge engineers (the authors) about their decision-making process when faced with similar challenges regarding whether to recommend closure of a roadway.

Our Preeminent Responsibility

As engineers, our first responsibility is to protect the public safety, health, and welfare. This is spelled out in most states' authorizing legislation that regulates the practice of engineering as well as in most engineering codes of ethics.

In 1914, the American Society of Civil Engineers (ASCE) adopted its first code of ethics to be used as a model for conduct for ASCE members. In the current version of this code, the first point under Canon 1 is the following statement:[2]

> *Engineers shall recognize that the lives, safety, health and welfare of the general public are dependent upon engineering judgments, decisions and practices incorporated into structures, machines, products, processes and devices.*

Assessing a Potential Closure Situation

As state bridge engineers, we both faced situations that required making the recommendation to close a given roadway or bridge due to findings from an in-service or construction inspection, or some other event, to ensure public safety, health, and welfare were maintained. When you are in that type of situation, bridge technical staff gather data and calculate scenarios:

- Let's look at what changed since the last inspection.
- Let's look at the plans and details.
- Let's look at the existing calculations.
- Let's make some new calculations.
- Let's use our manuals of practice to reevaluate the damage or deterioration.
- Have we looked at the distress and quantified the accumulated damage to the member(s)?
- What is the consequence of load exceedance?
- Is there public access on or below this bridge?
- Who owns the road on and/or below the bridge?
- Can the structural support system be changed—for example, by installing temporary supports—to mitigate the concern?
- Can traffic be shifted or loads reduced by a truck detour to temporarily mitigate the risk?

When you are in that same type of situation during construction, procurement managers and governmental legal counsel gather different types of data:

- Who has the contract authority to stop the construction?
- Was the contractor proceeding at their own risk?
- Can we suggest that the contractor close the project?
- Are you questioning the contractor's means and methods?
- What is the standard of care defined in the procurement document?
- Who owns this problem?
- Is the bridge fit for service, and can it meet its intended purpose?
- How is the insurance coverage addressed?
- How is the performance bond addressed?
- If this is a design-build job: Do we need to wait for others to propose the fix?
- If we do not own this problem yet, can we let their insurance carrier sort this out?

Sometimes, the engineers involved will conclude with complete certainty that the bridge needs to be closed because the engineering calculations and data clearly support that decision. Still, putting that decision into action isn't always as easy as it sounds.

Pushback

When making a recommendation to close a bridge, state bridge engineers inevitably hear the following types of objections from those around or above them:

- We can't close that road—10,000 vehicles per day use that facility.
- That's the only route to the school.
- That will create an hours-long detour.
- Closing the bridge will make us look like we don't know what we are doing.
- Commissioner So-and-so isn't going to be happy—that road is on his way home.
- We're likely to endanger more people with the traffic detour.

These objections are often followed by second-guessing:

- Aren't your capacity calculations always on the conservative side? Isn't there always more capacity than what we predict?
- Are we really going to see the live loads you used in your computations?

- Hasn't the structure been this way for several days, months, or years? Why close it now?
- We've had problems like this in the past and nothing bad happened. Why would we expect this to be any different?

In his October 28, 2019, concurring statement about the FIU bridge collapse (see the statement following this article), NTSB vice chairman Bruce Landsberg rightfully identified that the recommendation to close the roadway would have been an "unpopular step." Indeed, "unpopular" is probably an understatement at a time when most motorists expect to be able to go anywhere at any time with few impediments. In recent years, crashes involving 50 to 60 cars at a time during inclement winter weather illustrate this point.

For a state bridge engineer, advising the department of transportation (DOT) leadership that a major in-service roadway or bridge needs to be closed may even have career-limiting repercussions. That may be unfair, but the reality is you only get to go to that well so many times before you lose credibility. The traveling public and elected officials expect that the transportation owner will do everything in its power to keep traffic moving. Closing a bridge or roadway or shutting down a construction project is counter to that expectation.

Elected officials are often sensitive to how their constituents will react when their usual commute home from work gets disrupted by an unexpected transportation asset closure. DOT leadership can also be sensitive to the general message a specific closure sends to the traveling public: If this bridge had to be closed, what about other ones like it? These pressures are felt by everyone involved in these decisions. Ultimately, it is be better to be asked why you took preemptive action that might not have been needed, rather than trying to explain after a tragedy why you didn't; however, in the decision-making moment, this sentiment is of little consolation.

Who Makes the Call?

State bridge engineers typically do not have the direct authority to close any and all bridges in the state. The DOT may also lack the legal authority to close a bridge. Instead, this authority often lies with local law enforcement, local governments, or owners of the bridge. In these instances, the DOT must rely on these local officials or owners to take appropriate action.

Sometimes, convincing local officials of the urgency of the recommendations is challenged by their complacency or lack of understanding—"I drove over that bridge yesterday. What could possibly be wrong?" Instances like this call for strong leadership from not only the state bridge engineer but also those above the engineer in the chain of command.

Conclusion

As the recommendation for closure is escalated, the individual with the ultimate legal authority may be on the frontlines of a media firestorm when the closure is announced and implemented. To withstand this public scrutiny, the individual in charge requires not only courage but also complete confidence in the recommendation being brought to them by the state bridge engineer. While it may be a simple thing to say, "Just close the bridge," implementation of this decision can often be a challenge. Again, you only get to go to that well so many times.

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. https://www.ntsb.gov/investigations/AccidentReports/Reports/HAR1902.pdf.
2. American Society of Civil Engineers (ASCE). 2017. Code of Ethics. https://www.asce.org/code-of-ethics.

EDITOR'S NOTE

The entire text of the concurring statement by NTSB vice chairman Bruce Landsberg (see pp. 106–107 of Reference 1) is reproduced below. The text has been reformatted from the report to fit the page.

National Transportation Safety Board (NTSB) Member Statement

Vice Chairman Landsberg filed the following concurring statement on October 28, 2019.

Concurring Statement of Vice Chairman Bruce Landsberg re Miami Pedestrian Bridge Collapse

A bridge-building disaster should be incomprehensible in today's technical world. We have been building bridges in this country for over two hundred years, and long before that in other parts of the world. The science should be well sorted out by now—and for the most part, it is. The investigation clearly highlighted basic design flaws and *a complete lack of oversight by every single party* that had responsibility to either identify the design errors or stop work and call for a safety stand-down, once it was clear that there was a massive internal failure.

The "what" is very clear but the "why" is more elusive. Despite the public's anger, distress, and disappointment, none of the responsible organizations had any intent for this tragic event to occur or to cause any injury or loss of life. Sadly, good intentions do not suffice for competence and diligence.

Engineering schools will use this as a landmark case study for years—and they should. The Engineer of Record (EOR) employed by FIGG Bridge Engineers, Inc., was experienced, but his calculations were erroneous. Reflection on this event should go far beyond merely a technical review. The checks and balances that were required by the Florida Department of Transportation (FDOT) and American Association of State Highway and Transportation Officials guidance and incumbent upon Louis Berger (LB), the peer-reviewing organization, were completely lacking. LB lowered their bid to review the project by 43 percent in order

to get the business, but also reduced the scope of the review. The reason given was there wasn't enough money in the project to cover their efforts. That's both disingenuous and unconscionable. It also was in violation of FDOT's requirement that there be an independent second set of eyes to review everything—not just what was economically convenient.

It is likewise incomprehensible and unethical that LB would even bid on a job for which they lacked the requisite qualifications. That FDOT, which was supposed to review the plans, did not know, or think to ask, about their qualifications is more than just an oversight. It's just plain sloppy. Ditto for FIGG. FDOT claimed a technical error on the FDOT website and then, after the collapse, fabricated a disclaimer that they are not responsible for the data that they post. That's unacceptable in my view—either ensure the information is accurate or don't post it.

The bridge was not properly designed, and there was no qualified oversight on that design. When the inevitable began to happen—a creeping, catastrophic material failure, nobody did anything, despite what NTSB Chairman Sumwalt accurately described as the "bridge screaming at everyone that it was failing." Why?

Once the cracking became evident, *not one of the organizations involved was willing to take the essential and unpopular step to call a halt* and close the road. This is similar to the circumstances of the space shuttle Challenger disaster where the decision was made to launch in extremely cold weather. The engineers recommended against it because the O-Rings that were critical to fuel system integrity would be operating outside their design parameters. Rationalization, optimism, and schedule pressure contributed to what has been described in management training circles as "Group Think." Strong and confident personalities persuade everyone that everything will be OK. Despite misgivings and technical expertise that advise against such action, the team moves forward as a group.

It appears that the same mindset was in play here, in every organization: FIGG, LB, MCM (the construction company), Bolton Perez (the engineering firm overseeing the bridge construction), FDOT, and finally, Florida International University. It also appears that every organization absolved themselves of responsibility by rationalizing that if the EOR says it's OK, it must be OK, and if anything bad happens—it's on him. That is not the intent of peer review or safety oversight, and certainly fails the system of checks and balances in place to prevent catastrophes like these.

We've learned this the hard way too many times in transportation modes. The NTSB's stated role is not to lay blame, but some would say that's exactly what we do when we apportion causation. The human failing that affects all of us is complacency. It is not a term the NTSB uses often, but in my opinion, it is present in nearly every accident and crash. We are creatures of habit, and when we become comfortable through long repetitive experience, the guard often comes down—periodically with disastrous results. This is precisely what safety management systems are designed to prevent—to trap errors in process before they become catastrophes. While disasters may be perfectly clear in hindsight, the best organizations take proactive measures—constantly. Schedule pressure, economics, overconfidence, and complacency all work to counter good intentions and too often create tragedy.

It is my fervent hope that the organizations involved will take the NTSB recommendations seriously and quickly implement them. The lives lost and the families disrupted deserve at least that much.

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